MODELLING OF COASTAL OVERWASH

Nguyen Xuan Tinh

LUND - 2006

TVRL 2006:1

Modelling of Coastal Overwash

Review and application of state-of-art empirical, numerical and analytical methods

Nguyen Xuan Tinh

Department of Water Resources Engineering

Supervised by

Chantal Donnelly (Doctoral student)
Professor Magnus Larson

PREFACE

Nguyen Xuan Tinh

Lund, January 2006

ABSTRACT

Coastal overwash is the flow of water and sediment over the crest of the beach that does not directly return to the water body, such as the ocean, sea, bay or lake, where it originated. There are two different phases of overwash, runup overwash and inundation overwash. Washover is the sediment transported and deposited inland by overwash process. Coastal overwash may occur either during storm conditions such as hurricanes or severe storm events or non-storm conditions such as extreme tide levels or seiches in lakes.

The modelling of coastal overwash and the prediction of overwash occurrence are significant for coastal authorities and local managers to give evacuation warnings to people who live near the coast. Estimates of washover volume are useful for planners and engineers to mobilise equipment for cleaning up sediment after a storm. Profile change estimations are useful for planners creating "buffer zones" indicating where it is safe to build houses. Modelling overwash is also important for evaluating sediment budgets of barrier islands, especially where these islands are migrating due to overwash.

Beach response morphologies can be divided into five types, that is, crest accumulation, dune lowering, dune rollback, dune destruction, and barrier rollback. However, in reality it is not easy to identify what kind of morphologic response the beach has experienced, and in some cases a combination of beach response types are observed.

A new empirical formula for overwash volume was derived by taking into account the excess runup height over the beach crest and the duration of the overwash event. This formula yields predictions that are comparable to the measured field data. However, it needs more testing with other data sets before its general applicability has been validated.

A number of modelling studies were carried out in order to simulate the overwash processes. This study evaluates different levels of coastal overwash modelling: empirical, analytical, and numerical (SBEACH) models based on a number of high-quality, detailed data sets which were collected in the United States. The latest version of SBEACH with a new algorithm to simulate overwash flow either by runup or

inundation overwash was employed. Flow on the backside of barrier was considered in the enhanced model including the effects of lateral spreading and infiltration. It was shown that the simulated beach profile evolution was in satisfactory agreement with measured post-storm profiles.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Chapter 1: INTRODUCTION

1.1   BACKGROUND

Coastal overwash, a serious and widespread issue affecting barrier coasts around the world in general and the United States in particular, results from complex geologic and oceanographic processes, such as extreme storms or hurricanes, changes in sediment supply at the coast, and sea-level rise. Overwash is the flow of water and sediment over the crest of the beach that does not directly return to the water body (ocean, sea, bay, or lake) where it originated. Washover is the sediment transported and deposited landward by the overwash process. Overwash occurs when wave runup level or water level including storm surge exceeds the beach crest height. Overwash may also occur during non-storm conditions caused by extreme tide levels or seiches in lakes which exceed the beach crest (Schwartz 1975, Morton et al. 2000) or due to severe floods from lagoon to ocean as observed in Hoa Duan coast in the Thua Thien Hue province, Vietnam during a large flood in 1999. There are two different phases of overwash, runup overwash and inundation overwash (Sallenger et al. 2000, Donnelly et al. 2004). Runup overwash occurs when the wave runup height is higher than the dune or barrier island crest. Inundation overwash occurs when the water level exceeds the beach crest. Inundation overwash may cause the destruction of dunes and man-made structures on barrier islands.

Each year in the United States, several hurricanes and winter storms impact the coastline. Therefore, overwash is common on the barrier islands of the Atlantic Ocean and Gulf of Mexico coasts. Overwash also occurs on the coasts of the Great Lakes, and around the world on low-profile coasts of the mainland, on spits, and on gravel or shingle beaches. The occurrence of overwash is expected to increase because of sea level rise and diminished sediment supply along many coasts (Titus, 1998 and Larson et al. 2004). The capability to quantitatively predict and simulate overwash and washover deposits is just beginning to emerge (Donnelly et al. 2005). Prediction of overwash occurrence can provide valuable planning information for local authorities, emergency services, and coastal residents.

This study evaluates three different types of coastal overwash modelling: empirical, analytical, and numerical models. Williams (1978) and Tanaka (2002) both developed empirical formulas to estimate washover volume using laboratory data and field data, respectively. These formulas will be tested further using field data sets from the United States.

An analytical model to estimate magnitude of dune erosion, beach height reduction and dune rollback, originates from a mathematical model describing the main governing physical processes (Larson et al. 2005). The method involves many simplifications, but to a satisfactory level of accuracy, essential features of coastal overwash may be derived, isolated, and more readily comprehended than in complex approaches such as numerical and physical modelling.

The numerical model used in this study, SBEACH (Storm-induced BEAch CHange), was developed by Larson and Kraus (1989) to calculate profile change resulting from erosive storm conditions. Overwash was initially added to the model by Kraus and Wise (1993). This algorithm was updated by Larson et al. (2003) to provide a more physically based description of overwash processes and the model was able to successfully indicate the depth and inland penetration of sediment as well as beach face recession and beach crest height reduction; however, these algorithms only considered runup overwash. Recently, a new algorithm has been added to the SBEACH overwash model to calculate inundation overwash, and the algorithm to calculate the hydrodynamics on the backslope for both runup and inundation overwash has also been improved (Donnelly et al. 2005). This new model had only been tested for a limited number of storm events, so this study introduces a more comprehensive evaluation of the overwash algorithms using previously published data sets (Eiser et al. 1991, Stone et al. 2004, and Larson et al. 2004).

1.2 PROBLEM STATEMENT

Coastal overwash occurs primarily during severe storms. In the United States, overwash is most common during hurricanes and the severe winter storms known as northeasters. Washover can be transported more than a kilometre inland and on barrier islands in their natural state, contributes to the maintenance of an island's sediment budget during severe storms. On populated barrier islands, overwash may damage or destroy bridges and roads or simply render roads impassable due to large depths of deposited sand. Overwash can lead to coastal breaching when the overwashing waves and currents

open new inlets on barrier islands. An example of washover deposits can be seen in Fig 1.1a and Fig. 1.1b which shows Navarre Beach, Florida, before and after Hurricane Ivan, 2004, made landfall. The morphological change suggests that the island was inundated. Fresh deposits of sand are clearly visible pushing into Santa Rosa Sound. Note the reduction of the beach width.

The impacts of coastal overwash may cause significant hazards to buildings and infrastructure that are built too close to vulnerable shorelines. Societal costs, in dollars spent, can be very large.

Fig. 1.1a: Navarre Beach, FL before the Hurricane Ivan. Image from the USGS website; *http://coastal.er.usgs.gov/hurricanes/ivan/photos/florida.html*

Fig 1.1b: Overwash occurred during Hurricane Ivan, Sep 16th 2004. Navarre Beach, FL. Image from the USGS website; *http://coastal.er.usgs.gov/hurricanes/ivan/photos/florida.html*

Engineers must be able to predict where and how much coastal change will occur in order to locate new construction landward of coastal change hazards or to develop evacuation plans. Developing this predictive capability requires quantifying how coasts respond to extreme storms.

1.3   OBJECTIVES OF STUDY

The main objectives of this study are to quantify the deposited volume of sediment and to predict how a beach profile will change due to an overwash event. Three different

levels of model, that is, empirical, analytical, and numerical have been investigated. Empirical overwash models and the numerical SBEACH model for simulating beach profile evolution (both the latest published and new versions) are compared with new measured data sets. Development of analytical models is still ongoing, but they are reviewed for sake of completeness.

To the author's knowledge, the empirical models (Williams 1978, Tanaka 2002) have only been tested on limited data sets (a laboratory model using regular waves and an approximate field volume estimate taken from aerial photographs, respectively). The numerical model will give more detailed and accurate predictions of beach profile change, the penetration of washover sediment inland, and the risks of barrier breaching. The new algorithms within SBEACH to calculate runup overwash, inundation overwash, and the movement of water on the back slope have, to date, also only been tested on limited data sets. A further objective of this study was to validate the use of these algorithms on a wider range of data.

1.4   APPROACH AND METHODOLOGY

The approach taken in this study is developed based on the proposed objectives. The main thrusts of the study involve calculating volumes of overwash using two empirical formulas formulated by Wiiliams (1978), and Tanaka (2002), and modelling beach profile change due to overwash using both an older overwash algorithm (Larson et al. 2004) and a new overwash algorithm (Donnelly et al. 2005). These algorithms are implemented in the SBEACH profile change model (Larson and Kraus 1989). The following steps formed the methodology when undertaking this study:

- Collect beach profile, wave, and water level data for model testing
- Review and analyse all the data collected
- Review previous studies pertaining to modelling of coastal overwash processes.
- Study and understand modelling systems and their appropriate applications for the study.
- Apply the empirical model (Williams 1978, Tanaka et al. 2002)
- Apply the SBEACH overwash model with both the old and new overwash algorithms (Larson et al. 2004), and (Donnelly et al. 2005)
- Analyse results from the different types of models. Compare and contrast all results

- Discuss the applicability of the various models
- Outline overall conclusions and recommendations.

Chapter 2: LITERATURE REVIEW

2.1   COASTAL OVERWASH PROCESSES AND MORPHOLOGY

2.1.1   Introduction

Overwash processes and morphology have been studied by Fisher, Leatherman and Perry (1974); Schwartz (1975); Leatherman (1976); Williams (1978); Sallenger (2000), Morton et al. (2000), and Donnelly (2005) among others. Sallenger (2000) and Morton et al. (2000) have classified the overwash processes into two main mechanisms of overwash, namely runup overwash and inundation overwash. A comprehensive review of literature pertaining to overwash processes and modelling of overwash can be found in Donnelly et al. (2005).

Donnelly et al. (2005) described five different overwash regimes and resulting washover morphologies. Any of these regimes may occur within a storm, varying both spatially and temporally. The five different overwash regimes are described more in-depth in the following.

2.1.2   Runup Overwash

Runup overwash can be defined in terms of the relationship of water level, including surge height (S) and runup height (R), to the barrier elevation (d_c). Figure 2.1 is a schematic cross-section of a sand dune subject to high surge level and overtopping waves. The excess runup (ΔR) is the difference in elevation between the wave runup heights plus storm surge height and the barrier crest.

Runup overwash may be divided into two regimes. The first regime occurs when the sum of runup height and storm surge height is only slightly larger than d_c, and the excess runup is therefore small. Few waves overwash the barrier crest and those that do deposit entrained sediment on the crest (Fisher et el 1974, Leatherman 1976a), If this continues at the same water level, the accreted sediments may stop overwash in the future. This is called crest accumulation overwash.
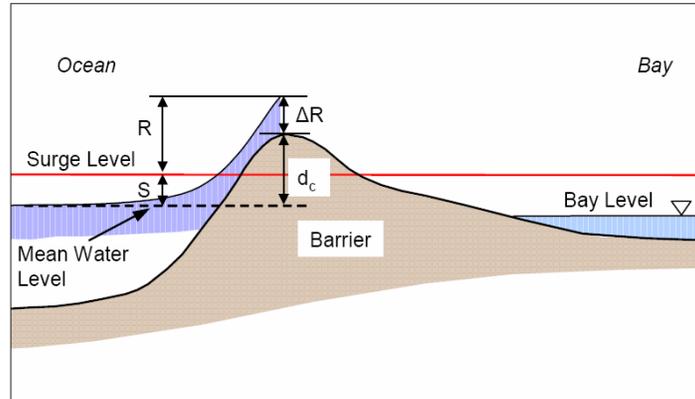
Fig. 2.1: Definition sketch showing the cross-section of a barrier beach subject to overwash by wave runup (after Donnelly et al 2004)

For the second regime, the storm surge level is still smaller than d_c; however, the storm surge or runup height is larger than for the first regime (crest accumulation) so that many waves have sufficient excess runup height to overtop the beach crest. In this case, the sediment is eroded from the beach and dune face and transported to be deposited on the backshore. The beach crest is normally getting lower, thus this case is called crest lowering.

2.1.3 Inundation Overwash

Inundation overwash occurs when the Still Water Level exceeds the beach crest height (Figure 2.2 and Figure 2.3). Inundation overwash may be divided into a further three overwash regimes.

For the third regime the beach crest is inundated, but the flow does not penetrate all the way to the back barrier bay, either because the barrier is too wide, the beach has rear dunes, or vegetation and infiltration stop flow. Thus, the inundation depth and velocity must be small, so that the overwash does not reach the bay. Washover is deposited as the bore slows.

The fourth regime involves constant flow over a prominent dune feature and this regime takes place if the storm surge height is greater than the dune crest height. Inundation overwash under such conditions may cause severe erosion on the backside of the dune In addition, waves on top of the barrier contribute to additional erosion and reduction of the crest. This type of overwash normally destroys the dune and moves sediment quite far inland.
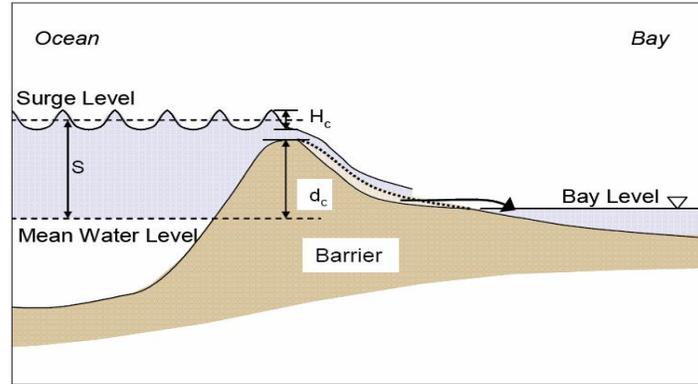
Fig. 2.2: Definition sketch showing cross section of a barrier with a prominent dune subject to overwash by overflow (after Donnelly et al 2004)

The fifth overwash regime involves complete inundation of a barrier and this occurs if the storm surge height is much greater than the beach crest height (Figure 2.3). The overwash flow and washover sediment reaches the back barrier bay, and coupling between the ocean and bay water levels occurs. Complete inundation overwash has a high tendency for causing breaching (Kraus and Wamsley 2003), and the sediment is often transported as sheet wash (Figure 2.3). Sheet wash is common on low confined barriers, but it may also occur after wave attack and overwash have reduced the existing dunes to a low elevation.
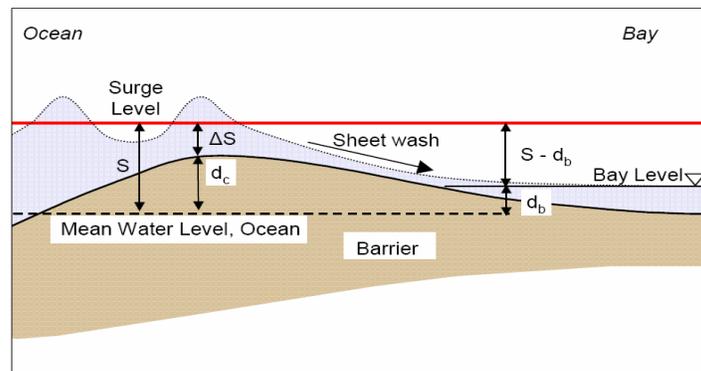
Fig. 2.3: Definition sketch showing cross-section of a barrier beach subject to overwash by overflow where barrier is fully inundated (after Donnelly et al 2004)

2.2   WASHOVER MORPHOLOGIES

Washover is defined as the sediment that is transported and deposited inland by overwash (Williams 1978). The topography of the foredune, the slope of the backside terrace, the width of the barrier island, the beach sediment, the presence of barrier vegetation and manmade structures, as well as the magnitude of storm surge and waves

all affect the morphology of the washover deposit. Three common forms of washover deposits are the washover fan, washover terrace, and sheetwash deposits. Figure 2.4 is a schematic plan view over a typical dune line or beach crest subject to overwash, illustrating the common overwash deposit types.

Washover fans occur on beaches with local gaps in otherwise high foredunes either due to runup or inundation overwash. Water and sediment is funnelled through a local gap in the dune line and spreads laterally on the back barrier. This gap is often called the throat of the overwash.

Washover terraces occur when the throats through which overwash occurs become more frequent, the borders between two fans become less defined, and the deposits form a single terrace. Washover terraces can also form where the beach crest is low and uniform. Washover deposits may reach to the back-barrier lagoon or bay depending on the backslope of the barrier, impedance due to vegetation and man-made structures, and the width of the barrier.

Sheetwash occurs on beaches which have uniformly low dunes, or on duneless barriers and the beach crest is uniformly overwashed. Where sheetwash occurs, lateral spreading may be neglected, and sediment can sometimes be carried and deposited a hundred meters or more towards the backbarrier bay or lagoon.
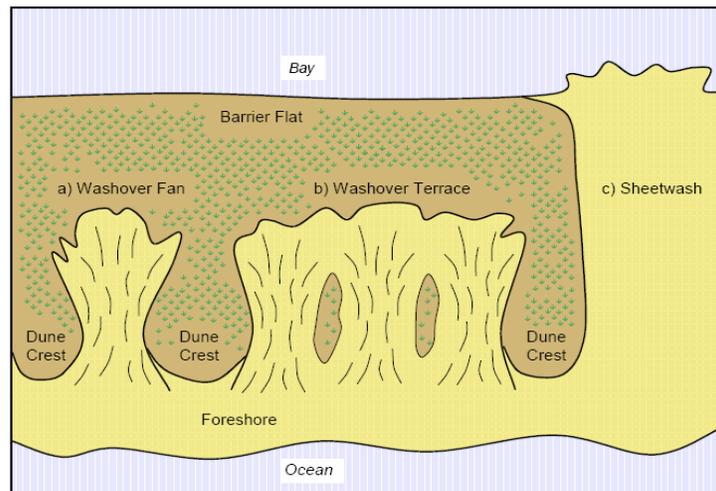
Fig. 2.4: Definition sketch of common morphological deposits occurring during overwash of dune such as a). a washover fan, b). washover terrace, c). sheetwash deposit (after Donnelly et al 2004)

The distance washover deposits are transported across a barrier island depends on the slope, permeability, vegetation of the backside terraces, and the velocity of the overwash

surges at the crest of the beach. In general, the steeper the slope of the back-barrier, the longer distances the washover sediment will be transported before it is deposited.

2.3 BEACH RESPONSE MORPHOLOGIES

Analysis of approximately 50 before and after field overwash profiles compiled by Donnelly, (personal communication, June 2005) has identified five different types of beach morphologic response in connection with overwash. The types encompass crest accumulation, dune lowering, dune rollback, dune destruction, and barrier rollback. These morphological changes are dependent on the hydrodynamic conditions, the beach and back barrier material and the topography and vegetation of the beach and back barrier .

Crest accumulation occurs during low magnitude runup overwash, where the sand is carried up from the beach and dune face to the top of dune crest and the sediment is deposited as the flow decelerates at the crest.

Dune lowering implies little or no change in position of the dune crest but a loss in crest elevation. Dune rollback and barrier rollback mean that the shape of the dune or barrier is more or less conserved but the whole system moves in the landward direction. When the water level is below the dune crest, waves attack the beach and dune face causing sediment to move in the offshore direction. Thus, the dune/barrier might be getting lower until there are sufficient wave energy and water level to move most of the sediment landwards.

Dune destruction implies that the dune is completely destroyed either by offshore transport of sediment due to wave attack, or by onshore transport due to overwash. If the sediment is transported offshore it is likely that the dune was destroyed prior to initiation of overwash. The sediments of a dune destroyed by overwash processes are transported onshore, usually because the surge level has exceeded the dune crest for a sufficient period.

It should be noted that wave characteristics and water levels (including storm surge and tides) during a storm are highly dynamic, and several overwash regimes may occur during one storm, making the estimation of a simple causal relationship between overwash regimes and washover morphologies difficult.

Chapter 3: DATA COMPILATION AND ANALYSIS

3.1 DATA COMPILATION

3.1.1 Introduction

The main data required for modelling purposes are beach profiles before and after a storm, together with time-series of wave height, period, and water level during the storm. The sediment characteristic, specifically, the median grain diameter, within the study area is also required, as well as a map over the study area. For this study, an attempt was made to assemble as many full data sets as possible. Beach profiles were compiled from previous studies (Eiser et al. 1991, Stone 2004, and Larson et al. 2004). If hydrodynamic data for the studied profiles were not measured during the relevant storm, hindcast time series of wave height and period were obtained from the Wave Information Studies (WIS) and time-series of recorded water levels were collected from the National Oceanic & Atmosphere Administration (NOAA).

Data on overwash events were compiled from three regions, all within the U.S.A. The first data sets are from Folly beach and Garden City beach, South Carolina, which were affected by Hurricane Hugo in 1989 (Eiser 1991). The second data sets are from Santa Rosa Island, Florida (Stone 2004). This area overwashed during Hurricane Opal, 1995, and Hurricane Georges, 1998. The third data sets are from Assateague Island, Maryland (Larson et al. 2004). This island was impacted by two northeaster storms in January 1998. Many other pre- and post-storm data sets were considered for this study, but were not used either due to the lack of hydrodynamic time-series data during the overwash event or due to the existence of a hard structure, such as a seawall or revetment on the beach front. Data sets from these three regions provided an excellent opportunity to monitor the effects of hurricanes and extreme storm events on beach profile shape.

3.1.3 South Carolina Profiles

Folly Beach Profile Data

According to Eiser et al. (1991), Folly beach is located approximately 20km south of the Charleston Harbour jetties and in the forward left quadrant of Hurricane Hugo's onshore movement track, as shown in Figure 3.1. In 1988 the South Carolina Coastal Council (SCCC), the state's coastal zone management agency, installed a network of 350 beach survey monuments at approximately 600 m spacing along the developed shoreline of South Carolina. These monuments were used for a program of twice-a-year beach profile surveys, designed to quantify beach erosion trends in the state.

Pre-Hugo profiles at Folly Beach were collected on the 5th November 1988, extending seaward from the monument to a depth of -1.5 m. The post-Hugo profiles were measured by SCCC on the 1st November 1989, ten days after Hurricane Hugo made landfall. In the days following Hugo a number of survey crews, including the U.S. Geological Survey, Florida Department of Natural Resources, U.S. Army Corps of Engineers, and the College of Charleston, arrived on the scene to collect profile data and to contribute to the post-Hugo beach survey efforts. Since the entire coastal region was declared a federal disaster area, logistical problems and lack of communication between survey crews hampered efficient efforts; however, the post-Hugo profiles were recorded at all surviving monuments of the SCCC.

The median grain size (d_{50}) at Folly beach is 0.17 mm.
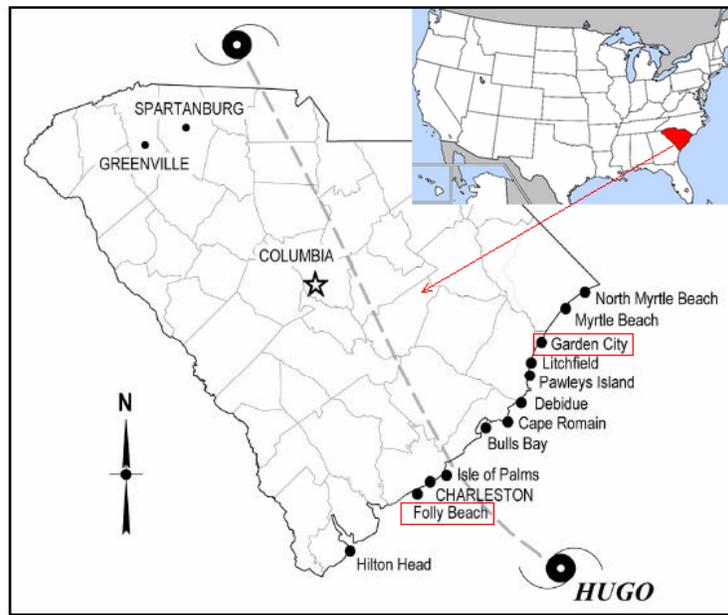
Fig. 3.1: Map of South Carolina (USA), showing Hurricane Hugo's storm track and the locations of Folly and Garden City Beaches (after Timothy W. Kana 2004)
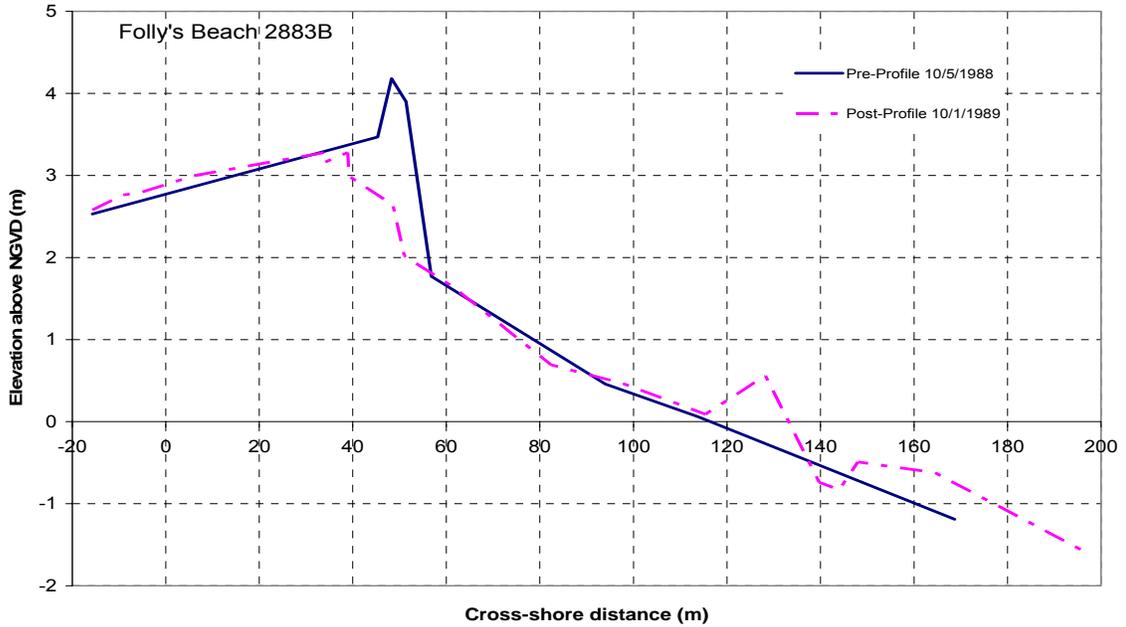
Fig. 3.2: The pre- and post-Hugo cross-shore beach profiles from Folly Beach, SC, Profile line 2883B

In order to compare the predicted overwash sediment volume from the empirical model with the measurements, three suitable profiles were chosen from Folly Beach namely, 2801, 2815, and 2823 as shown in Figures 3.3-3.5.
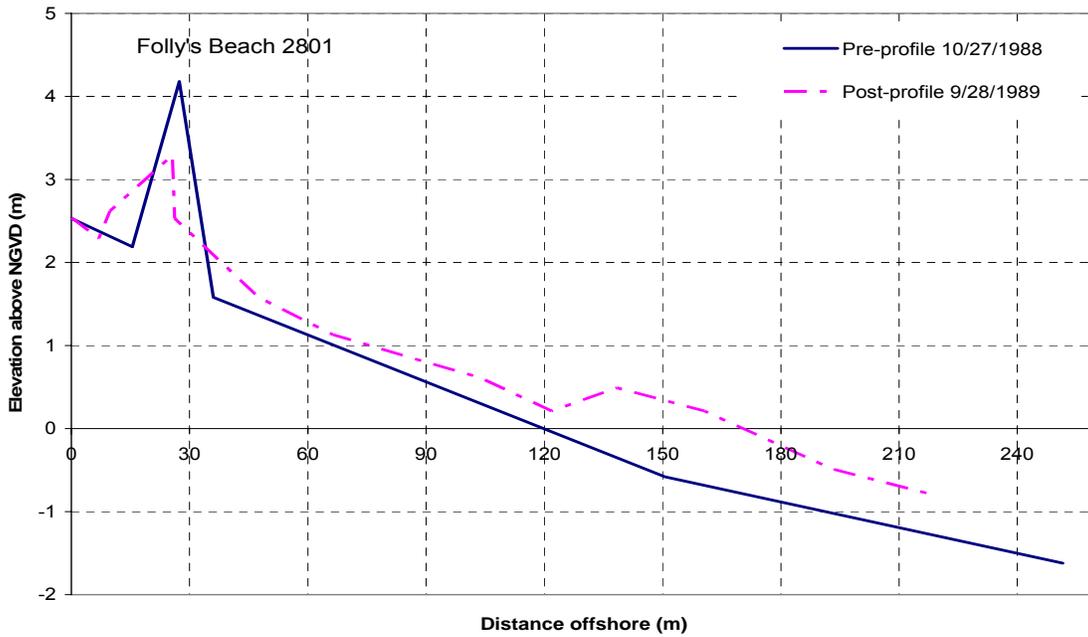
Fig. 3.3: Folly beach profiles number 2801 before and after Hurricane Hugo
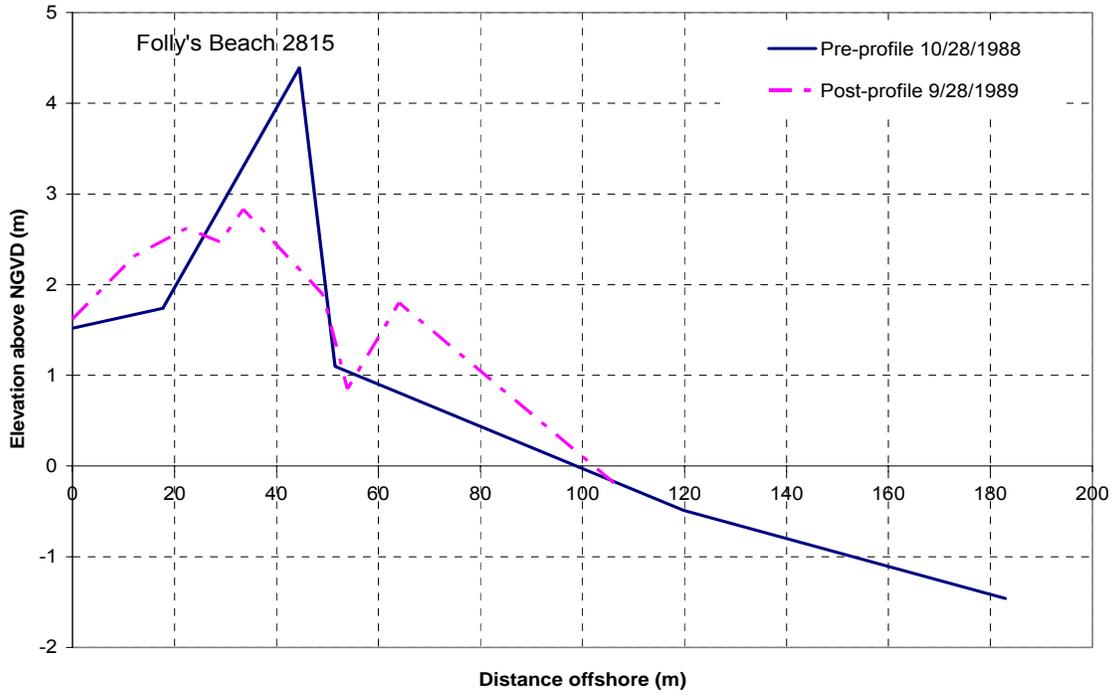
Fig. 3.4: Folly beach profiles number 2815 before and after Hurricane Hugo
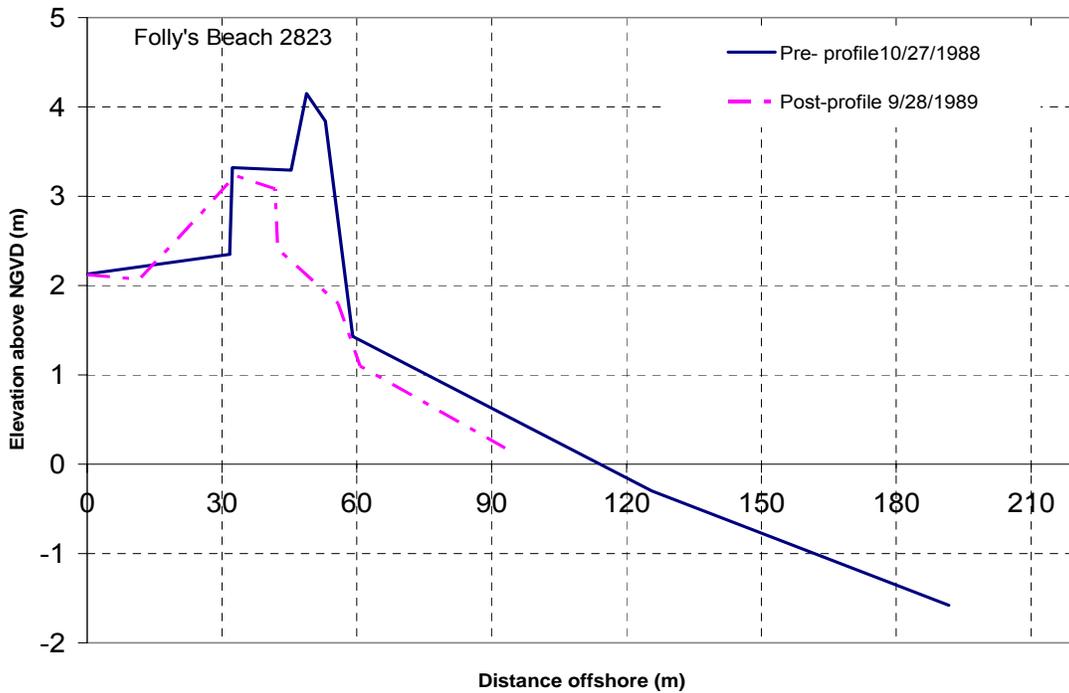
Fig. 3.5: Folly beach profiles number 2823 before and after Hurricane Hugo

Garden City Beach Profile Data

Garden City is located about 105 km northeast of Charleston Harbour which was in the forward right quadrant of Hurricane Hugo's onshore movement track, as can be seen in Figure 3.1. Because of this hurricane, 14 survey monuments were destroyed. Twelve monuments survived and were surveyed after the storm. The most recent pre-Hugo profile was collected on 4th September 1989 and the post-Hugo profiles were measured 6 days after the hurricane. The profiles extended seaward to -1.5 m depth. According to Eiser et al. (1989) the southern half of Garden City is a narrow, low-lying spit, while the north third is almost continuously armoured. This armoured section, where seawalls and revetments encroached on the beach, was narrow and had a substantial sand deficit prior to Hugo. Profiles in the armoured section were therefore not suitable for this study. Figure 3.6 shows the pre- and post-Hugo cross-shore beach profiles at monument 4930 at Garden City beach.

The median grain size (d_{50}) at Garden City beach is 0.44 mm.
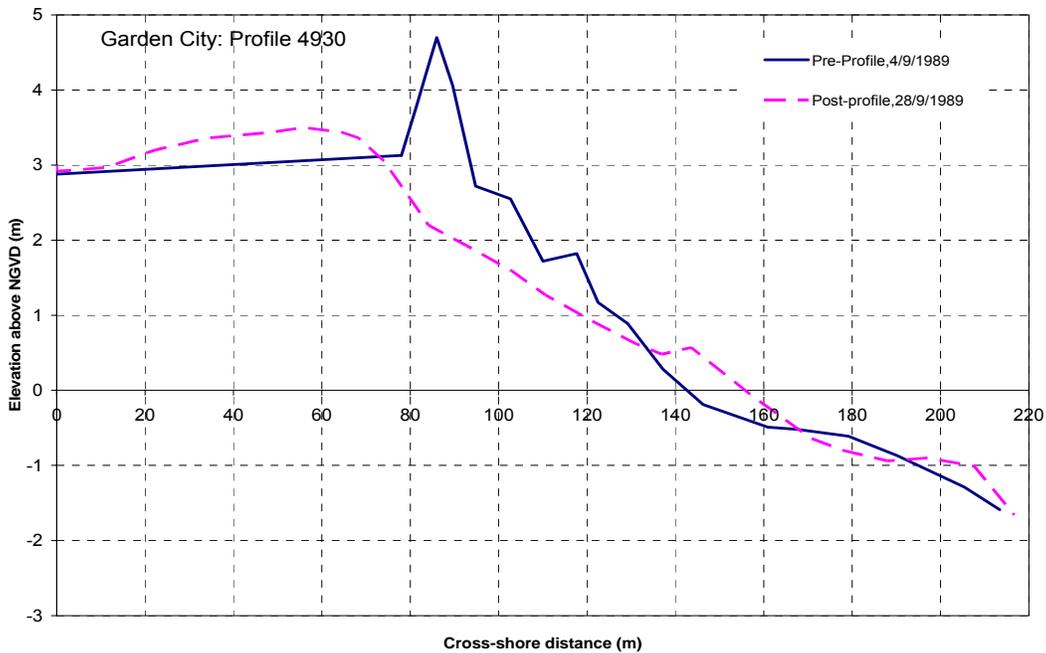
Fig. 3.6: The pre- and post-Hugo profile in Garden City beach, SC.

Hurricane Hugo Hydrodynamics

Hurricane Hugo crossed the South Carolina beaches on the night of September 21st 1989, causing a storm surge with a maximum still water elevation of 6.16 m above NGVD (National Geodetic Vertical Datum of 1929). The hurricane made landfall at Charleston. Time-series of significant wave heights and wave periods along the South Carolina Coast during Hugo were obtained from the WIS hindcast website. The WIS

hindcast wave data are generated by numerical simulation of past wind and wave conditions for U.S. coastal waters. Measured wave time-series data was also available from the National Data Buoy Center but the buoy stations were located in deep water and at large distances offshore, so the shallow WIS stations were used instead of buoys to avoid issues with energy dissipation over the continental shelf.

The water level observations, including the tide level and storm surge height, were obtained from the National Oceanic & Atmospheric Administration (NOAA). In order to get the data for study areas, the closest station was chosen, data extracted for the same time period bracketed by the pre and post-storm surveys and all data was converted to the NGVD datum. The water level stations closest to the selected Folly beach and Garden City profiles are the Charleston and Winyah Bay gauges, respectively. The difference between Mean Lower Low Water (MLLW) and NGVD at the Charleston water level station is 0.658 m and at the Winyah water level station is 0.372 m.
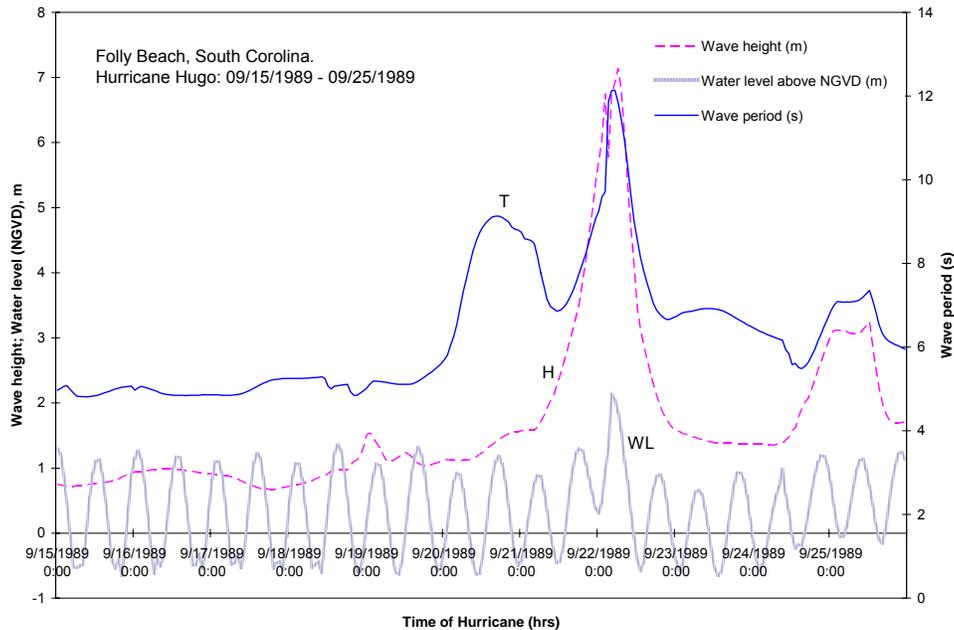
Fig. 3.7: Hurricane Hugo conditions at Folly beach S.C. from 09/15/1989 to 09/25/1989. WIS station number 348; Water level data at Charleston station

Cyclonic circulation in the northern hemisphere is counter-clockwise around the centre of the low pressure system or eye of the hurricane and clockwise rotation in the southern hemisphere. The continental United States is located in the northern hemisphere so that the onshore winds are to the right of the eye. Since it is predominantly the frictional effect of the wind on the sea surface or wind stress that is responsible for the storm surge, maximum increases in water level will be to the right of the eye (Garcia et al. 1990), as can be seen in Figure 3.8. Because Folly beach is located on the left of Hugo's track (Figure 3.1) the peak storm surge recorded at Charleston Harbour was probably

greater than that at Folly Beach. Garcia et al. (1990) published some results of a storm surge hindcast model of Hurricane Hugo on the S.C. coast, showing the variations in peak surge water level along the S.C. coast (Figure 3.8). Using these results, it was decided to reduce the Charleston gauge water levels during the peak of the storm so that they matched the hindcast water levels at Folly beach published in Garcia et al. (1990). The difference was approximately 2.5 feet (≈ 0.762 m) at 4:00 a.m. on 22nd September 1989 and this was subtracted from the peak storm surge at Charleston gauge, 2.83m, to approximately, 2.01m, at Folly Beach. The water depth at the station is 16 m.
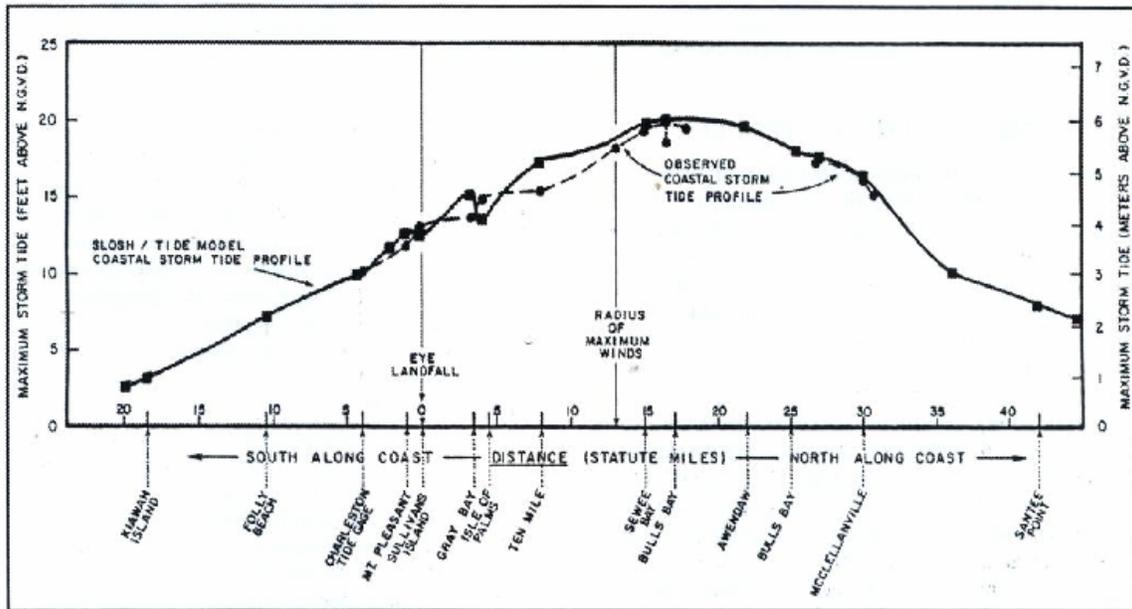
Fig. 3.8: The storm surge height results from SLOSH model along South Carolina coast by (after Garcia et el. 1990)
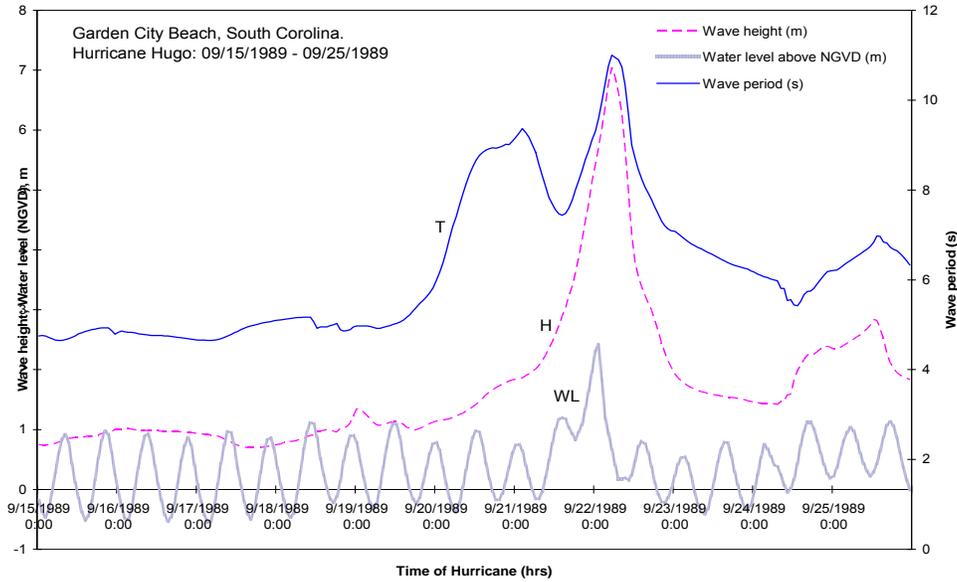
Fig.3.9: Hurricane Hugo conditions at Garden City beach S.C. from 09/15/1989 to 09/25/1989. WIS station number 348, Water level data at Winyah station

3.1.4 Santa Rosa Island Profiles

Santa Rosa Island Beach Profile Data

Santa Rosa Island, located along the Florida panhandle, extends approximately 95 km west of Destin to Pensacola Pass (Figure 3.10). This is a low profile barrier and supplied with 99% quartz sand from the Pleistocene "headland" at Grayton Beach to the east and an internal source of sediment along the west flank of the island (Stone et al. 1992). The width of the island varies from 162 to 1872 m with an average of approximately 514 m, and foredune elevations average 4.12 m above Mean Low Water (MLW). The median grain size (d_{50}) is 0.26 mm. This island is frequently overwashed. Pre- and post- storm profiles from Hurricanes Opal and Georges are presented in Stone et al. (2004).

Fig. 3.10: Location of Santa Rosa Island, Florida, USA (http://maps.google.com/)

According to Stone et al. (2004), the pre-Opal profile was surveyed on the 31st of September 1995. Beach profiles were surveyed following standard-level and transit-survey procedures using an electronic total station. Benchmarks established by the Florida Department of Environmental Protection (FDEP) were used and elevation was referenced to NGVD, which is approximately 0.08 m above Mean Low Water (MLW) at Santa Rosa Island. The post-Opal profile was measured three days after Hurricane Opal (Figure 3.11).

The most recent profile prior to Hurricane Georges had been obtained in June, 1998 thanks to the cooperation of the Coast Guard Air Station in Clearwater, Florida. The U.S. Geological Survey's severe storms project started in July, 1996 to assess coastal erosion from severe storms by acquiring pre- and post-storm aerial data sets at a constant scale. The USGS, NASA, and NOAA have established a cooperative agreement to expand the response effort to include aerial laser altimetry (Lidar) data to make quantitative measurements of beach topography. Pre- and post-Storm beach profiles for Hurricane Georges are illustrated in Figure 3.12. It should be noted that the beach crest height did not change considerably after Hurricane Georges, despite the large morphological changes either side of the crest. It is suggested that a seawall or some other hard structure must have been present at the beach crest, and hence modelling results from this profile were disregarded.
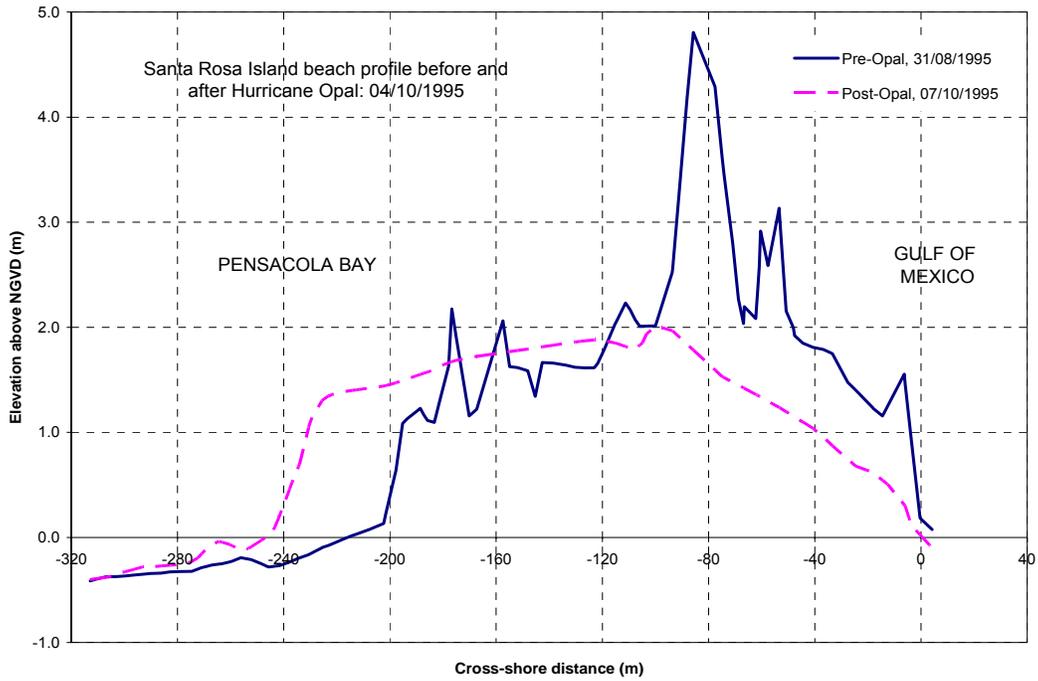
Fig. 3.11: Pre- and Post-Hurricane Opal profile at Santa Rosa Island, Florida
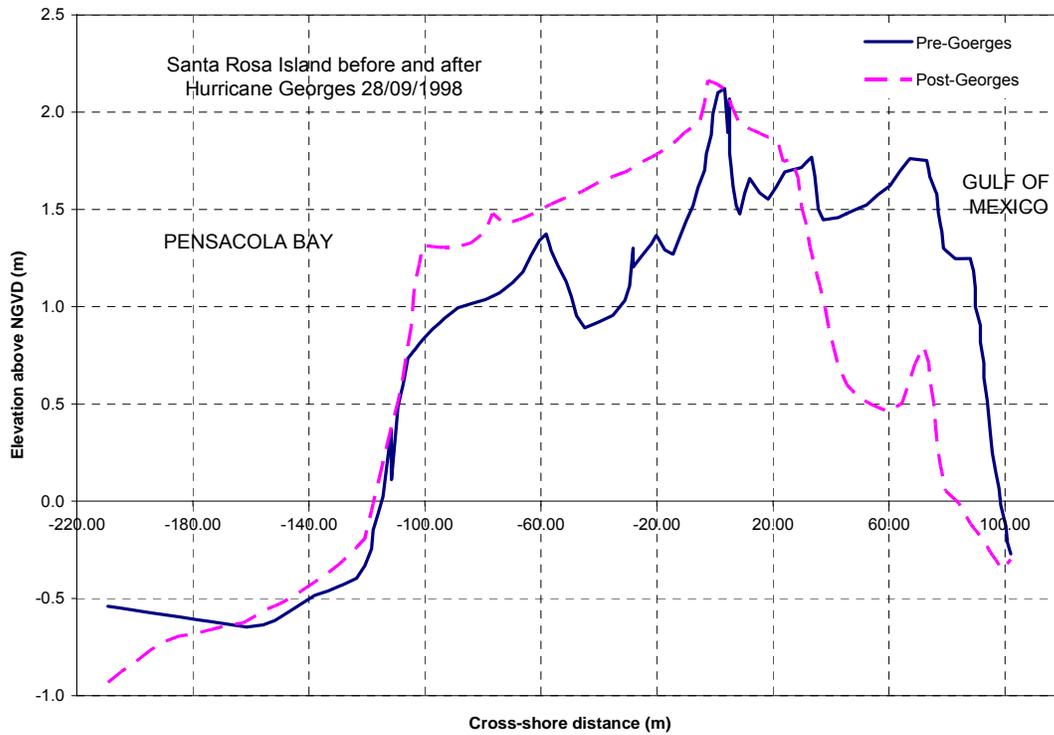
Fig. 3.12: Pre- and Post-Hurricane Georges profiles at Santa Rosa Island, Florida

Hurricane Opal and Georges

<table>
<tr><td>2005</td></tr>
</table>

Between 1995 and the beginning of 2004, Santa Rosa Island has been impacted by six hurricanes and/or tropical storms, and one of the most significant storms was Hurricanes Opal on the 4th of October 1995 (Stone et al., 2001). It was the strongest of 18 hurricanes to have impacted Santa Rosa Island over a 100-year period (Stone et al.1996 and Stone 1998). It impacted more than 2000 km of coastline along the northern Gulf of Mexico and it made landfall on Santa Rosa Island. The hydrodynamic conditions of Hurricane Opal can be obtained from the NOAA and USGS. Figure 3.13 shows the hydrodynamic conditions recorded during Hurricane Opal. Note that the maximum significant wave height was 8.18 m in a water depth of 27 m. Recently, Santa Rosa Island was inundated during Hurricanes Ivan (2004) and Dennis (). The destruction of the foredunes, as illustrated in figure 3.11 probably contributed to the effects of these later storms.
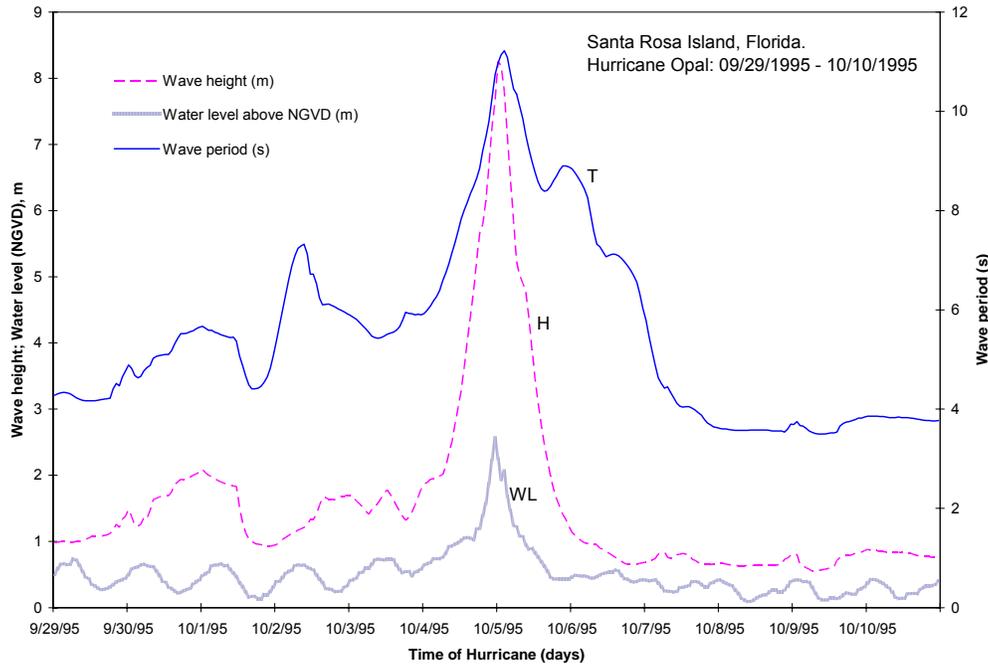
Fig. 3.13: Hydrodynamic conditions during Hurricane Opal from 09/29/1995 to 10/10/1995. WIS station number 173, Water level at Panama station (8279210).

Hurricane Georges made landfall along the Mississippi/Louisiana coast on the 28th of September 1998. The result from the USGS's surveys showed that there was significant overwash, particularly at narrow portions, and breaching of the barrier Island (Stone et al. 2004). Hurricane Georges was the second most destructive storm of the 1998 Atlantic hurricane season. It was the sixth storm to make landfall in the United States that year. Hurricane Georges was near category 3 strength. The maximum sustained wind was estimated at 47 m/s. The hydrodynamic conditions of Hurricane Georges were

also obtained from the NOAA and USGS (Figure 3.14). The maximum wave height exceeded 8m in 20 m depth.
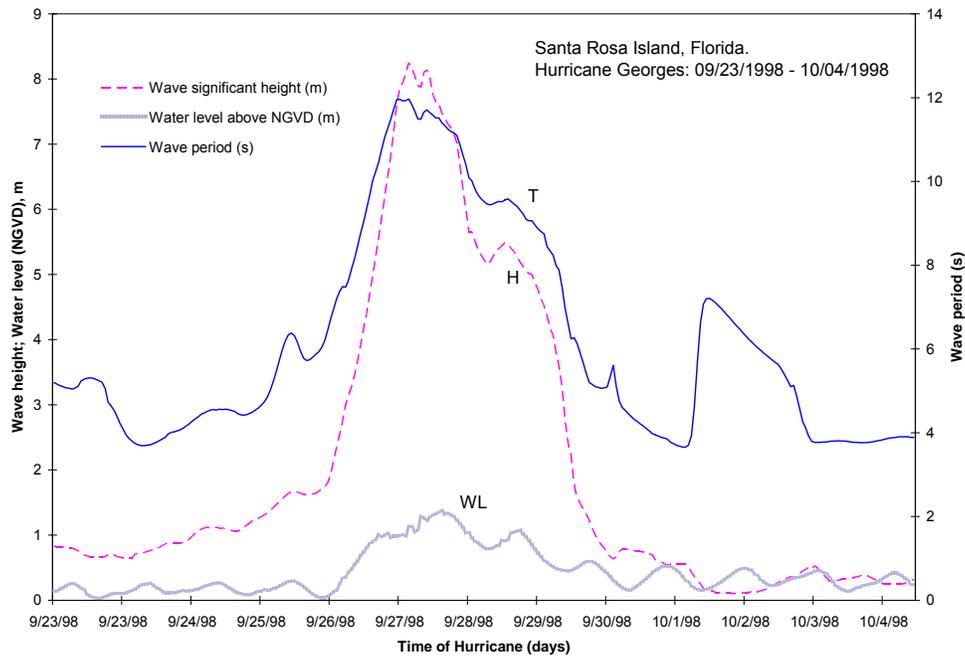
Fig. 3.14: Hydrodynamic conditions recorded during Hurricane Georges from 09/23/1998 to 10/04/1998, WIS station number 167, Water level at Pensacola station (8729840).

3.1.5   Assateague Island Profiles

Beach Profile and Hydrodynamic Data

Assateague Island is a low-lying barrier island, comprising the southern portion of Maryland's (MD) Atlantic coast and part of Virginia's Eastern Shore (Figure 3.15). Due to its low profile and the accessibility of its northern end, Assateague Island is a convenient location to measure overwash. During January and February 1998, two major winter storms, or "northeasters", severely impacted Assateague Island National Seashore. The USGS together with National Aeronautics and Space Administration (NASA) surveyed the Island using scanning airborne laser altimetry (LIDAR) prior to and after the two storms. The most recent pre-storm profiles were measured on the 1st of September 1997 and the post-storm beach profiles were taken in February 1998. The overwash profiles are labelled GPS1, GPS3, and GPS4 and are shown in Figures 3.16, 3.17, 3.18, respectively. The pre-profile condition at GPS1 was a broad low berm approximately 160 m wide with a crest at 2 m above NGVD backed by a 30 m wide dune with a crest at 3.5 m above NGVD. The profiles GPS3 and GPS4 are duneless barrier island sections with crests at about 2 m NGVD and an almost flat (1%) back barrier slope. The median grain size (d_{50}) is 0.3 mm. The hydrodynamic conditions of the northeaster storms are shown in Figure 3.19.
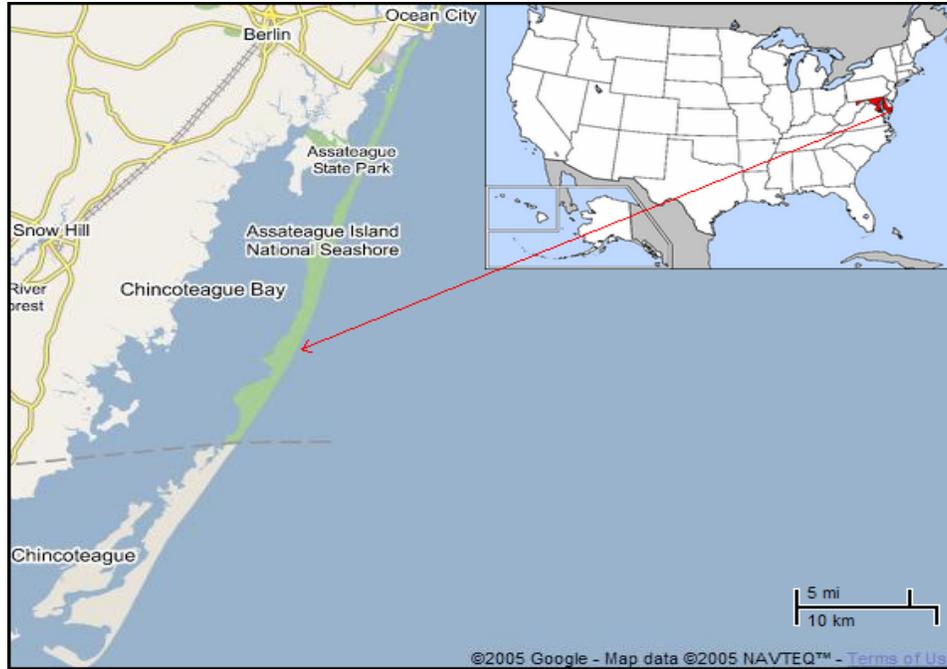
Fig. 3.15: Location of Assateague Island, Maryland on the map of USA.
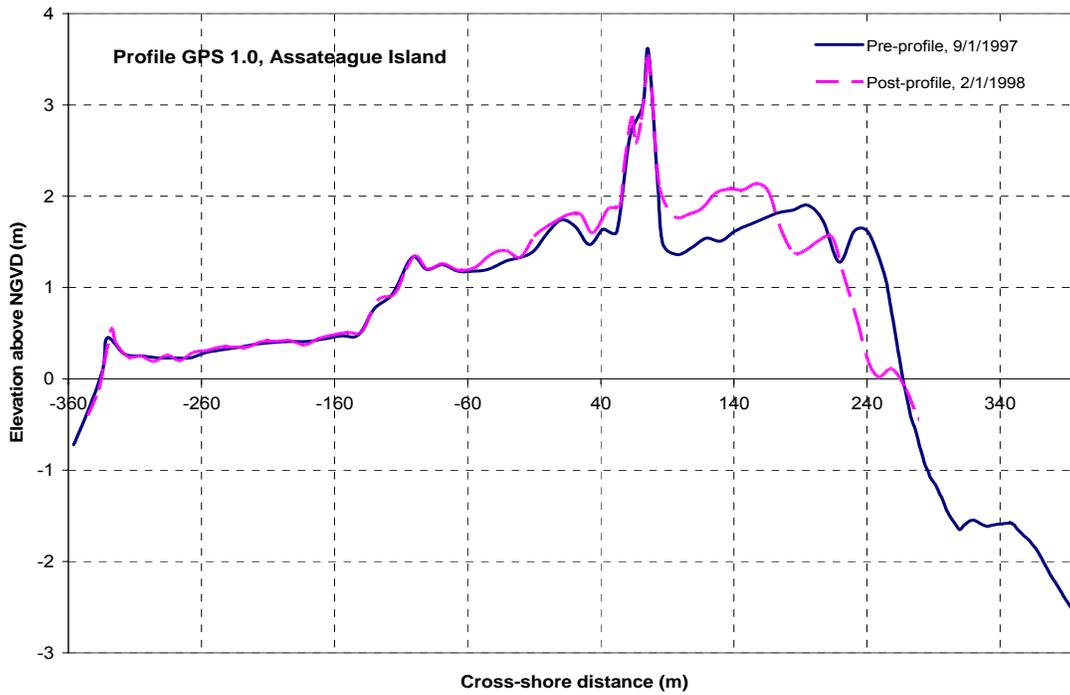
Fig. 3.16: Beach profile survey GPS1 at Assateague Island before and after Northeasters in January and February, 1998
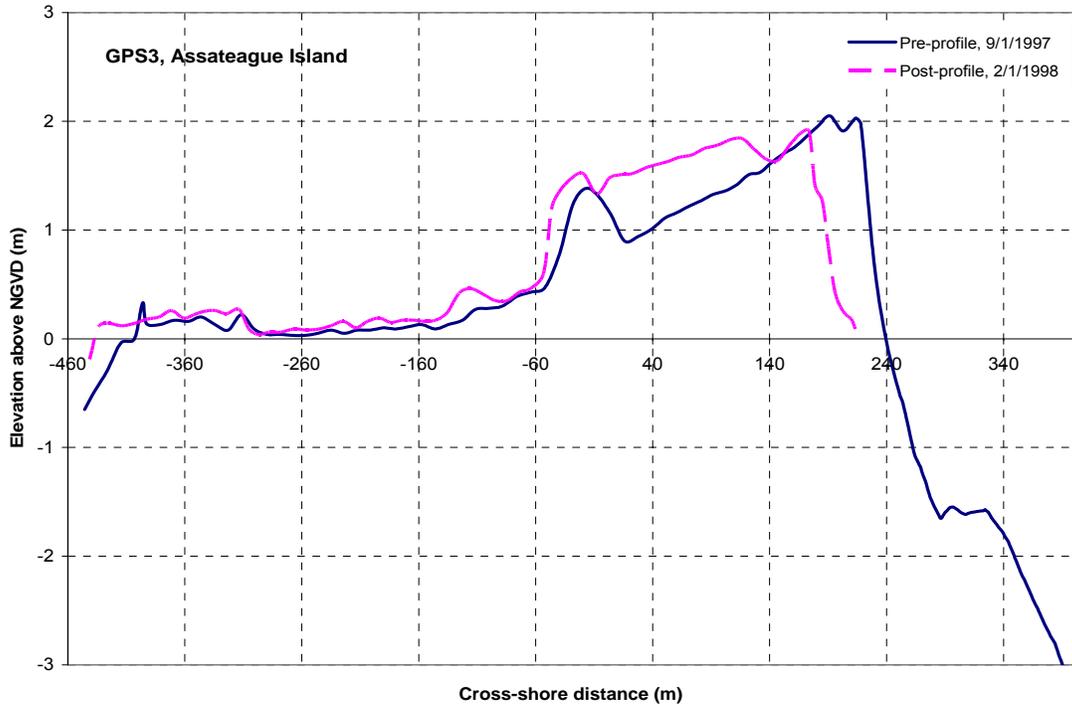
Fig. 3.17: Beach profile survey GPS3 at Assateague Island before and after Northeasters in January and February, 1998
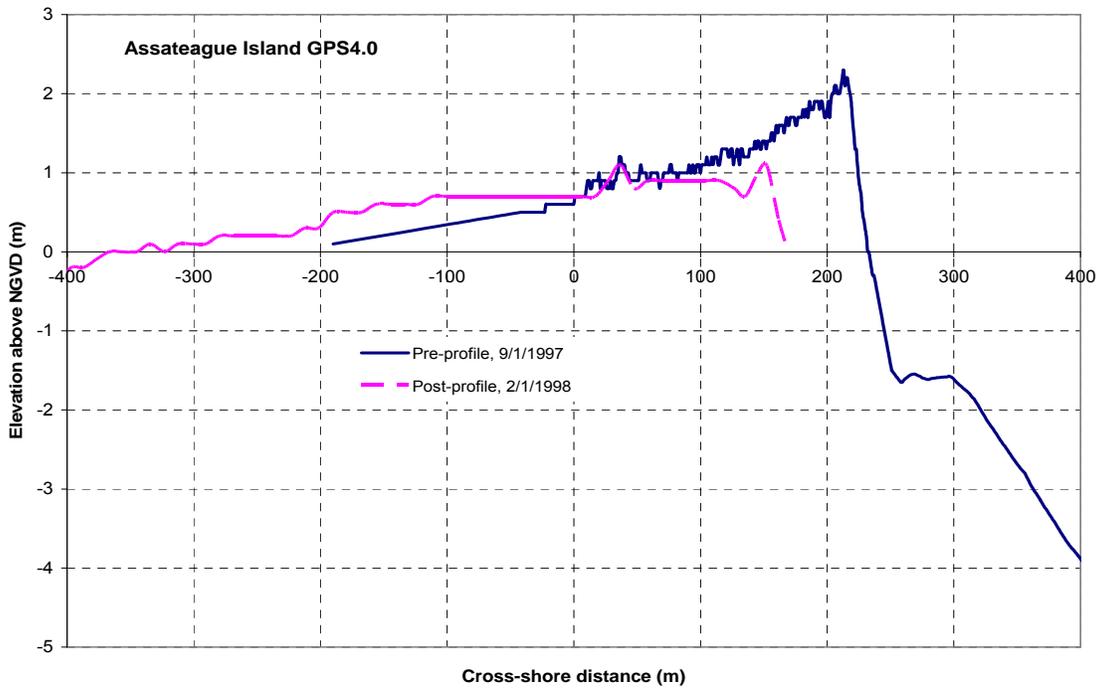
Fig. 3.18: Beach profile survey GPS4 at Assateague Island before and after Northeasters in January and February, 1998
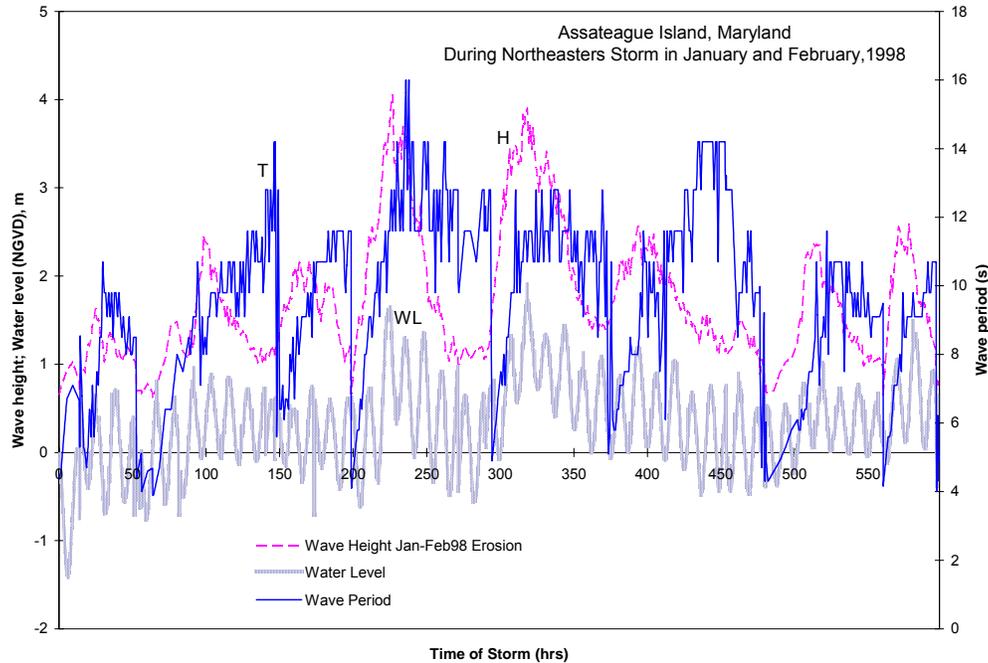
Fig. 3.19: Hydrodynamic conditions recorded during Northeaster storms from January to February 1998 at Assateague Island, Maryland.

3.2   DATA ANALYSIS

3.2.1   Dune and Barrier Morphology Analysis

In order to estimate the effects of overwash on beaches, a good first estimate is to categorise the type of morphological change observed,using the before and after storm cross-shore beach profiles. In this section, all the profiles discussed above will be employed to get an overview of the changes to dunes and barrier islands caused by storm overwash.

The vertical and horizontal distances between pre- and post-profile beach crest heights were used (summarized in Table 3.1) to see if there were any trends or causal links between

All the collected beach profiles can be divided into two different types: dune or duneless beach, depending mainly on the characteristics of the upper beach slope. Coastal dunes form where the backslope can support them and onshore winds promote the accumulation of sand blown inland from the beach. A barrier island is a low sandy formation running parallel to the shore that may or may not include dunes. In this study, the lowest limiting slope for a dune feature was set to 10%.

Dune destruction was identified at Folly Beach profiles 2823, 2883B, at Garden City Beach profile 4930, and at Santa Rosa Island for Hurricane Opal. The foredunes were destroyed by the storm and washover desposited on the back barrier. Note that at Santa

Rosa Island for Hurricane Opal, barrier rollback appears to have occurred following the destruction of the dune. Barrier rollback is also observed at Assateague Island GPS3 and GPS4, that is, the barrier retains almost the same shape but is translated in the landward direction.

Table 3.1: Beach response for each profile

Profile name	Dune/Barrier attenuation pre- and post profile (m)	Dune/Barrier landward movement pre- and post-profile (m)	Summary
Folly Beach 2801	0.89	1.8	Dune lowering
Folly Beach 2815	1.56	11	Dune lowering
Folly Beach 2823	0.92	15.9	Dune destruction
Folly Beach 2883B	0.9	9.4	Dune destruction
Garden City 4930	1.2	29.9	Dune destruction
Santa Rosa Island, Opal	2.82	11.7	Dune destruction and barrier rollback
Santa Rosa Island, Georges	-0.03	4.02	Crest accumulation
Assateague Island, GPS1	-0.24	38.5	Crest accumulation
Assateague Island, GPS3	0.14	17.7	Barrier rollback
Assateague Island, GPS4	1.2	60.9	Barrier rollback

Dune lowering was observed at Folly Beach profiles 2081 and 2815 (Figure 3.14). GPS1 illustrates an unusual case, where a dune with crest at 3.5 m above NGVD exists several hundred meters behind the beach crest, so that washover was deposited immediately seaward of this dune as flow decelerated.

To examine if there is any simple relationship between the storm characteristics, the pre-storm profile characteristics, and the beach profile change statistics summarised in Table 3.2 and 3.4, an analysis of beach profile characteristics such as the beach slope, dune slope, rear dune slope and rear barrier slope was made. Figure 3.20 shows a definition sketch for these typical beach profile parameters. The nearshore beach slope is defined as the slope between offshore water depth survey and the MWL. Sub-aerial beach slope is from MWL to the dune crest. The slope of the rear dune is defined as the slope from the dune crest to the point landward where a significant change in slope is observed. Rear barrier slope characterizes the area from the rear dune slope until the end of profile.
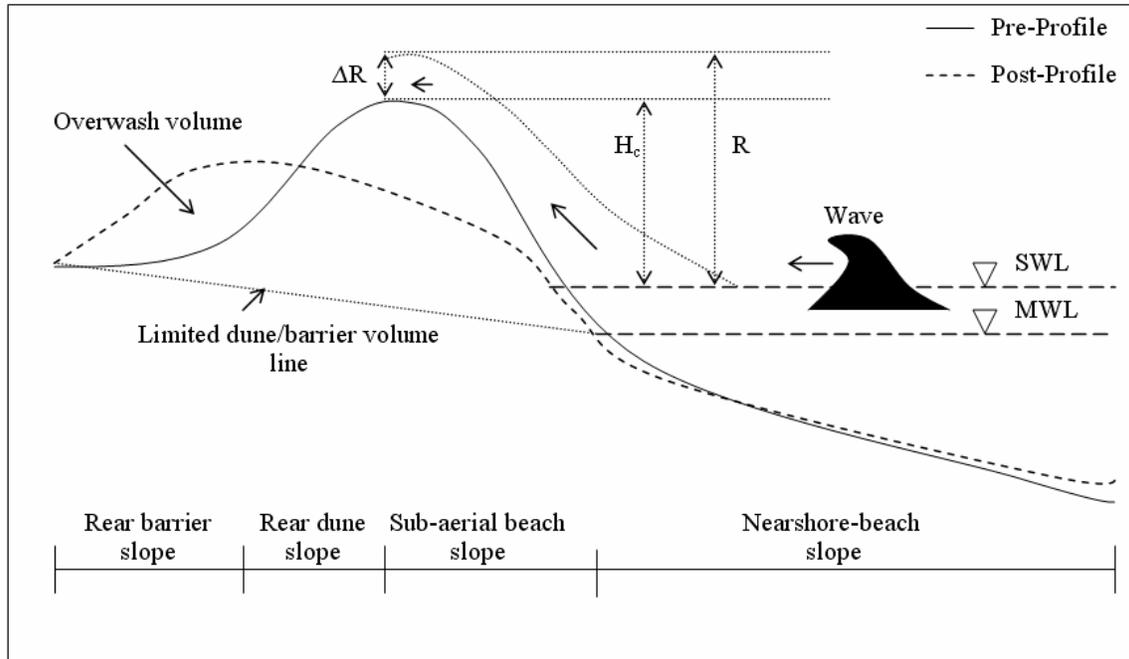
Fig 3.20: Definition sketch for typical beach profile parameters

A summary of the beach profile characteristics can be seen in Table 3.2. The beach slope (defined as the offshore slope) before and after the storm does not change much because the wave and water level does not directly affect this slope. The beach profile was assumed to have an equilibrium shape. During the storm, the dune/barrier island face was impacted by the wave runup and the variations in water level, thus beach crest was getting lower and the dune/barrier slope before and after the storm varied significantly. Pre-storm dune slopes varied from approximately 22.5 to 47% while post-storm dune slopes varied between 4.8 and 10 %. Post-storm back barrier slopes decreased compared with pre-storm profile slopes, but the difference was small since the back barrier slope is quite mild.

Table 3.2: The analysis of Dune and Barrier Island before and after storm

Profile name	BEFORE STORM				AFTER STORM			
	Near-shore beach slope	Sub-aerial beach slope	Rear dune slope	Rear barrier slope	Near-shore beach slope	Sub-aerial beach slope	Rear dune slope	Rear barrier slope
Folly Beach 2801	0.015	0.3	0.17	0.023	0.016	0.1	0.054	-
Folly Beach 2815	0.019	0.47	0.1	0.012	0.031	0.074	0.04	-
Folly Beach 2823	0.023	0.25	0.11	0.007	0.052	0.048	0.054	-
Folly Beach 2883B	0.026	0.28	0.24	0.015	0.023	0.058	0.013	-
Garden City 4930	0.036	0.225	0.2	0.003	0.029	0.063	-	-
Santa Rosa Island, Opal	0.011	0.053	0.04	0.005	0.011	0.021	-	0.004
Santa Rosa Island, Georges	0.016	0.07	0.012	0.006	0.016	0.025	-	0.007
Assateague Island, GPS1	0.028	0.09	0.084	0.003	0.021	0.09	0.078	0.003
Assateague Island, GPS3	0.016	0.021	-	0.01	0.016	0.046	-	0.003
Assateague Island, GPS4	0.015	0.085	-	0.01	0.01	0.072	-	0.002

In general, the dune crest decreases as the beach crest is overwashed, because the overwashing flow removes sand from the crest making the rear dune slope after the storm lower (Table 3.2). The rear dune slope was eroded by flow overtopping the crest and accelerating down the relatively steep dune slope. The rear barrier slope was generally accreted due to the deposition of sediments as the flow decelerated. This flow deceleration is caused by friction (vegetation), infiltration and lateral spreading. The accretion rate decreases landward generally causing the slope to become less steep. In some profiles, this slope was impossible to define because the surveyed beach profile was too short in the landward direction, which was the case at Folly beach and Garden City beach.

Another method for analysing the profile change is to estimate the available sand volume in the foredune before and after the storm and compare this to the volume of sediment overwashed. The available sand volumes were approximately defined by the area between the pre- and post-storm profiles with a limited dune volume line as shown in Figure 3.20. The overwash sediment volume was obtained from the area between the initial and final profiles and was defined as the volume per meter of beach (alongshore).

Negative values on the difference between pre- and post-storm dune volume in Table 3.3 indicate that the dune was reduced in volume and that the sediment was carried in the offshore direction. The positive values indicate that the sediment on the beach face was transported onshore to build up the dune or barrier island, which was the case for Assateague Island GPS3. The measured overwash sediment volume will be used to evaluate empirical formulas for overwash.

Table 3.3: Estimations of dune volume and overwash volume

Profile name	Dune/Barrier volume before storm (m^3/m)	Dune/Barrier volume after storm (m^3/m)	Change in Dune/Barrier volume (m^3/m)	Overwash Volume (m^3/m)
Folly Beach 2801	22.63	21.85	-0.78	4.75
Folly Beach 2815	55.36	36.99	-18.37	11.89
Folly Beach 2823	55.56	39.47	-16.09	5.58
Folly Beach 2883B	67.97	55.36	-12.61	4.19
Garden City 4930	134.9	104.4	-30.47	17.58
Santa Rosa Island, Opal	488.2	328.1	-160.1	55.7
Santa Rosa Island, Georges	333.8	322.6	-11.2	57.13
Assateague Island, GPS1	352.9	334	-18.9	50.1
Assateague Island, GPS3	441.8	461.3	19.5	101.4
Assateague Island, GPS4	475.2	400.6	-74.6	91.5

3.2.2 Hydrodynamics Analysis

The characteristics of storm hydrodynamics, such as the maximum wave height, wave period, and water level including storm surge and the wave runup height are of interest when estimating overwash response and for developing and applying the empirical and analytical formulas discussed in this report.

Larson et al. (2004a) developed a formula to predict the runup height in connection with dune erosion by wave impact, *R*, based on Hunt's runup formula (1959). The relationship for the runup height is given by,

$$R = 0.158\sqrt{H_0 L_0} \tag{3.1}$$

where H_0 is the wave height, $L_0 = 1.56 T_0^2$ is the wavelength, and T_0 is the wave period. The subscript, *0*, denotes the conditions in deep water, and the root-mean-square (rms) wave height should be used for random waves. For the Rayleigh distribution of wave heights, the relationship between significant wave height and rms wave height is calculated by $H_{rms} = H_0 / \sqrt{2}$ (SPM 1984).

The excess runup, ΔR, is defined as the difference in elevation between the potential runup and the crest of the dune or the barrier island referenced to still water level (SWL). Note that some authors such as Williams (1978) and Tanaka (2002) used mean sea level (MSL) as a reference when they calculate wave runup height. The surge duration is defined as the period of time over which excess runup occurred (i.e. where the runup height exceeded the dune crest). The runup height (R) was calculated for every time step. All the results are listed in Table 3.4.

Table 3.4: Extreme Hydrodynamic conditions

Profile	H0 (m)	Hrms (m)	T0 (s)	Water depth (m)	Max. water level (m)	Runup above SWL (m)	Hc above SWL (m)	Max. ΔR (m)	Surge duration (hrs)	d50 (mm)
Folly Beach 2801	6.69	4.78	10.79	16	2.13	4.65	2.02	2.51	8	0.17
Folly Beach 2815	6.69	4.78	10.79	16	2.13	4.65	2.23	2.30	8	0.17
Folly Beach 2823	6.69	4.78	10.79	16	2.13	4.65	2.02	2.51	8	0.17
Folly Beach 2883B	6.69	4.78	10.79	16	2.13	4.65	2.05	2.48	8	0.17
Garden City 4930	7.05	5.04	10.95	16	2.42	4.85	2.28	2.57	10	0.44
Santa Rosa Island, Opal	8.18	5.84	11.22	27	2.57	5.35	2.24	3.11	13	0.26
Santa Rosa Island, Georges	8.25	5.89	11.92	20	1.38	5.71	0.74	4.97	77	0.26
Assateague Island, GPS1	4.06	2.90	16	15	1.91	5.38	0.39	4.99	283	0.3
Assateague Island, GPS3	4.06	2.90	16	15	1.91	5.38	0.14	5.24	357	0.3
Assateague Island, GPS4	4.06	2.90	16	15	1.91	5.38	1.69	3.69	69	0.3

3.3 EQUILIBRIUM BEACH PROFILE

Since almost all of the surveys of beach profiles did not extend very far offshore or to sufficient depths for calculations using SBEACH, the concept of an equilibrium beach profile was applied to extend the profiles seawards. A fundamental assumption in the study of beach profile change is the existence of an equilibrium profile to which a beach will tend if exposed to constant wave conditions for a sufficiently long time. The idea is that the beach profile in its equilibrium state dissipates incident wave energy without significant net change in shape (Larson et al. 1999). If an equilibrium profile did not exist, the beach would continue either to erode or accrete indefinitely, if exposed to the same wave conditions and with no restrictions in the sand supply. A great difference between initial profile and equilibrium profile for a specific wave climate and grain size implies that a large amount of sand must be redistributed in the process of reaching equilibrium.

Dean (1977) derived a simple analytical expression for the equilibrium beach profile shape in the surf zone based on the concept of a constant dissipation of wave energy per unit water volume. This expression agrees well with the relationship established by Bruun (1954) on empirical grounds from field data. Dean's equilibrium beach profile shape may be written as following.

$$h = A * x^{2/3}$$
(3.1)

where

h is the still-water depth

x is the horizontal distance from shoreline (h=0)

A is the shape parameter

The shape parameter is mainly a function of grain size, in which a coarser grain size gives a larger value of A and a steeper beach. Kriebel et al (1991) used field data to obtain the following equation for the shape parameter of A.

$$A \approx 2.25 * \left(\frac{w^2}{g} \right)^{1/3}$$
(3.2)

where w is the sediment fall velocity that is a function of temperature and median grain size (d_{50}) and g is the gravitational acceleration. Equation 3.2 is applicable for a water temperature of approximately 20^0C and $1 \leq w \leq 10$ cm/s.

For an equilibrium profile developed during a storm, a bar normally forms in the vicinity of the break point (Larson and Kraus 1989). According to Larson and Kraus (1989) equation 3.1 is expected to apply only to the portion of the surf zone shoreward of the bar, where strong turbulence is present and energy dissipation is related to the breaking wave height and water volume. If wave reformation occurs, several areas along the profile may exist in which profile change is controlled by energy dissipation per unit volume and the profile in these areas is expected to be well approximated by equation 3.1.

Chapter 4: MODELLING OF COASTAL OVERWASH

4.1   INTRODUCTION

The modelling of coastal overwash processes and beach profile evolution play an important role for coastal authorities and engineers, in terms of environmental planning, development planning and post-storm response planning. The prediction of overwash occurrence is significant for coastal authorities and local managers in order to give evacuation warnings to people who live near the coast. Washover volume estimates are useful for planners and engineers to mobilise equipment for cleaning up sediment after a storm, or for studying sediment budgets of barrier islands. Profile change estimations are good for planners creating "buffer zones" in order to define safe zones for building houses. The ability to model overwash processes is therefore of great importance. This study discusses three different levels for modelling overwash processes: empirical, analytical, and numerical models.

The ability to accurately estimate washover volumes resulting from a single storm and cross-shore profile change due to an overwash event are only just emerging (Donnelly et al. 2005). Studies involving modelling of overwash processes include those of Leatherman (1976a), Williams (1978), Tanaka et al. (2002), Baldock et al. (2005), as well as Larson et al. (2004a, 2004b, 2005) and Donnelly (2005) This section will detail various attempts to model overwash and evaluate the current capabilities for modelling profile evolution due to overwash.

4.2   EMPIRICAL MODEL

4.2.1   Williams' formula

Williams (1978) derived two different empirical models to predict overwash volumes based on the results of laboratory experiments. The formulation of the predictive equation for overwash volume was based on three concepts:

a). The volume of sediment deposited on the back barrier is related to the volume of water which overtops the dune.

b). There is a minimum (critical) discharge or velocity for fluid flows below which sediment transport does not occur.

c). The sediment transport is proportional to the number of surges and hence inversely proportional to the wave period.

The first model was derived based on the assumption that the rate of overwashing is proportional to the excess runup raised to some power, α, and that the rate of sediment transport increases for increasing values of excess runup. The formula can be expressed as following,

$$q_s = \frac{K_1}{T}(\Delta R(t) - \Delta_*)^\alpha \qquad\qquad (4.1)$$

where: q_s is the rate of sediment overwashing

ΔR *(t)* is the excess runup (function of time as a result of changes in beach profile or wave and surge conditions)

Δ_* is the critical excess runup (The critical excess runup, Δ_*, is defined as the minimum height which the potential runup must exceed the dune crest for sand to be transported by overwash surges.)

K_1 is a proportionality constant

T is the wave period

The second model considers the sediment transport rate to increase rapidly for small values of excess runup and to asymptotically approach zero in an exponential form as excess runup increases.

$$q_s = \frac{K_2}{T}(\Delta R(t) - \Delta_*)e^{-K_3(\Delta R(t) - \Delta_*)} \qquad\qquad (4.2)$$

where K_2, K_3, are proportionality constants.

The constants, K_1, K_2, K_3, α, and Δ_* were evaluated using a nonlinear least squares analysis to fit the theoretical equations to the experimental data and it was found that the best parameters for the first model are K_1 = 0.09, α = 0.04, and Δ_* = 0.40 and for the second model are K_2 = 0.24, K_3 = 0.66 and Δ_* = 0.32. It should be noted that the equations were derived and fitted to experimental data of regular wave overwash in a flume. There is no dependency on storm or overwash duration.

The excess runup height formula $\Delta R(t)$ can be calculated as equation follows,

$\Delta R(t) = R(t) - D(t)$ $\qquad\qquad$ (4.3)

where *R(t)* is the runup height, and *D(t)* is the height of dune above MWL, which both are functions of time. In reality such measurements are very difficult to conduct. In order to get an estimation of sediment overwashing rate using the Williams's formula we assumed that the excess runup can be calculated from the wave runup height of Larson et al. (2004) $R = 0.158\sqrt{H_0 L_0}$ and the dune height, which is taken as the distance between the initial dune crest and the mean sea level (MSL) at the time of extreme

conditions. The total washover volume (Q) was established by multiplying the sediment overwashing rate (q_s) with the surge duration (t_D).

Model 1: $\qquad Q = q_{s1} * t_D = \dfrac{t_D}{T} K_1 (\Delta R(t) - \Delta_*)^{\alpha}$ (4.4)

Model 2: $\qquad Q = q_{s2} * t_D = \dfrac{t_D}{T} K_2 (\Delta R(t) - \Delta_*) e^{-K_3 (\Delta R(t) - \Delta_*)}$ (4.5)

Both of these models compared well with his observed laboratory measurements. The Williams model 2 was derived based on the assumption that the overwash transport rate increases rapidly for small values of excess runup and asymptotically approaches zero in an exponential form as excess runup increases; this assumption is not really true in reality since the more excess runup level the more overwash sediment volume will be expected. Thus, the Williams model 2 is not suitable for cases of high excess runup, which often occurs during the hurricanes or extreme storms.

4.2.2   Tanaka's formula

Tominaga and Sakuma (1971) carried out experiments of wave overtopping on a coastal dike with the front slope of 1:3, and proposed an empirical formula similar to the Williams formula (model 1),

$$Q_0 = \alpha (R_u - H_c)^n$$ (4.6)

Tanaka et al. (2002) derived a simple formula to calculate the overtopping rate based on the formula of Tominaga and Sakuma (1971) by considering the time-variation in the sea water level, η_{tide}.

$$Q_0 = \alpha (R_u + \eta_{tide} - H_c)^n$$ (4.7)

where Q_0 is the overtopping volume during one wave cycle (cm³/cm).

H_c is the average height of the dune crest (cm); (not a function of time)

η_{Tide} is the variation in sea water level due to tide and storm surge

n and α are respectively assumed to be 2.0 cm/s and 0.5 cm/s (Tominaga and Sakuma 1971)

R_u is the runup height, calculated using a formula by the Ogawa and Shuto (1984),

$$\frac{R_u}{H_0} = 0.767 s^{-0.12} (\frac{H_0}{L_0})^{-0.2} \left[3.04 s^{1.07} - 0.129 A^2 s^{-0.09} m^2 (\frac{H_0}{L_0})^{-0.75} \right] + 0.212 A \pi m^2 (\frac{H_0}{L_0})^{-1} \quad (4.8)$$

where H_0 is the deep water wave height, L_0 is the deep water wavelength (L_0 = $1.56 T_0^2$), s is the surf zone slope, m is the foredune zone slope, and the parameter A is given by,

$$A = \frac{1 + \dfrac{f}{a^2 m}}{(1 + 2a)(1 + a)} \tag{4.9}$$

where $f = 0.02$ is the friction coefficient and $a = 0.26$ for $m > 0.1$ and $0.13m^{-0.3}$ for $m < 0.1$. Tanaka (2002) adjusted the formula of Tominaga and Sakuma (1971) for irregular waves using the method described by Goda (1985). Additionally, he assumed that the sediment overwash rate is linearly related to the overtopping rate as was proposed by Hancock and Kobayashi (1994). The resulting order-of-magnitude estimate was comparable to volumes of sediment deposited in Gamo Lagoon, Japan, during an overwash event.

The formula to estimate the deposited sediment volume is expressed as,

$$Q_s = 0.039 * Q = 0.039 * Q_0 * \frac{t_D}{T} \tag{4.10}$$

where: Q_s is the deposited sediment volume (cm^3/cm)

Q is the total overwash discharge (cm^3/cm)

T is the wave period (s)

t_D is the excess runup duration time (s)

$N = \dfrac{t_D}{T}$ is the number of wave

4.3   ANALYTICAL MODEL

Larson et al. (2005) developed an analytical model to estimate erosion and profile evolution for dunes exposed to wave impact and overwash. The case of erosion and overwash of a schematized barrier islands was also investigated. At present, the analytical model will produce reliable quantitative estimates of dune recession and erosion during storms, provided that the forcing conditions are known and that the geometry of the dune configuration is similar to the one assumed by Larson et al. (2005), that is, a plane-sloping foreshore backed by a vertical dune. In practical applications it is recommended to use a range of values on the transport rate coefficient in the formula to obtain an estimate of the variability in dune response.

Concerning the impact of overwash on dunes and barrier islands, only limited validation exists so far for the analytical models developed. The relationship employed to estimate the overwash transport has been indirectly validated through application in numerical modeling of dune and barrier island evolution (Larson et al. 2004b, Donnelly et al. 2005). However, it remains to assess how realistic the schematization of the profile geometries

are, although some studies on overwash of barrier islands support the simplified geometry used in the analytical solution.

It was considered beyond the scope of the present study to validate the analytical model and to establish reliable coefficient values to be applied for predictions. However, for the sake of completeness the models were briefly reviewed in this report.

4.4   SBEACH NUMERICAL MODEL

4.4.1   Introduction

The Storm-induced BEAch profile CHange (SBEACH) model was developed by Larson and Kraus (1989). SBEACH is an empirically based numerical model for simulating two-dimensional cross-shore beach change. A basic assumption of the SBEACH model is that profile change is produced solely by cross-shore processes, resulting in a redistribution of sediment across the profile with no net gain or loss material. Long-shore processes are considered to be uniform and neglected in calculating profile change. This assumption is expected to be valid for short-term storm-induced profile response on open coasts away from tidal inlets and coastal structures. The model assumes that cross-shore sediment transport is mainly produced by breaking waves and water level changing in calculating beach profile change. An overwash algorithm was initially developed and implemented in SBEACH by Kraus and Wise (1993). This algorithm was updated and improved by Larson et al. (2004) to simulate the sediment transport and subaerial profile response by runup overwash, here called the original SBEACH version. Recently, Donnelly et al. (2005) added a new algorithm to simulate overwash by inundation and improve the algorithm for flow and sediment transport on the back barrier, here denoted as the new SBEACH version. In this study, the SBEACH numerical overwash model in both the original and new versions will be applied and tested with different field data sets from the United States.

4.4.2   Theoretical background

The SBEACH profile change model utilizes a meso-scale approach to calculate sediment transport across the profile, where the direction and rate of sediment transport are expressed in terms of wave, water level, profile, and sediment characteristics. Initial version of SBEACH investigated the cross-shore transport rate under monochromatic waves. Larson (1994, 1996) modified the monochromatic transport relationship for random waves to develop better simulation results under broken and unbroken wave conditions. The profile was divided into different zones of cross-shore transport based on characteristics of hydrodynamics across the profile (Miller 1976, Svendsen, Madsen and Hansen 1978, Skjelbreia 1987). Sediment transport relationships were developed for each of the zones. Those include I – Pre-breaking zone, II – Breaker transition zone, III – Broken wave zone, IV – Swash zone, V – Dune crest zone, and VI – Landward zone as

shown in Figure 4.1. The pre-breaking zone extends from the seaward limit of significant profile change to the break point. In the pre-breaking zone (I) the transport rate is influenced by transport in the zone of wave breaking through the sediment flux at its shoreward boundary. Breaker transition zone (II) corresponds to the breaker transition region and is located between the break point and the plunge point. Broken wave zone (III) is from location of the plunge point to the point of wave reformation, where the waves are fully broken and gradually decay. In this region the energy dissipation of the waves due to breaking become fully developed (Larson and Kraus 1989). If there are several break points occur with intermediate wave reformation, several zones of type II and III will be present along the profile.

Swash zone (VI) is from the shoreward boundary of the surf zone to the shoreward limit of runup or beach crest. In this region, cross-shore sand transport is expected to depend mainly on properties of the runup bore, local slope, and sediment characteristics. The sediment transport relationship of these first three zones in detail can be found in Larson et al. (1989, 1996, 2005). In summary, the sediment transport rate in the different zones can be written,

Zone I: $\qquad q = q_b e^{-\lambda_1 (x-x_b)}$ $\qquad\qquad$ if $x_b < x$ $\qquad$ (4.11)

Zone II: $\qquad q = q_p e^{-\lambda_2 (x-x_p)}$ $\qquad\qquad$ if $x_p < x \leq x_b$ $\qquad$ (4.12)

Zone III: $\quad q = \begin{cases} K\left[D - D_{eq} + \dfrac{\varepsilon}{K}\dfrac{dh}{dx} \right], D > \left[D_{eq} - \dfrac{\varepsilon}{K}\dfrac{dh}{dx} \right] \\[3mm] 0, D \leq \left[D_{eq} - \dfrac{\varepsilon}{K}\dfrac{dh}{dx} \right] \end{cases}$ $\qquad$ if $x_s \leq x \leq x_p$ $\qquad$ (4.13)

Zone IV: $\qquad q_{sw} = K_c 2\sqrt{2g} R^{3/2} (1 - \dfrac{z}{R})^2 (\tan \beta_l - \tan \beta_e)$ $\qquad$ (4.14)

Zone V: $\qquad q_D = K_B 2\sqrt{\dfrac{2g}{R}} (R - z_D)^2$ $\qquad\qquad\qquad\qquad$ (4.15)

Zone VI: $\qquad q_f = \dfrac{q_D}{1 + \mu s / B_D}$ $\qquad\qquad$ if $x < x_D$ $\qquad$ (4.16)

where q is the transport rate, q_b is the transport rate at the break point, q_p is the transport rate at the plunge point, q_{sw}, q_D, q_f is the transport rate in swash zone, dune or beach crest zone, and landward zone, respectively, D the energy dissipation per unit water volume and $D_{eq} = \dfrac{5}{24} \rho (gA_s)^{1.5} \gamma_b^2$ its equilibrium value, $\lambda_1 = m_* (\dfrac{D_{50}}{H_b})^{0.47}, \lambda_2 = 0.2\lambda_1$ are empirical decay coefficients, K, K_c, K_B and ε are empirical transport coefficients, x is a

cross-shore coordinate, x_b is the location of break point, x_p is the location of plunge point, x_s is the location of the boundary between the swash zone and surf zone, and x_D is the location of dune crest.
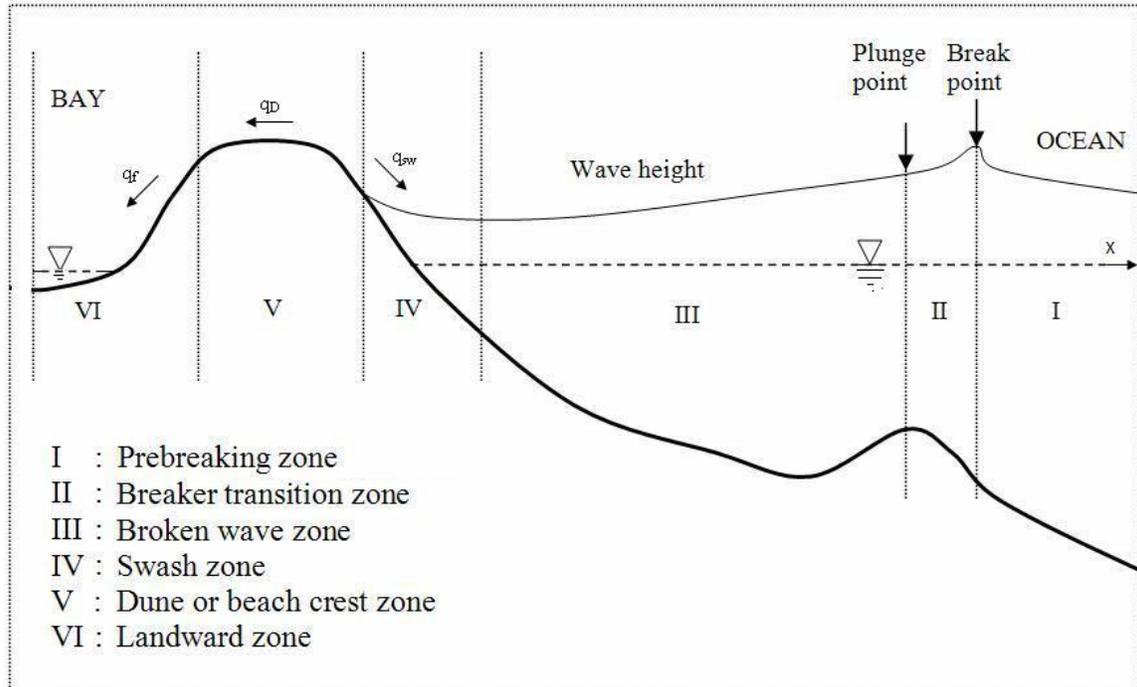
Fig. 4.1: Definition sketch for different zones of cross-shore sand transport (modified from Larson and Kraus 1989)

4.4.3   The original overwash algorithm

Kraus and Wise (1993) introduced a simple algorithm into SBEACH to calculate the inland transport of sediment due to overwash, but some of the assumptions that they made were mainly intuitively based. Larson et al. (2004) further developed the overwash algorithm in SBEACH expressing runup overwash considering the physical processes. The profile was divided into three regions: the swash zone, the beach/dune crest zone, and the landward of the crest zone. The swash zone is from the shoreward boundary of the surf zone to the beach crest, whereas the crest zone is from the beach crest point to the significant slope change on the shoreward side of the crest. The landward of crest zone is the third region, and is located between the beach crest and the landward limit of overwash penetration.

In the swash zone, Larson et al. (2001, 2004) developed a formula calculating the sediment transport q_{sw}, over many cycles,

$$q_{sw} = K_c \frac{u_b^3}{g} (\tan \beta_l - \tan \beta_e) \frac{t_0}{T} \tag{4.17}$$

where K_c is an empirical transport coefficient, u_b is the front speed of the uprushing wave, g is the acceleration due to gravity, β_l the local foreshores slope, β_e the equilibrium foreshore slope, t_0 the time duration which a specific location is submerged, and T the swash period.

In Equation 4.17, the transport is expressed with respect to the deviation from an equilibrium foreshore slope, which is a model input, and the time of submergence is taken into account.

The speed of the uprushing wave front is estimated based on simple ballistics theory as follows,

$$u_b^2 = u_{bs}^2 - 2gz \tag{4.18}$$

where u_{bs} is the speed of wave at boundary between the surf zone and swash zone, where the wave uprush starts, z is the vertical distance from the SWL to the location where u_b is calculated. The time of submergence is given by,

$$\frac{t_0}{T} = \sqrt{1 - \frac{z}{R}} \tag{4.19}$$

where $R = \dfrac{u_{bs}^2}{2g}$ is the runup height which is assumed to depend on u_{bs}.

Substitution of equation 4.18 and 4.19 into equation 4.17 produces the following sediment transport for the swash zone,

$$q_{sw} = K_c 2\sqrt{2g} R^{3/2} (1 - \frac{z}{R})^2 (\tan\beta_l - \tan\beta_e) \tag{4.20}$$

In the crest zone, Larson et al. (2004) assumed that the sediment transport rate in the overwash is proportional to the average rate of water flow crossing the top of the beach during a swash cycle. This yields the following expression for the average transport over the beach crest during an overwashing wave,

$$q_D = K_B 2\sqrt{\frac{2g}{R}} (R - z_D)^2 \tag{4.21}$$

where q_D is the transport rate over the beach crest, K_B is a non-dimensional empirical coefficient (about = 0.005), z_D is the height of the beach crest above the SWL including the effect of possible surge.

In the zone landward of the crest, the landward flow will depend on the incoming hydrodynamics, back barrier topography, and sediment characteristics. In this region, the reduction in velocity of the landward flow is mainly due to lateral spreading and infiltration (Larson et al. 2004). If it is assumed that the back barrier is saturated, thus reduction in the velocity is mainly due to lateral spreading, and assuming that the transport rate on the back barrier (q_f) is proportional to the velocity cubed, the transport rate in the flow is,

$$q_f = \frac{q_D}{1 + \mu s / B_D} \qquad\qquad x < x_D \qquad\qquad (4.22)$$

where the expression is normalised by q_D, and μ is an empirical coefficient, s is a coordinate originating at $x = x_D$ pointing landward, and B_D is the width of flow at the beach crest in the case of confined flow.

4.4.4   A new algorithm to simulate inundation overwash

Donnelly et al. (2005) developed a new algorithm to calculate inundation overwash by assuming that the sediment transport rate is linearly proportional to the flow rate over the crest and that water flow is described by the weir equation as following,

$$q_{DI} = 2K_B \sqrt{2g} h^{3/2} \qquad\qquad (4.23)$$

where q_{DI} is the overwashing transport over the beach crest K_B is an empirical coefficient, and h is the excess water level over the crest.

In reality, the discharge over the dune or beach crest begins as runup overwash and then the water level rises up until inundation overwash starts to occur. In order to get a smooth transition between two phases the following simple matching is employed,

$$q_D = q_{DR}\{R - z_D\} \qquad\qquad \text{if still-water level} < z_D \qquad\qquad (4.24)$$

$$q_D = q_{DR}\{R\} + q_{DI} \qquad\qquad \text{if still-water level} > z_D \qquad\qquad (4.25)$$

where q_D is the sediment transport rate over the crest, q_{DR} is the runup overwash transport rate, q_{DI} is the inundation overwash transport rate, R is the wave runup height, and z_D is the dune or beach crest height above mean water level.

As mentioned before, the water flow decelerates on the back barrier and the velocity decreases, mainly due to lateral spreading and infiltration. Donnelly et al. (2005) assumed that the flow on the back barrier is steady and described by a block of water moving down a slope, assuming that the infiltration velocity is proportional to the height

of the block. By using the continuity equation for conservation of water for the block, the equation describing the sediment transport rate is given by,

$$\frac{q_f}{q_D} = \left(\frac{u}{u_D}\right)^3 = \xi^3$$

(4.26)

where $\xi = \dfrac{u}{u_D} = \dfrac{1}{\sqrt{1+s'}}e^{-\frac{1}{2}\alpha'\frac{s'}{\xi}}$ is the non-dimensional velocity of block, $s' = \mu\dfrac{s}{B_D}$ is the lateral spreading parameter, and $\alpha' = \dfrac{\alpha B_D}{\mu u_D}$ is the infiltration parameter.

This overwash algorithm was implemented into the SBEACH numerical model, and the model version is denoted as the new SBEACH version, as previously pointed out.

Chapter 5: RESULTS AND DISCUSSIONS

5.1 EMPIRICAL MODEL

As mentioned before, the Williams model 2 was derived based on the assumption that the overwash transport rate increases rapidly for small values of excess runup and asymptotically approaches zero in an exponential form as excess runup increases and hence does not realistically represent overwash processes. In this section, only the Williams model 1 and Tanaka's formula will be tested with the compiled data sets.

To apply the empirical overwash models the extreme conditions of each storm as shown in table 3.4 are taken as being representative during the entire time of excess runup. The results show that the Williams formula grossly underestimated the measured field volumes (Figure 5.1). The constants in the Williams formula (model 1) were derived based only on his laboratory experiments, so scale effects might be a problem when applying his formula to the field data.

The empirical constants in Tanaka's formula were derived based mainly on the laboratory experiments of overtopping discharge by runup and two measured points in Gamo lagoon (1993, 1994). The effect of inundation overwash and the excess runup duration were not considered. Tanaka's formula overestimates the measured overwash volumes in the field, as shown in Figure 5.2. In principle, Tanaka's formula is similar to Williams', but the predictions by the formulas are an-order-of-magnitude different because of the difference in the values of the empirical constants.

Table 5.1: Calculated overwash sediment volumes using Williams (1978), Tanaka (2002), formula compared with measured field volumes.

Profile name	Measured overwash volume (m3/m)	Williams's formula (m3/m)	Tanaka's formula (m3/m)
Folly Beach 2801	4.75	0.0299	156.84
Folly Beach 2815	11.89	0.0298	170.48
Folly Beach 2823	5.58	0.0299	139.96
Folly Beach 2883B	4.19	0.0299	136.19
Garden City 4930	17.58	0.0362	185.87
Santa Rosa Island, Opal	55.7	0.047	338.6
Santa Rosa Island, Georges	57.13	0.2678	89.39
Assateague Island, GPS1	50.1	0.7275	4.43
Assateague Island, GPS3	101.4	0.9194	3.42
Assateague Island, GPS4	91.5	0.1745	716.69
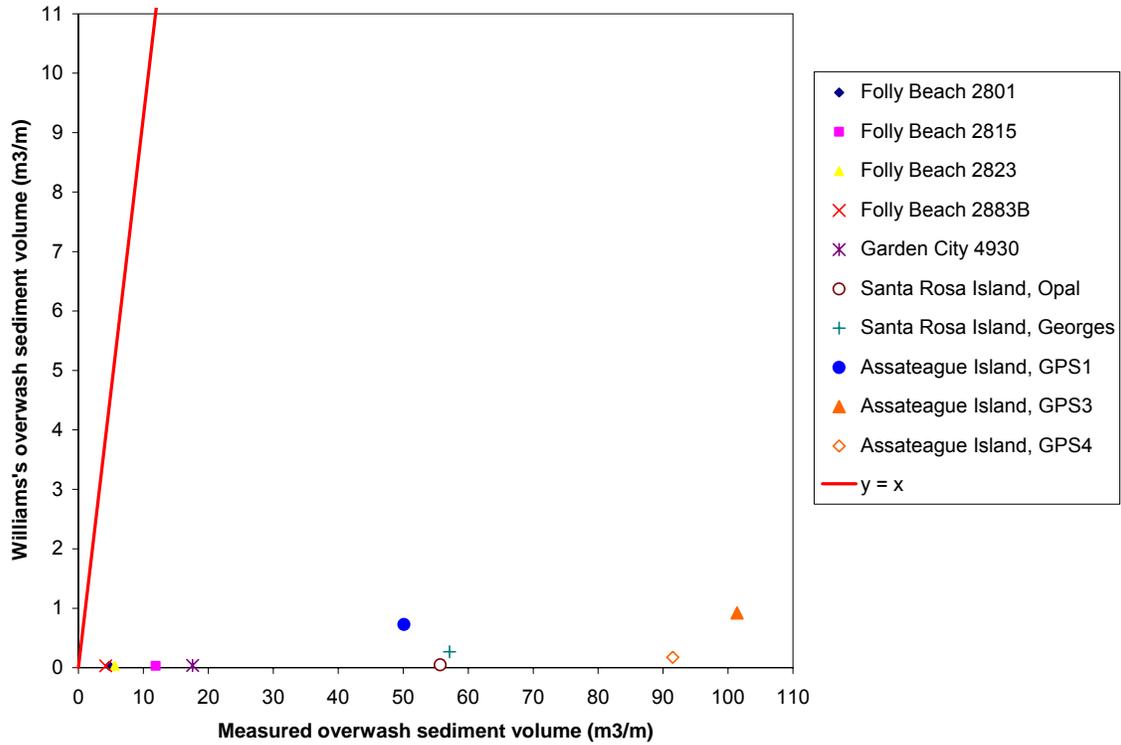
Fig. 5.1: Williams's overwash formula compared with measured overwash sediment volume.
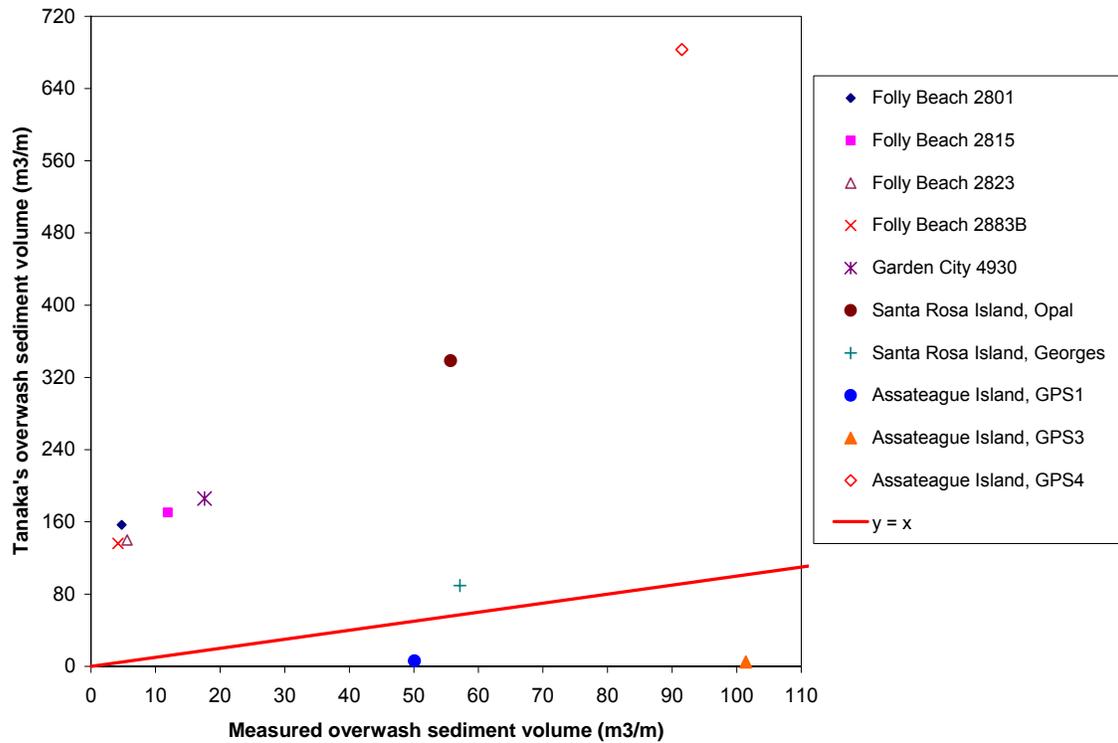
Fig. 5.2: Tanaka's Overwash formula compared with measured overwash sediment volume.

5.1.1 A New empirical formula for overwash volume

There are a limited number of studies on sediment transport volume such as Williams (1978), Tanaka et al. (2002), and Larson et al. (2004). The constants in Williams' formula were based on laboratory experiments, whereas Tanaka's formula was derived from laboratory data and limited field measurements. Since both formulas were essentially derived from small-scale experiments, there might be a problem in applying them to the prototype. Larson's formula is untested with any data sets because of the lack of full field measurements. In this study, a formula for the prototype scale is derived based on the compiled data sets. The sediment transport rate in the overwash is assumed to be proportional to the excess runup height squared and the inverse of the wave period. This formulation has similarities to Williams (1978), Tominaga and Sakuma (1971), and Larson et al. (2004), but is written as:

$$q = \alpha \frac{H_c}{R} \frac{1}{T} (R - H_c)^2 \tag{5.1}$$

The sediment transport volume is proportional to the sediment transport rate multiplied by the excess runup duration (t_D),

$$Q = qt_D = \alpha \frac{H_c}{R} \frac{t_D}{T} (R - H_c)^2 = \alpha \frac{H_c}{R} \frac{t_D}{T} \Delta R^2 \tag{5.2}$$

where Q is the total sediment transport volume (m³/m), R is the wave runup height (m), and H_c the beach crest height (m) referenced to the still-water level. The term $\dfrac{H_c}{R}$ is added in the formula to consider the effects of a limited dune height on the mobilization of material from the dune (pers. comm. with M. Larson – November 2005), t_D is the overwash duration (s), T is the wave period (s), and α is an empirical coefficient.

Using the data set encompassing eleven overwash events from United States, the best fit was obtained when the empirical coefficient, α, was set to 0.0011 (Figure 5.3). The predictions by the new formula compared quite well with the measured washover sediment volume:

$$Q = 0.0011 \frac{H_c}{R} \frac{t_D}{T} (R - H_c)^2 \tag{5.3}$$

Most of the predicted overwash volumes fell within a factor two of the observed volumes, as shown in Figure 5.4, indicating that the goodness of fit of the empirical coefficient

Table 5.2: New formula results compared with measured field volumes.

Profile name	Measured overwash volume (m3/m)	A new formula (m3/m)
Folly Beach 2801	4.75	8.54
Folly Beach 2815	11.89	7.87
Folly Beach 2823	5.58	8.54
Folly Beach 2883B	4.19	8.45
Garden City 4930	17.58	11.22
Santa Rosa Island, Opal	55.7	18.6
Santa Rosa Island, Georges	57.13	81.89
Assateague Island, GPS1	50.1	126.35
Assateague Island, GPS3	101.4	63.09
Assateague Island, GPS4	91.5	72.96
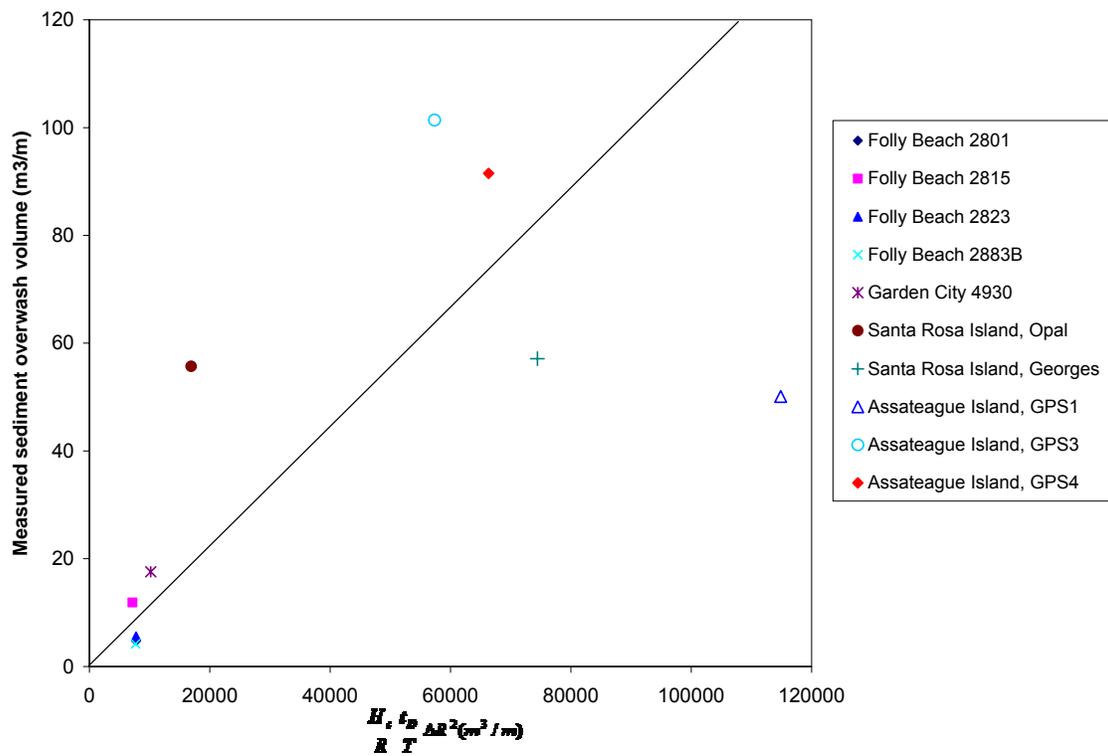
Fig. 5.3: The empirical formula for predicting overwash
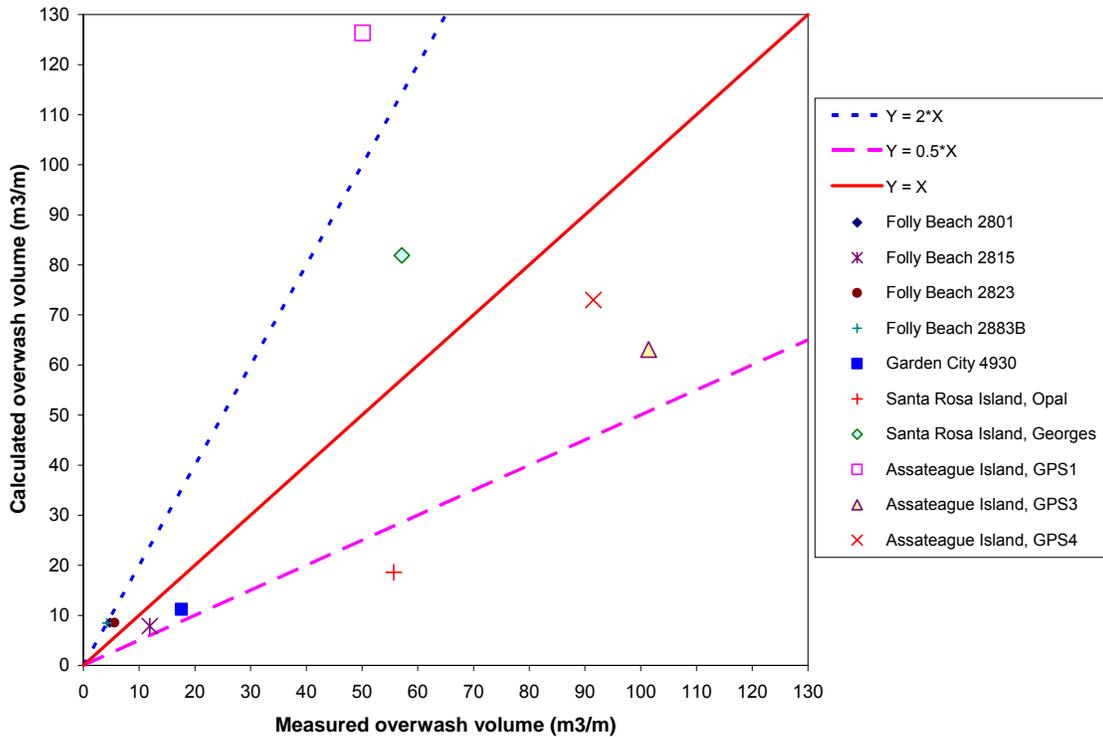
Fig. 5.4: A new formula results compared with measured overwash sediment volume

5.2 SBEACH NUMERICAL MODEL

5.2.1 Calibration and validation

The original and new SBEACH versions were calibrated using default values employed in previous SBEACH applications as much as possible for all parameters that have limited influence on the overwash. The overwash-related parameters were then changed to find the best visual fit to the measured beach profiles. Main parameters that affect overwash transport are the sediment transport rate coefficient (*K*), the overwash transport parameter, the lateral spreading of the flow (*CMY*), and the infiltration parameter (*CAL*). Each parameter has an effect on various aspects of beach profile change as modelled in SBEACH. Best agreement was achieved with the parameter values shown in tables 5.3 to 5.6.

The *K* value influences the overall magnitude of overwash because it controls the amount of seaward erosion and recession of the foreshore slope, which in turn controls when and to what extent overwash occurs. In general, values of *K* which provide the best agreement in terms of foreshore erosion and washover volumes are typically a bit larger than the default value presently recommended for SBEACH, although the scatter in the *K*-values is marked. The calibration procedure was sometimes difficult to carry out, since increasing *K* to sufficiently erode the beach face reduced the washover sediment volume and vice versa.

In the new SBEACH version, the flow on the backbarrier is assumed to be steady and described by a block of water moving down-slope. The hydrodynamics on the back barrier are calculated taking into account the effects of lateral spreading (*CMY*) and infiltration (*CAL*) on the flow. The lateral spreading of a washover deposit on the backbarrier is largely controlled by the backbarrier topography. In this study, the new SBEACH was tested with values on *CMY* of 0.075, 0.005, and 0.025. The infiltration parameter may influence the landward movement of sediment if the barrier is dry in the initial stages of an overwash event. In such cases, infiltration into the back barrier is rapid and deposition occurs as the flow rate on the back barrier is reduced. It was assumed that the backbarrier was saturated in most of the present simulations and the infiltration parameter was set to 0.01 based on trial and error. Some sensitivity analysis was also performed with respect to the infiltration parameter.

Other parameters that affect the profile response are the seaward depth of the foreshore (*DFS*) and the median sediment grain size. The parameter *DFS* defines the landward boundary of the surf zone and it effects the magnitude of transport in the swash zone. A higher value on *DFS*, corresponding to a greater depth at the boundary between the surf zone and swash zone, typically increases the transport in the swash zone, producing more erosion of the foreshore and dune. Conversely, lower values of *DFS* typically decrease the erosion in this region (default value of *DFS* is 0.3 m).

The median sediment grain size is a key parameter in calculating the transport rate and in determining the equilibrium energy dissipation. A reliable estimate of the median grain size is necessary to obtain reliable results from the model. In cases where there is uncertainty about the proper value of the median grain size, this parameter could be estimated by fitting an equilibrium profile to the studied profile and back-calculating the grain size from the shape parameter A. Of course, this was only possible where the beach profile extended sufficiently seaward to define an equilibrium beach profile.

In most cases, the measured beach profiles were extended by using the equilibrium concept. Profiles were extended until -15 m water depth to be able to neglect the effect of waves on the ocean floor at the seaward boundary. The horizontal length of the modelled profiles was approximately 1500 m and the profiles were divided into two portions with different grid resolution. The shoreward portion of the grid had a cell size of 2 m and extended from the most landward point to a water depth of about -1 m, whereas the seaward portion of the grid had a cell size of 10 m.

Table 5.3: Calibrated parameters for running original version of SBEACH overwash model

| Profile name | Original SBEACH calibrated parameters | | | | | | | |
| | Reach Configurations | | | Sediment transport parameters | | | | |
	DFS (m)	D_{50} (mm)	Aval. angle	Transport rate coefficient (m4/N)	Overwash transport parameter	Eps (m2/S)	Lamm.	Water temp. (^{0}C)
Folly Beach 2883B	0.03	0.17	30	1.20E-06	0.0005	0.005	0.3	20
Garden City 4930	0.05	0.44	30	1.0E-06	0.0047	0.005	0.5	20
Santa Rosa Island, Opal	0.05	0.26	30	2.50E-06	0.0047	0.002	0.3	20
Santa Rosa Island, Georges	0.02	0.26	30	2.70E-07	0.00029	0.002	0.3	20
Assateague Island, GP1	0.1	0.30	30	2.00E-06	0.017	0.004	0.4	20
Assateague Island, GP3	0.5	0.30	30	2.50E-06	0.005	0.004	0.3	20
Assateague Island, GP4	0.5	0.30	30	2.50E-06	0.009	0.005	0.5	20

Table 5.4: Calibrated parameters for running new version of SBEACH model with CMY=0.075, and CAL=0.01

| Profile name | New SBEACH simulation CMY=0.075, CAL=0.01 | | | | | | | |
| | Reach configurations | | | Sediment transport parameters | | | | |
	DFS (m)	D_{50} (mm)	Aval. angle	Transport rate coefficient (m4/N)	Overwash transport parameter	Eps (m2/S)	Lamm.	Water temp. (^{0}C)
Folly Beach 2883B	0.03	0.17	30	1.00E-06	0.0008	0.002	0.3	20
Garden City 4930	0.05	0.44	30	2.0E-06	0.0047	0.005	0.5	20
Santa Rosa Island, Opal	0.05	0.26	30	2.50E-06	0.00896	0.002	0.3	20
Santa Rosa Island, Georges	0.02	0.26	30	2.50E-07	0.0005	0.002	0.3	20
Assateague Island, GP1	0.5	0.30	30	2.50E-07	0.018	0.004	0.4	20
Assateague Island, GP3	0.5	0.30	30	2.50E-06	0.004	0.004	0.3	20
Assateague Island, GP4	0.5	0.30	30	1.00E-06	0.0025	0.005	0.5	20

Table 5.5: Calibration parameters for running new version of SBEACH model with CMY=0,025 and CAL=0.01

| Profile name | New SBEACH simulation CMY=0.025, CAL=0.01 | | | | | | | |
| | Reach Configurations | | | Sediment transport parameters | | | | |
	DFS (m)	D_{50} (mm)	Aval. angle	Transport rate coefficient (m4/N)	Overwash transport parameter	Eps (m2/S)	Lamm.	Water temp. (^{0}C)
Folly Beach 2883B	0.03	0.17	30	7.00E-07	0.0008	0.002	0.3	20
Garden City 4930	0.05	0.44	30	1.75e-006	0.005	0.005	0.5	20
Santa Rosa Island, Opal	0.05	0.26	30	2.50E-06	0.00347	0.002	0.3	20
Santa Rosa Island, Georges	0.02	0.26	30	2.50E-07	0.00014	0.002	0.3	20
Assateague Island, GP1	0.5	0.30	30	2.50E-07	0.009	0.004	0.4	20
Assateague Island, GP3	0.5	0.30	30	5.00E-07	0.0013	0.004	0.3	20
Assateague Island, GP4	0.5	0.30	30	2.50E-06	0.0006	0.004	0.3	20

Table 5.6: Calibrated parameters for running new version of SBEACH model with CMY=0.005 and CAL=0.01

| Profile name | New SBEACH simulation CMY=0.005, CAL=0.01 | | | | | | | |
| | Reach Configurations | | | Sediment transport parameters | | | | |
	DFS (m)	D_{50} (mm)	Aval. angle	Transport rate coefficient (m4/N)	Overwash transport parameter	Eps (m2/S)	Lamm.	Water temp. (^{0}C)
Folly Beach 2883B	0.03	0.17	30	1.20E-06	0.0008	0.002	0.3	20
Garden City 4930	0.05	0.44	30	1.5e-006	0.0045	0.005	0.5	20
Santa Rosa Island, Opal	0.05	0.26	30	2.50E-06	0.0048	0.002	0.3	20
Santa Rosa Island, Georges	0.02	0.26	30	2.70E-07	0.00029	0.002	0.3	20
Assateague Island, GP1	0.5	0.30	30	2.50E-07	0.016	0.004	0.4	20
Assateague Island, GP3	0.5	0.30	30	2.50E-06	0.0009	0.004	0.3	20
Assateague Island, GP4	0.5	0.30	30	2.50E-06	0.001	0.005	0.5	20
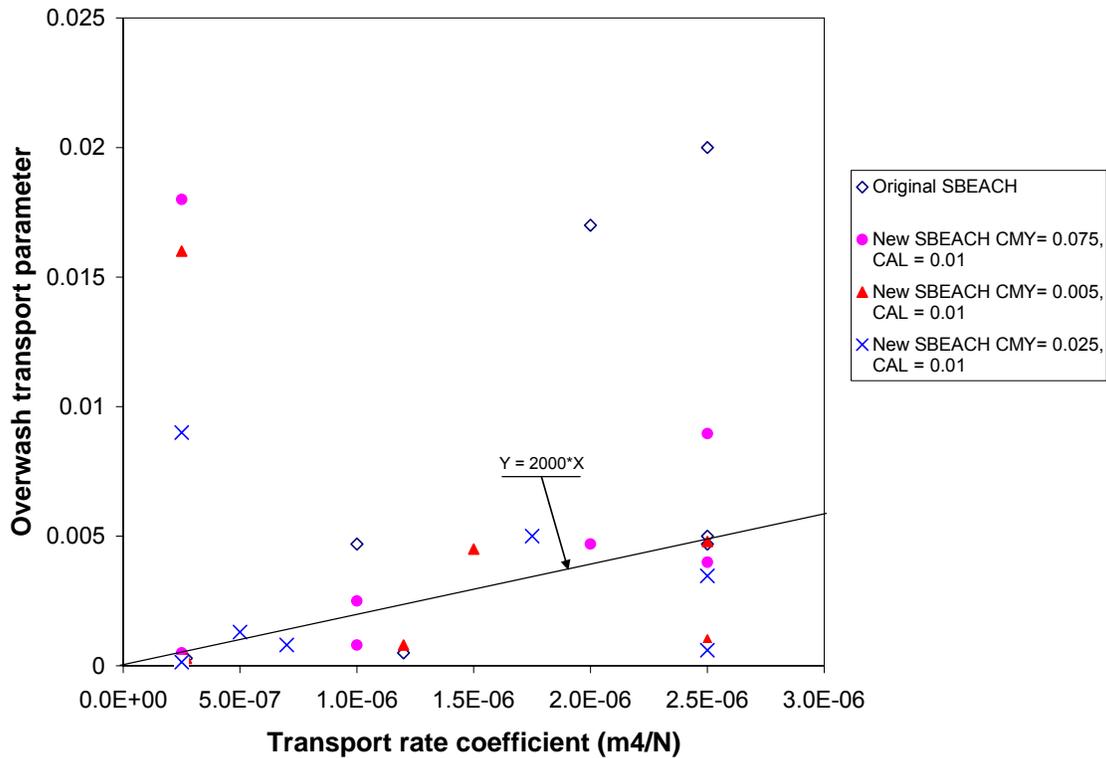
Fig. 5.5: Relationship between the overwash transport parameter and the transport rate coefficient

Although the calibration parameters ideally should be uncorrelated, it might be difficult in practice to achieve this for a complex model. Thus, *K* and the overwash transport parameter displayed some degree of correlation and a relationship between these two parameters could be developed. In general, the overwash transport parameter increases with *K* and a simple linear relationship was derived to describe the relationship (Figure 5.5). It is suggested that this equation may be used to provide a first estimate on the overwash transport parameter in the calibration procedure.

5.2.2 Simulation results

In this section, the simulation results of the original and new SBEACH model are presented. Both SBEACH versions can simulate runup overwash and associated beach profile response due to extreme storms. However, the original SBEACH model does not have an algorithm for inundation. If the SWL exceeds the crest, the dune or beach crest is treated as part of the surf zone, and the SWL is assumed to be constant towards the end of the grid. Donnelly et al. (2005) added a new algorithm to simulate inundation overwash, denoted as the new SBEACH model here. The comparison of both SBEACH versions with measured field data is discussed in the following section.

Folly Beach

Dune erosion and overwash in Folly beach is modelled for hurricane Hugo from the 15th to 25th September 1989. The hydrodynamic conditions of Hugo are plotted in Figure 3.9. Hurricane Hugo had a peak water level of 2.01 m at Folly Beach, and a peak wave height of 6.69 m at water depth of 16 m, producing significant dune erosion and overwash. The model results indicated that the shape of washover deposition and its landward extent are well reproduced. Beach erosion and the dune crest locations are somewhat underpredicted. In general the simulation results of the old and new SBEACH versions are in good agreement with the measurements. Both versions are equally capable of reproducing the observed profile change (Figure 5.6).
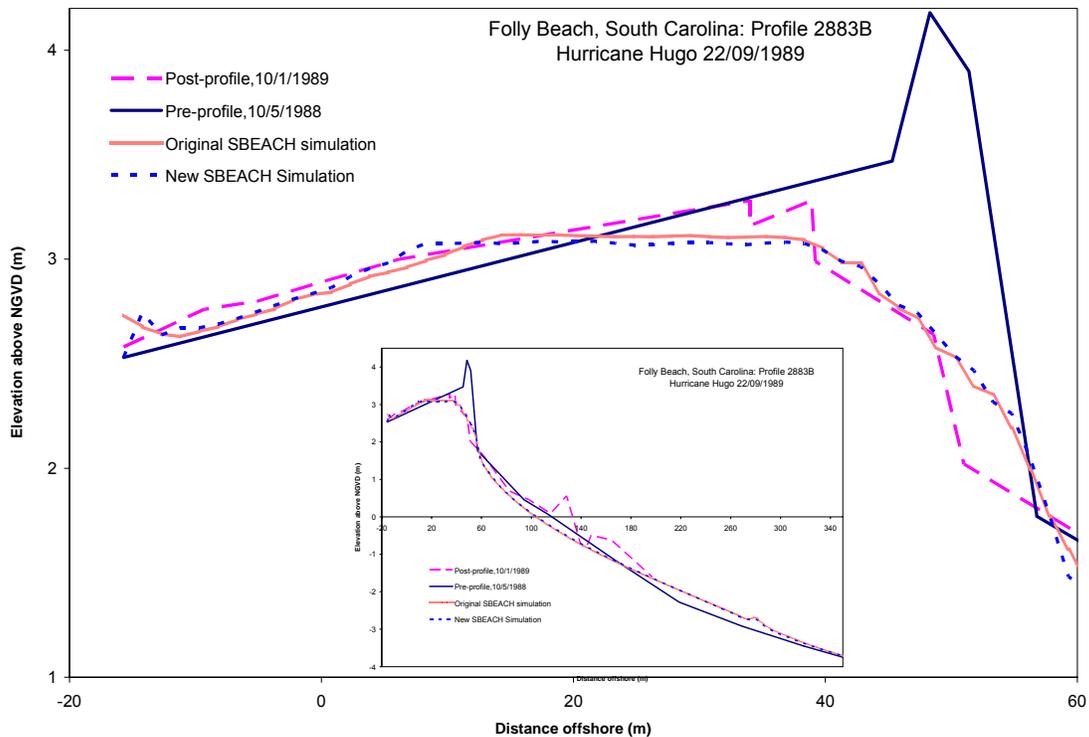

Fig. 5.6: Simulation results of original and new SBEACH version: Folly Beach 2883B profile

Garden City Beach

The post-Hugo profile in Garden City Beach was surveyed 6 days after landfall and indicated that Hurricane Hugo contributed significantly to beach and dune destruction, as well as causing overwash, with sediment being transported both offshore and onshore. SBEACH simulated well the total deposited washover volume, the change in profile elevation, and the landward extend of sediment deposition. The results indicate that the new version gave a better simulation of washover deposition. Both of the models failed to accurately reproduce the beach face response. This case shows that if *K* is increased to fit the foredune erosion very little sediment goes over the dune crest, indicating a shortcoming in the model or that perhaps extra sediment was taken from the profile by longshore transport processes. The simulation results are shown in Figure 5.7.
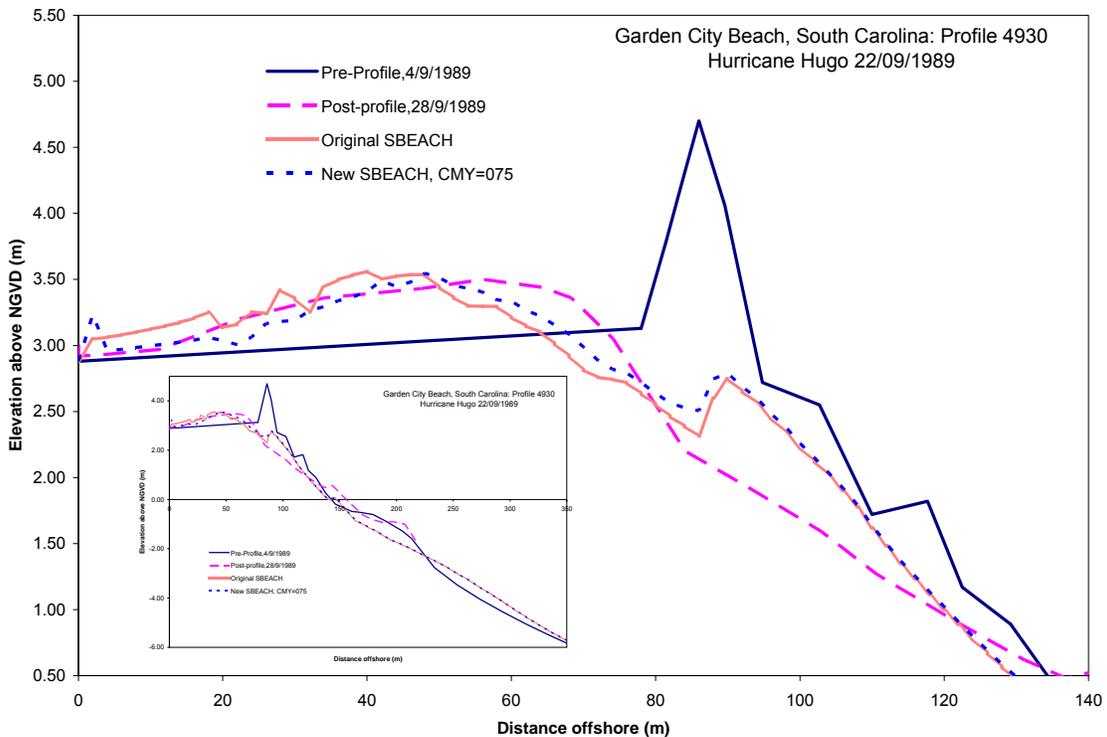
Fig. 5.7: Simulation results of original and new SBEACH version: Garden City Beach 4930 profile

Santa Rosa Island – Hurricane Opal

Santa Rosa Island is a barrier island which was impacted by hurricane Opal on the 4th of October 1995. The hydrodynamic conditions are plotted in Figure 3.13. Hurricane Opal had a peak wave height of 8.2 m in 27 m of water depth, a peak wave period of 11.2 s, and a peak water level of 2.6 m. These extreme conditions destroyed the foredunes and inundation overwash occurred. The post-Opal beach profile was measured three days after the storm and beach recovery was probably negligible. The simulation results for both SBEACH versions reproduced the beach profile change satisfactorily in terms of

beach erosion, washover sediment volume, dune destruction, as well as the landward extent of deposition. The modelled offshore beach slopes are also in agreement with the measured profile (Figure 5.8, inset).
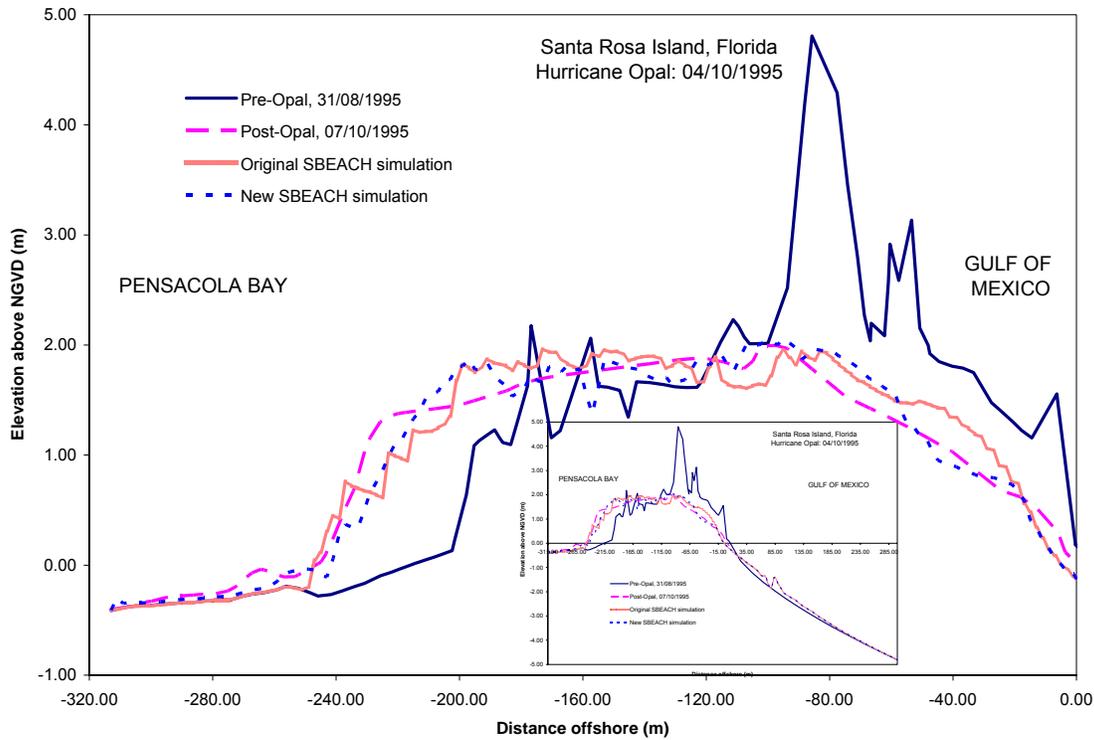
Fig. 5.8: Simulation results of original and new SBEACH version: Santa Rosa Island-Opal profile

Santa Rosa Island – Hurricane Georges

The beach crest height at this location is low because the foredunes were destroyed by Hurricane Opal in 1995 and did not recover before Hurricane Georges occurred on the 28th of September 1998. Georges had a maximum wave height of 8.2 m measured in 20 m of water depth, and a peak water level of 1.4 m above NGVD. The barrier crest height is of 2.12 m above NGVD. The post-Georges profiles show that there is significant erosion on the dune face and washover sediment deposits, but the peak of the barrier island was even higher than before the storm. If there was only a dune present, it would most likely have eroded away. Thus, it is probable that some kind of hard structure exists at this location, but it was not possible to simulate its effect in the SBEACH simulations (Figure 5.9). The SBEACH model is capable of simulating beach profiles with structures such as seawalls and revetment, but at this stage, not overtopping of these structures.
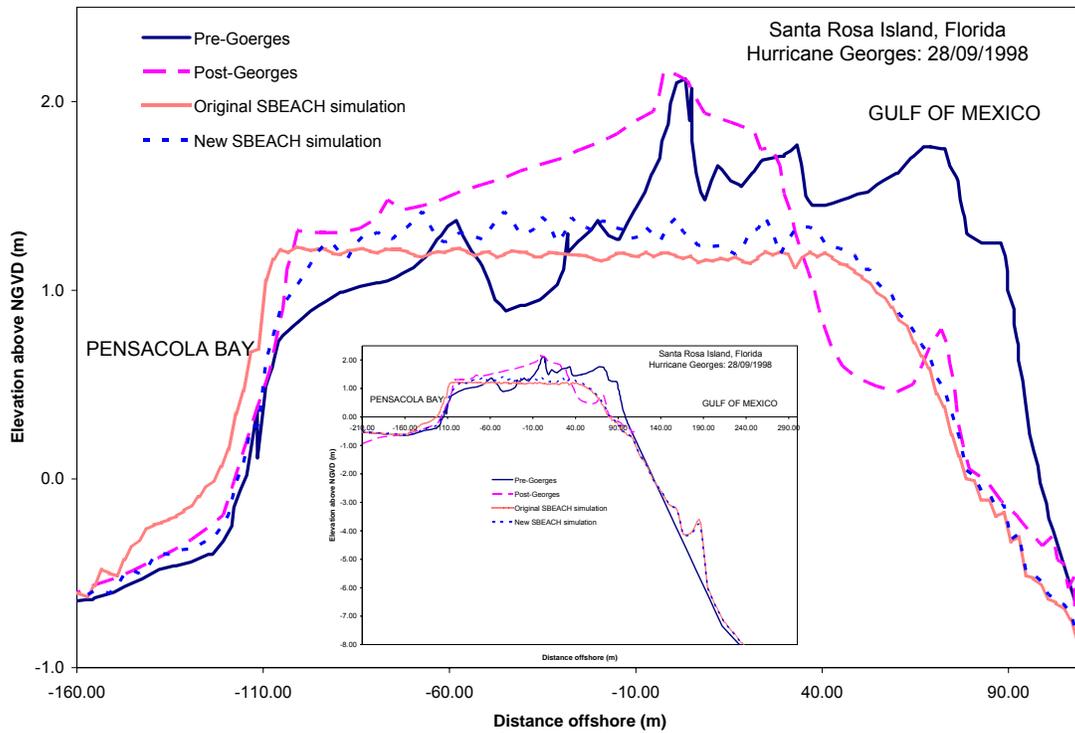
Fig. 5.9: Simulation results of original and new SBEACH version: Santa Rosa Island-Georges profile

Assateague Island – Northeasters

Assateague Island is a low barrier Island that was inundated by the northeasters that occurred in January and February 1998 (Figure 3.19). There were two major storms in this period with a maximum wave height of 4.1 m in 15 m of water depth, 1.6 m peak water level, and a peak wave period of 16 s. Pre-storm profiles were surveyed on the 1st of September 1997 and the post-storm measurements were taken in February 1998.

Figures 5.10, 5.11, and 5.12 show the measured and modelled profile response for the three transects GPS1, GPS3, and GPS4, respectively. GPS1 shows an unusual barrier island profile with a significant dune located towards the rear part of the barrier. This dune caused sediment to accumulate on its seaward side because the water flow was blocked. The model accurately simulated horizontal erosion of the berm crest, and the landward deposition. Both the measured and modelled response show washover deposits extending approximately 160 m landwards of the pre-storm berm crest, up to the base of the dune. The modelled volume of washover is lower than indicated by the measurements, but the elevation of the berm crest lowering is simulated well by the original model. The new SBEACH model slightly overestimates the crest elevation reduction. Post storm changes are observed on the rear dune slope of the measured profile. The pre-storm profile on the backside of the second dune of GPS1 has a lower

resolution than the post-profile which is surveyed from LIDAR with very high resolution; hence theses irregularities may also exist for the pre-storm profile, but are not seen. It is unlikely that the irregularities were caused by overtopping of the rear dune. In general, both of SBEACH versions are in agreement with the measured field profile (Figure 5.10).
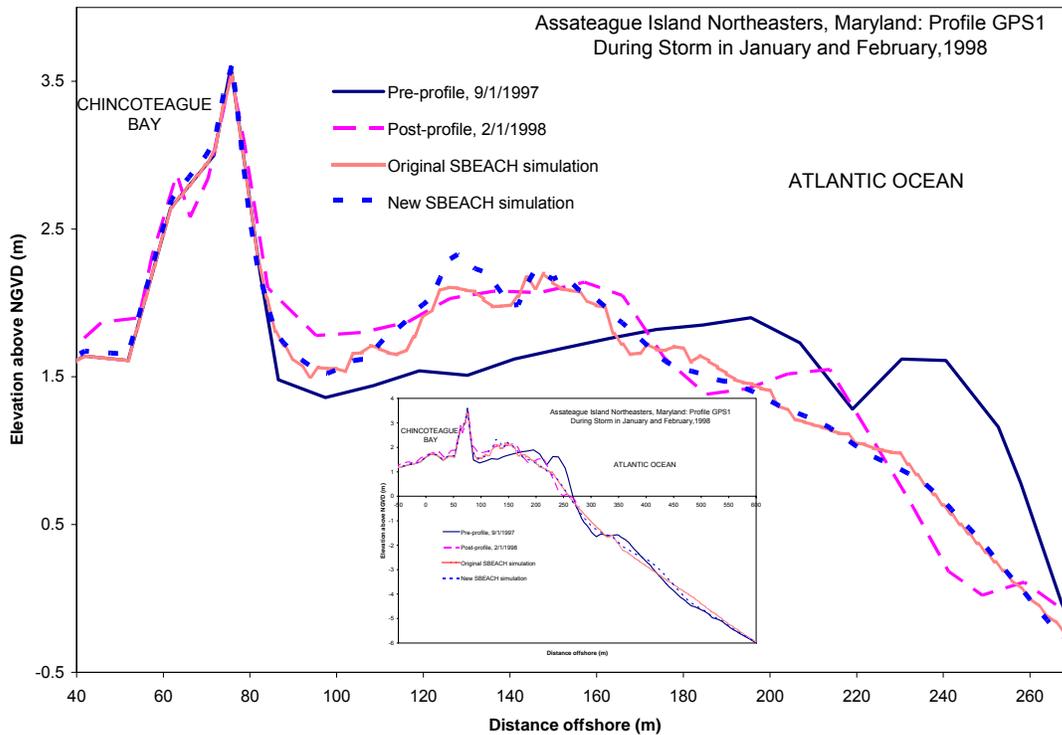

Fig. 5.10: Simulation results of original and new SBEACH version: Assateague Island GPS1 profile

At the Assateague Island GPS3 profile, the northeasters caused significant erosion of the berm crest and deposited large volumes of washover sediment. The simulations of both SBEACH versions reproduced the shape of the washover deposits well, the peak of the barrier lowering, and the landward extent of sediment intrusion, but the barrier face erosion and horizontal recession of the beach crest are somewhat overestimated. Note that this profile also has a rear dune which, to some extent, blocked and reduced the landward overwash flow. In this case, however, the rear dune was overwashed. Both versions of SBEACH produced comparable results concerning the shape of the rear dune. The new SBEACH simulated the rollover of the rear dune well. In general, the model results are in good agreement with the measured beach profiles. The new SBEACH version works performs better than the original one at the rear of the profile (Figure 5.11).

Assateague Island GPS4 profile has a low and relatively flat back barrier. The simulation results with both SBEACH versions yield a lower beach face recession than the measurements suggest (note that the post-storm profile does not extend very far

seaward). Upper beach face recession and landward migration of the crest are overestimated. Washover sediment volume calculated by the models is comparable to the measurements. The new algorithm works better for a horizontal slope and for the case of inundation, since the new algorithm is not dependent on the rear slope, including factors such as the effects of lateral spreading and infiltration (Figure 5.12).
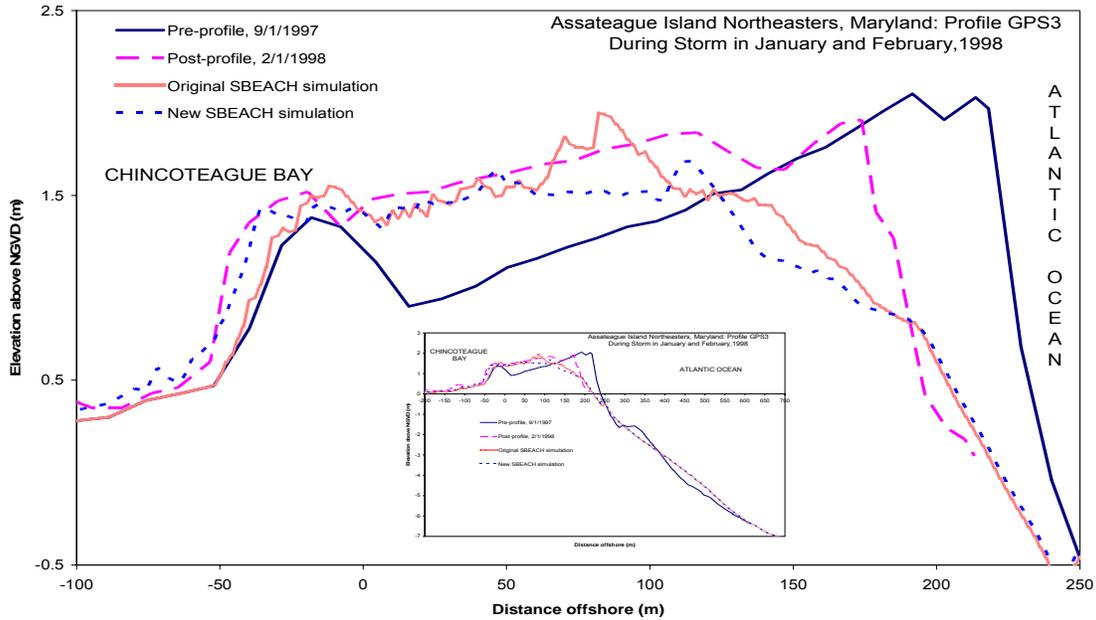

*Fig. 5.11: Simulation results of original and new SBEACH version: Assateague Island GPS3 profile*
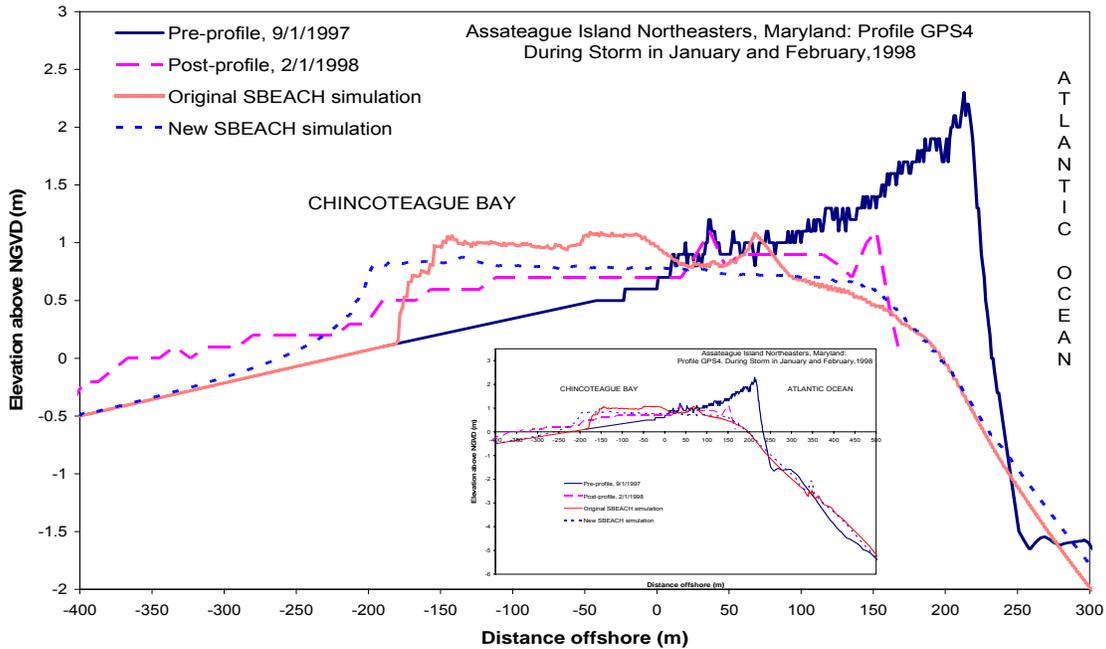
Fig. 5.12: Simulation results of original and new SBEACH version: Assateague Island GPS4 profile

5.2.3 Sensitivity

In this section, the sensitivity of the original and new SBEACH overwash models to changes in some key input variables, such as wave height and water level, and key input parameters, such as lateral spreading (*CMY*) and infiltration (*CAL*), will be investigated.

Effects of the wave height

In order to test the effects of variation in wave height on the SBEACH models, the wave height was varied from 7.64 m to 9.14 m and the wave period from 12.35 s to 13.85 s, while the water level was held constant. The profile used is a schematized version of Folly Beach 2883B, and the purpose of the test was to only simulate runup overwash. As expected, the larger the wave height is the larger the washover sediment volume is. No major differences between the old and new SBEACH simulations were observed, since profile changes were mostly due to runup overwash (Figure 5.13 and Figure 5.14). The magnitude of the crest recession relative to the magnitude of wave height change is approximately 50 cm for every 0.5 m increase in wave height for the old SBEACH model and 35 cm for every 0.5 m increase in wave height for the old SBEACH model. The landward end of the crest receeds 3 m for every 0.5 m increase in wave height, for both models.
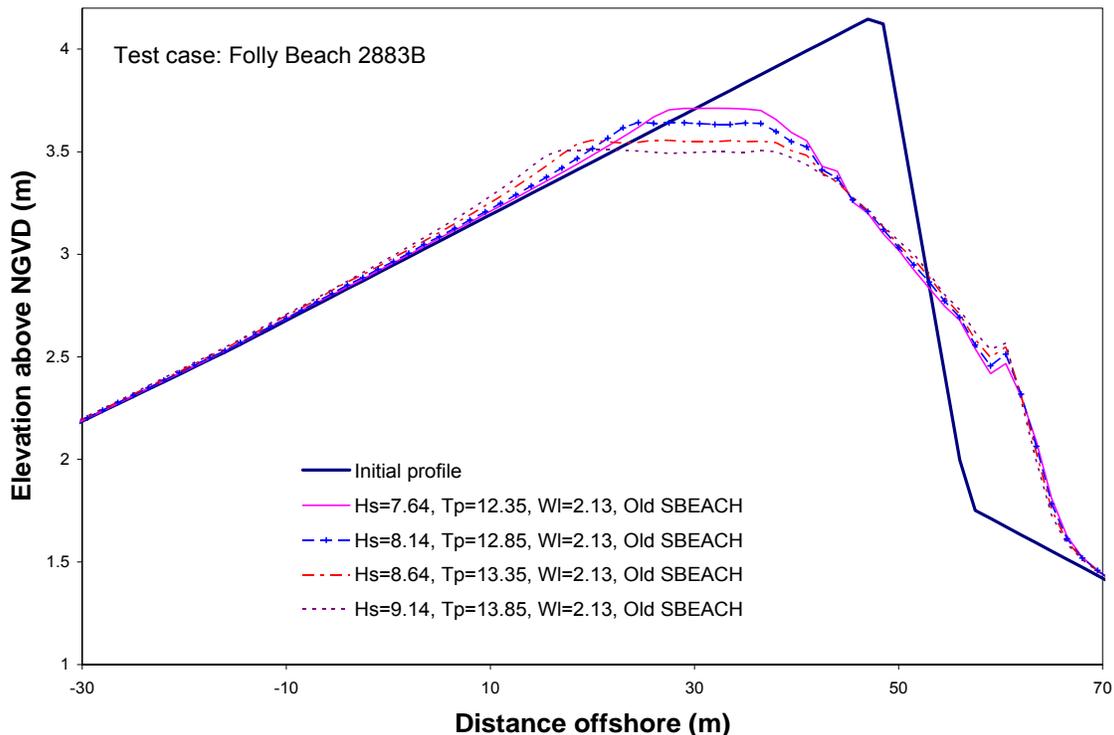
Fig. 5.13: The effect of Wave height changing on a schematize profile in the original SBEACH overwash model.
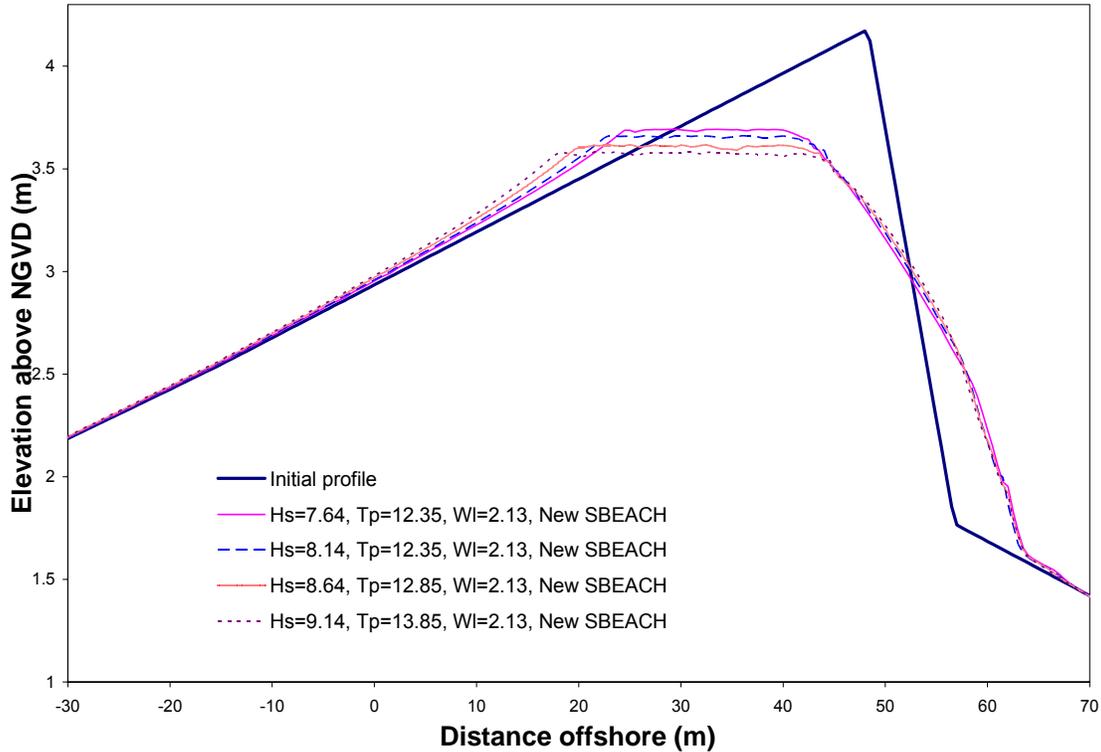
Fig.5.14: The effect of Wave height changing on a schematize profile in the new SBEACH overwash model

Effects of water level

The same schematized beach profile was used to evaluate the model sensitivity to changes in water level. The water level was varied from 2.13 m to 4.63 m. The wave height and wave period were held constant at 7.64 m and 12.85, respectively. Note that the beach crest is 4.18 m so that means the beach profile is inundated in the two last cases. The original SBEACH simulations show that it cannot handle the two last case of inundation overwash; the beach crest reacts like a bar, as illustrated in Figure 5.15. The version with the new algorithm to simulate inundation overwash worked much better, since larger washover volumes are predicted as the excess water level increases (Figure 5.16). Both versions result in similar results for the first case because the water level is much lower than the beach crest, implying that overwash is only due to wave runup.
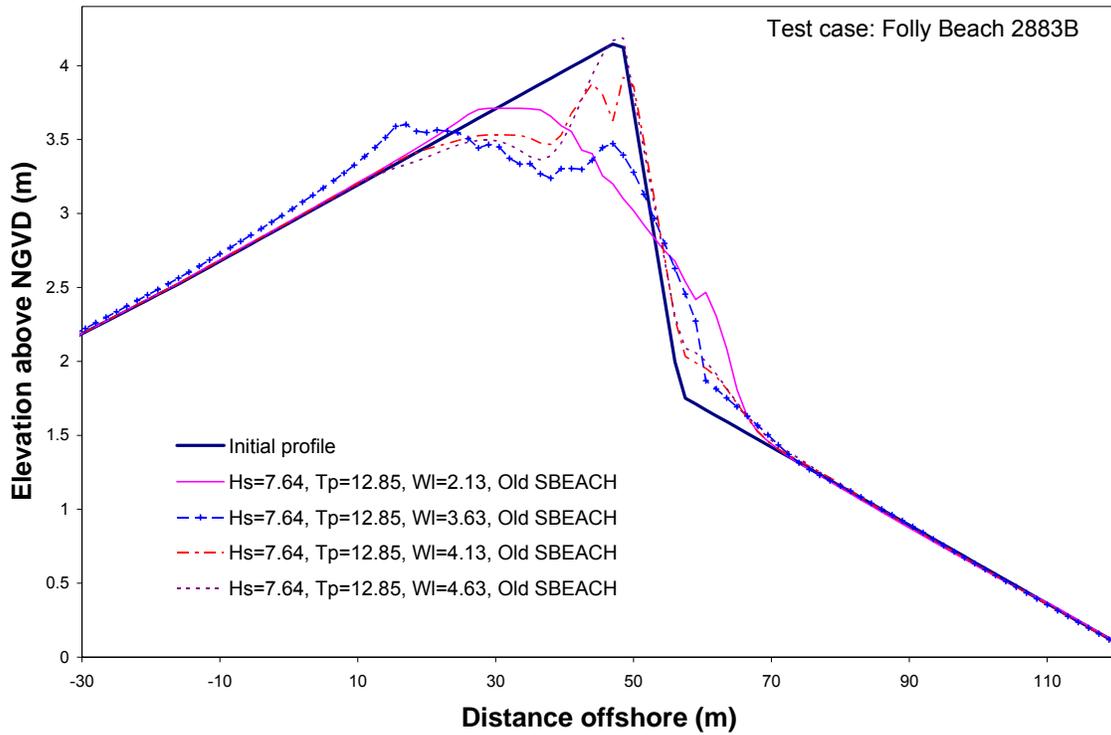
Fig. 5.15: The effect of water level change on a schematized profile with the original SBEACH overwash model.
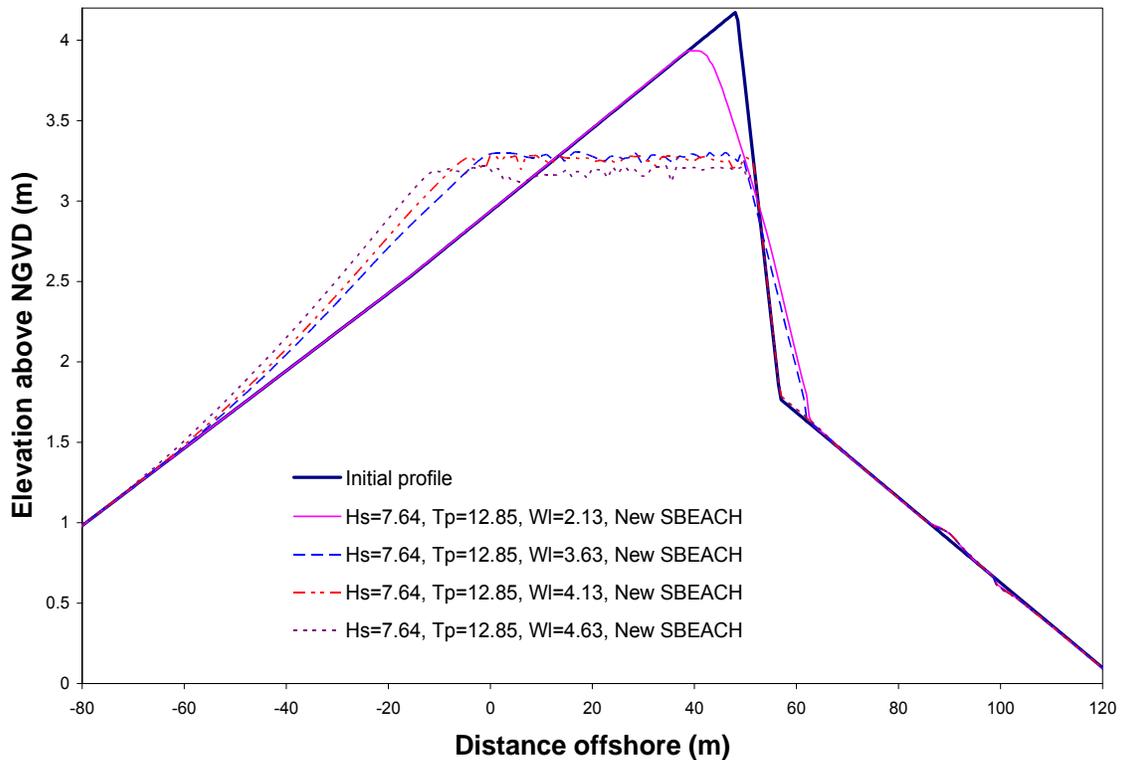
Fig. 5.16: The effect of water level change on a schematized profile with the new SBEACH overwash model

Two different combinations of water level and wave height, which together give a similar maximum water elevation, were simulated to test whether the water level or wave height mainly affects the overwash sediment processes. As mentioned before, the original SBEACH version can only simulate runup overwash, which is clearly seen in Figure 5.17. Case 1 with a high water level of 5 m, implying inundation overwash, did not cause any significant the change in the profile shape, as expected.
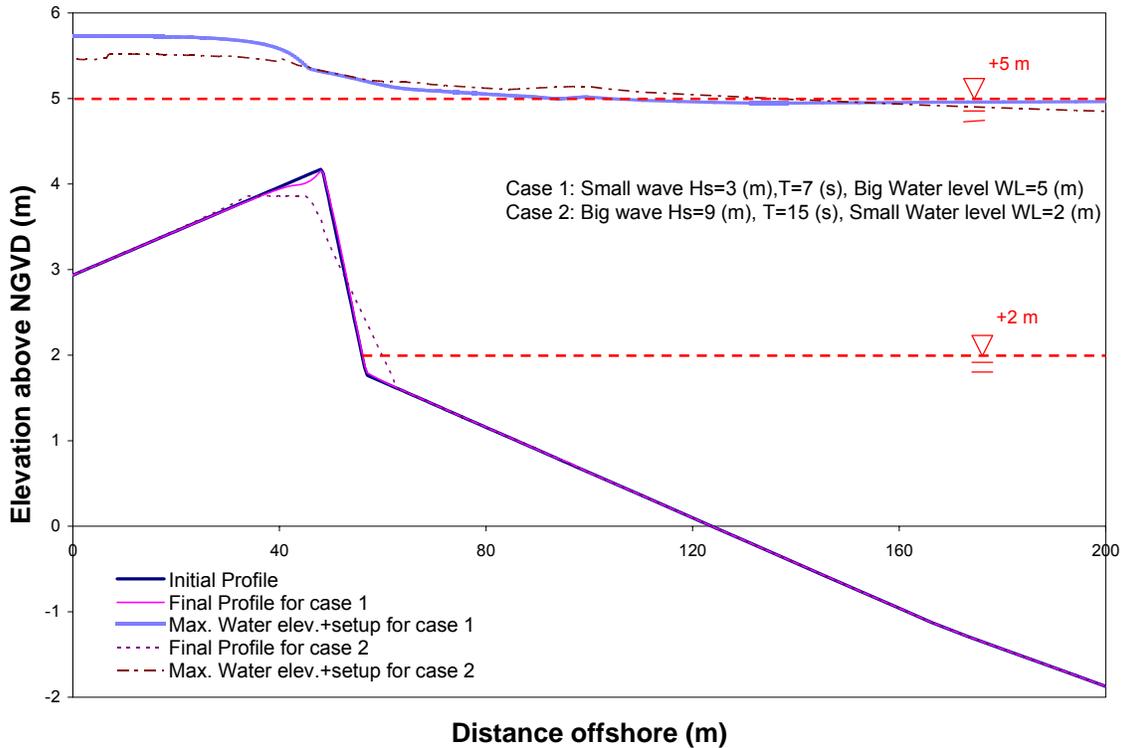
Fig. 5.17: Two cases to test the overwash by runup and inundation in original SBEACH overwash model

Figure 5.18 illustrates the results with the new SBEACH version. For runup overwash (Case 2) the results is the same as for original version, but for the inundation case the new version produces a more realistic profile shape than the original model. If the entire dune profile is inundated, considerable overwash deposits landward of the dune crest is expected.
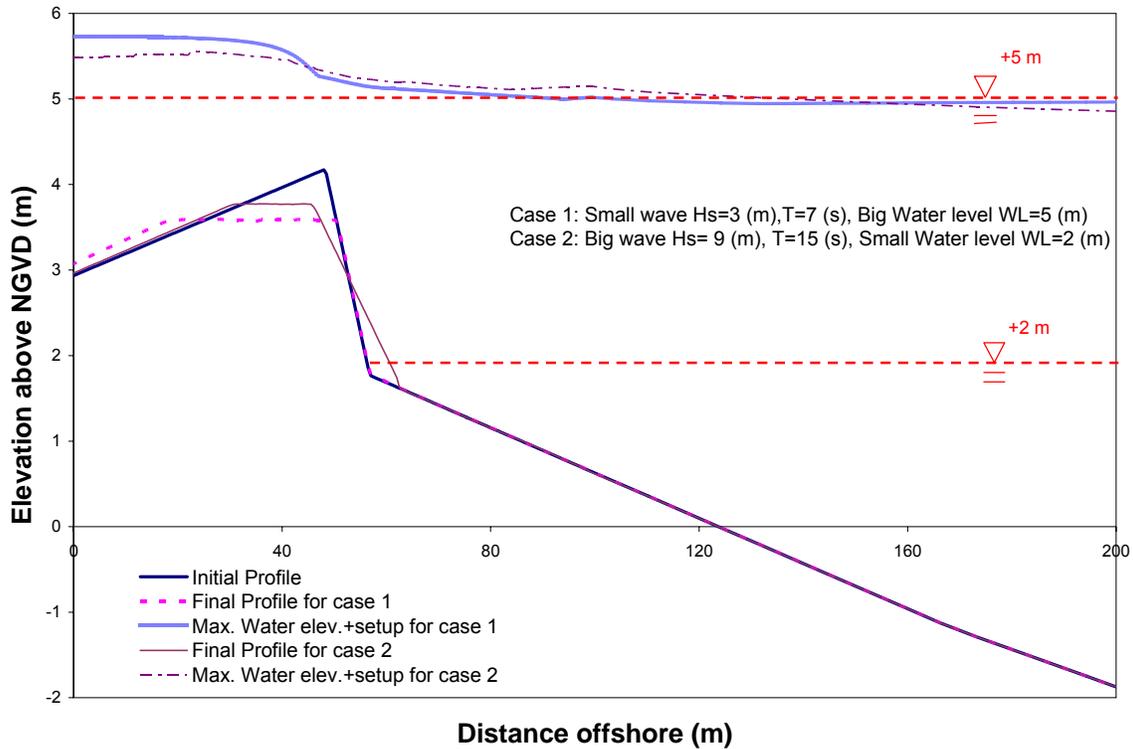
Fig. 5.18: Two cases to test the overwash by runup and inundation in new SBEACH overwash model

Sensitivity testing of calibration parameters

The two main processes affecting morphology change on the backside of the barrier are lateral spreading and infiltration, which are represented by *CMY* and *CAL* in the model, respectively. In order to see the effects of lateral spreading (*CMY*), all other parameters were held constant and the lateral spreading parameter increased from 0.005 to 0.01. Figures 5.19 and 5.20 show that the reduction of the lateral spreading rate allows the washover deposit to penetrate further landward because a reduced lateral spreading rate means that the flow decelerates less. Additionally, the recession of the barrier peak is reduced. This factor does not affect the beach face and the profile seaward of the beach face.

By increasing the value of the infiltration parameter from 0.005 to 0.01 with all other parameters held constant in the model, the effects of the infiltration parameter on the profile response can be observed. In the new model, the overwashing flow was assumed steady and was described by a block of water moving down a slope. The infiltration rate was assumed proportional to the height of this block of water. If the barrier is dry in the initial stages of an overwash event, infiltration rate on the backbarrier is quite rapid and deposition occurs as the flow rate on the backbarrier decreases. As the barrier becomes wetter, the washover sediment may penetrate further inland. Increasing the infiltration parameter causes the washover sediment to be deposited closer to the beach crest and

the reduction of the peak is smaller, because the flow volume is faster reduced and hence decelerates quicker.
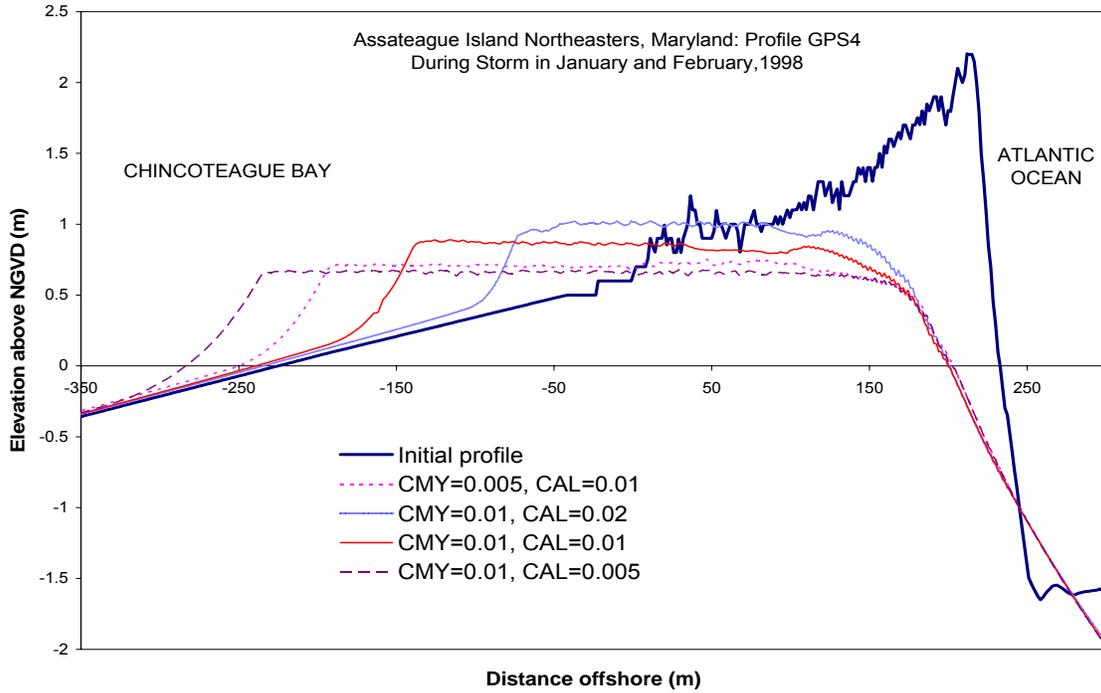

Fig. 5.19: Profile GPS4: Testing the effects of infiltration parameter on the new SBEACH overwash model
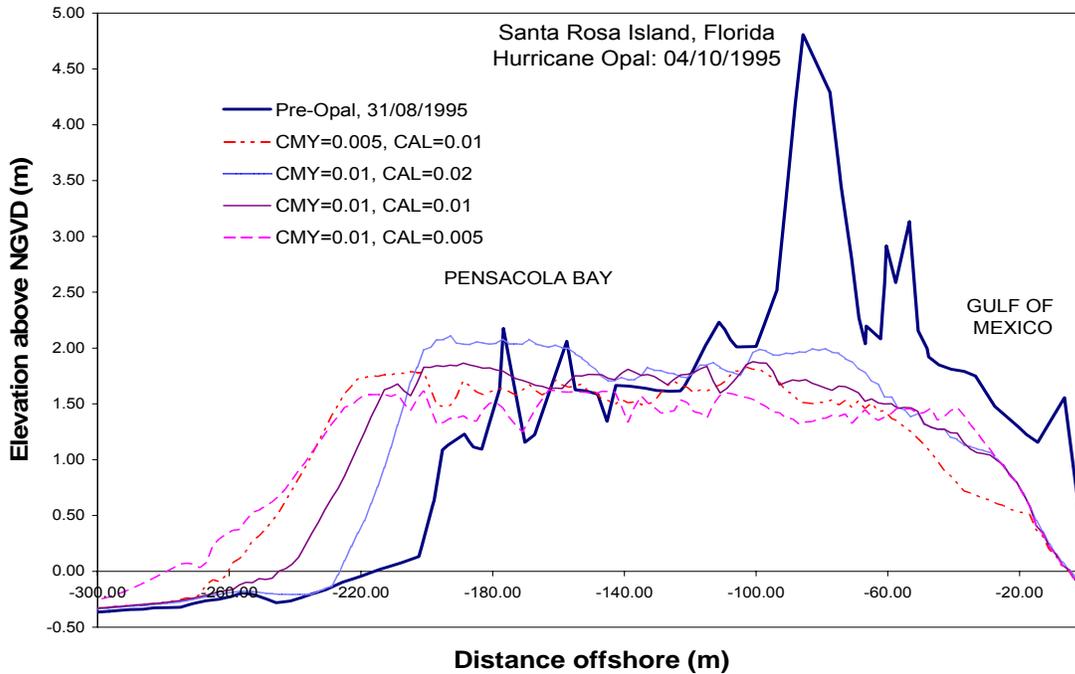

Fig. 5.20: Profile Santa Rosa Island: Testing the effects of infiltration parameter on the new SBEACH overwash model

5.2.4 Laboratory simulations

In early 2005, Donnelly et al. (2006) conducted an overwash experiment in a 3.05 m wide, 0.9 m deep, and 63 m long, two-dimensional wave flume at the Coastal and Hydraulics Laboratory, Vicksburg, United States. The flume was constructed of concrete, with a 1:47 slope between a distance 0.4 m and 20.7 m downstream of the wave generator at the end of the flume. In other parts the concrete bottom was flat and horizontal. Six consecutive glass windows provided an opportunity to view a length of 14.6 m through the side of the flume. The windows were positioned 9.7 m from the downstream end of the flume. The flume was equipped with a piston type wave generator. The median grain size during the experiment was d_{50}=0.16.

The barrier profile was designed to replicate a low, flat, duneless barrier island. The seaward slope was 1:6, while the landward slope shoreward of the crest was 1:100, and the rear landward slope 1:3. Three experimental tests were conducted on this profile using a low water level and overwashing waves, such that 1st regime runup overwash (crest accumulation) occurred, and a higher water level and overwashing waves such that overwash bordering on the threshold to inundation overwash occurred. The runup overwash experiment was conducted with both regular and irregular waves. The hydrodynamic conditions of the three tests are summarized in the Table 5.6. Beach profiles were measured at the sides (North and South), and the centreline of the flume, before and after each experiment using a total station.

Table 5.7: Hydrodynamic conditions of three tests

Test No.	Profile name	Barrier height (m)	Wave height (m)	Wave period (s)	Water level (m)	Wave characteristic
1	Centre-OWB1	0.6	0.23	7	0.4572	Runup regular
2	Centre-OWB2	0.6	0.23	5	0.4572	Runup Irregular
3	Centre-OWB3	0.6	0.23	7	0.53	Inundation regular

Figure 5.21 shows the measurement and simulation results for Test 1. In this test the water level was held constant and regular waves attacked the beach face for the test duration of 30 minutes. In the beginning, wave runup overtopped the barrier crest and this water continued to flow along the rear slope. It was observed that the flow on the rear slope was relatively clear and that no sediment transport occurred. On the crest, however, sediment accumulated, eventually impeding the magnitude of successive overwashing bores. At the rear back barrier slope, the flow accelerated, causing supercritical flow and antidunes which migrated landwards, as did the slope change. The

SBEACH model simulated the beach face slope changes and the change of the steeper rear barrier slope; however, it failed to simulate the crest accumulation.
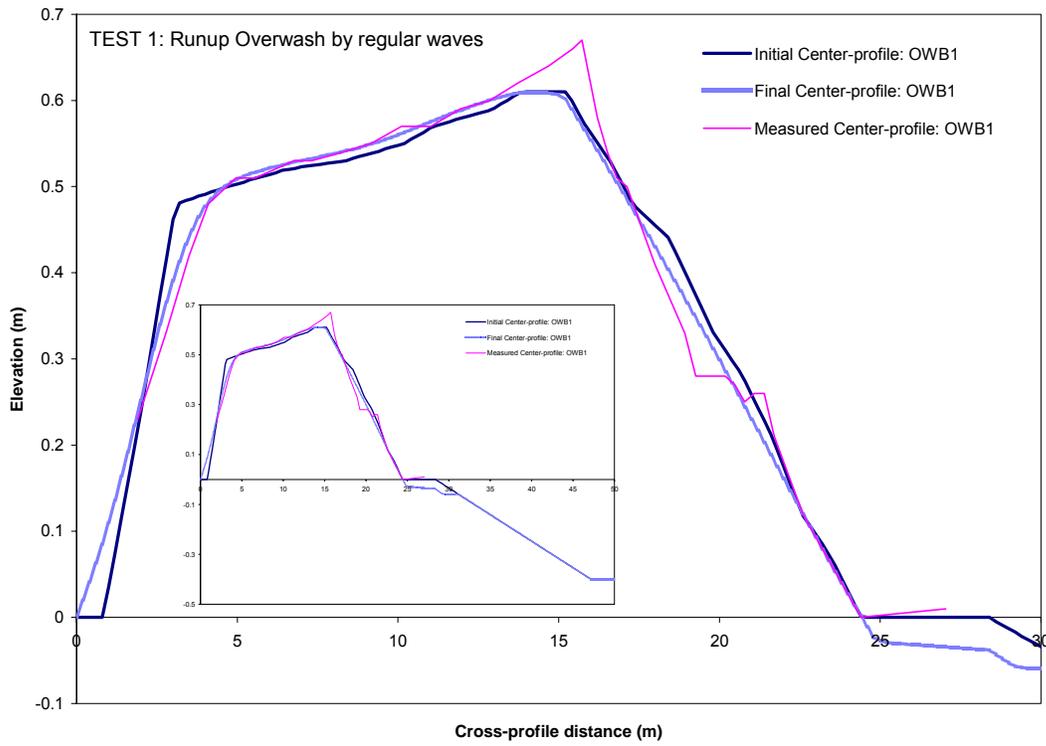

Fig. 5.21: Simulation results of the centreline OWB1 profile

The only difference between Test 1 and Test 2 is that irregular waves were employed in the latter case. Beach profile change was similar to that observed in test 1, but with a significantly smaller magnitude. Crest accumulation was also observed, but it was quite limited compared to the regular waves in Test 1. The SBEACH model was less successful to simulate the beach face steepening, crest accumulation and erosion on the steeper rear slope. It general, it is shown that the SBEACH overwash model does not simulate the case of crest accumulation well (Figure 5.22), probably because this case is not explicitly considered in the algorithms to describe overwash.

For test 3, the water level was increased until overwash bordering on the threshold between runup and inundation occurred. The wave height and wave period were the same as in Test 1. The measured profile shows that there is significant change on the back barrier and large erosion of the beach face. SBEACH underestimated the washover volume and the seaward migration of the steeper rear slope. The algorithm describing overwash flow landward of the beach crest, was made slope independent, because the velocity on the backslope was normalised with the velocity at the crest. This example shows why this algorithm won't work for complex rear slope geometries where large slope changes affect the flow regime.
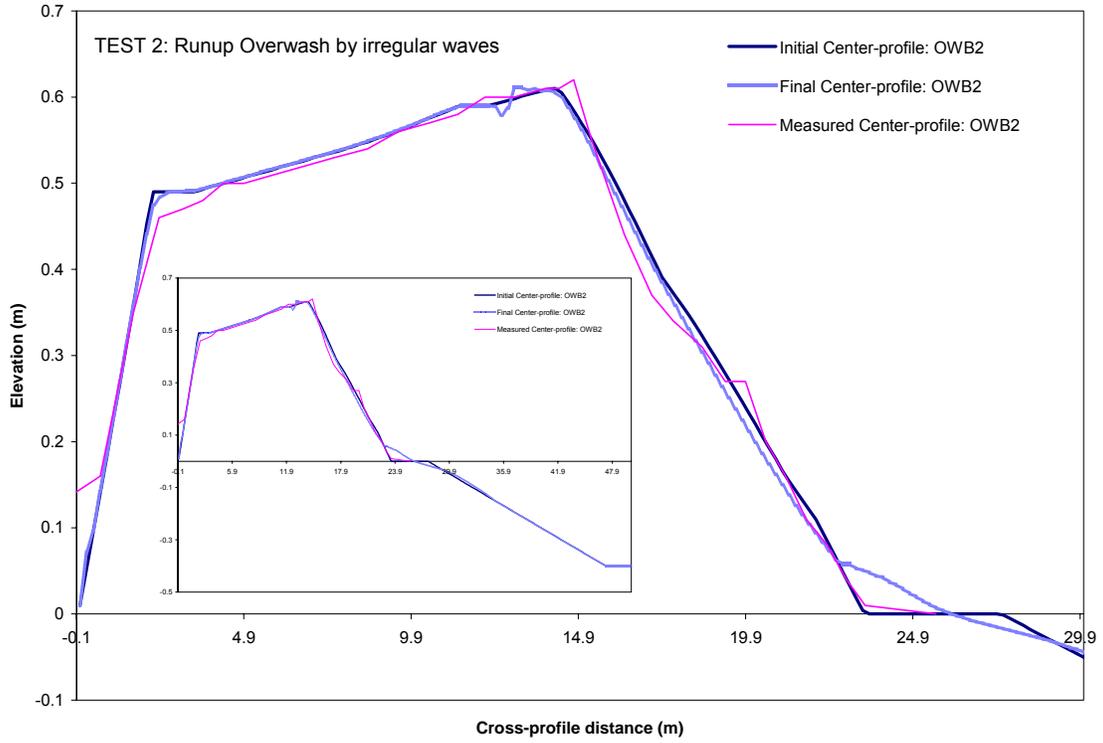
*Fig. 5.22: Simulation results of the centreline OWB2 profile*
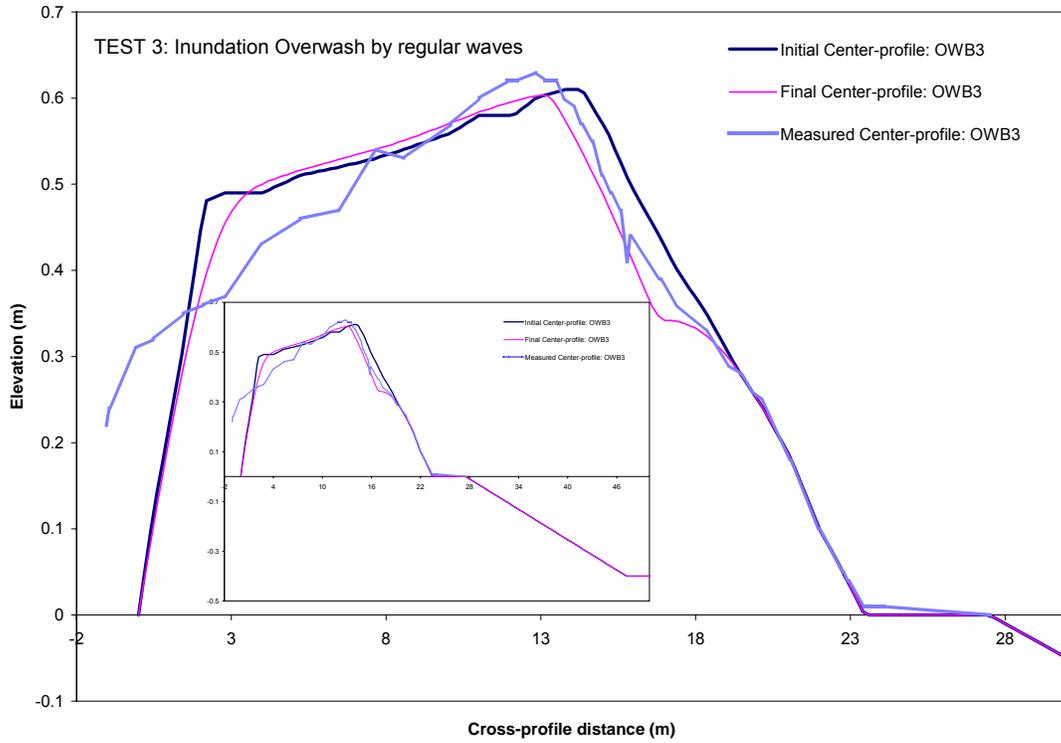
Fig. 5.23: Simulation results of the centreline OWB3 profile

Chapter 6: CONCLUSIONS AND RECOMMENDATIONS

6.1   CONCLUSIONS

The ability to model coastal overwash and the prediction of overwash occurrence are significant for engineers, local management authorities, and coastal residents. Concerning coastal overwash, two main mechanisms can be identified, namely runup overwash and inundation overwash, which can be divided into five different flow regimes. The morphologic response of beach can be divided into five different types, that is, crest accumulation, dune lowering, dune rollback, dune destruction, and barrier rollback. Linking the proposed flow regimes to the observed morphology changes is not simple, as several different flow regimes occur during a single overwash storm.

The field data sets which were compiled from three regions within United States provided an excellent opportunity to monitor the effects of hurricanes and extreme storm events on the beach profile shape, and to develop and evaluate empirical as well as numerical models. These data sets included wave height and period and water level time-series during the storm, as well as the pre and post-storm beach profiles and also the median grain size of the study areas.

The Williams (1978) and Tanaka (2002) empirical models for predicting overwash volume were evaluated using the compiled data. The results show that Williams' formula seriously underestimates the volume, whereas Tanaka's formula overestimates it, compared with the measured field data. The constants in the two formulas formula were based on laboratory experiments and very limited field data, respectively. Therefore, scaling between model and prototype is a problem, and the coefficient derived for the Tanaka model is biased towards the limited data used to derive it. A new equation was developed by modifying the assumptions of the previous empirical models and calibrating to the new data set. It was assumed that the excess runup is moving the sediment landward for duration of excess runup, t_D, and the empirical coefficient derived from the new data set is 0.0011. The new predictive formula for washover sediment volume is thus written:

$$Q = 0.0011 \frac{H_c}{R} \frac{t_D}{T} (R - H_c)^2$$

where Q is the washover sediment volume (m3/m), H_c is the beach crest height (m), R is the wave runup height (m), t_D is the excess runup duration (s), and T is the wave period (s). Overall, the formula could be well fitted to the data, within a factor of 2; however, the

performance of the model can not be entirely evaluated until the formula is tested on other field data sets in the future.

The numerical SBEACH overwash model is, to date, a very powerful tool for engineers and management authorities. The new algorithm that was recently implemented in SBEACH to simulate inundation overwash, including the profile development on the landward side of the beach crest, was validated with data on overwash from eleven locations in the United States, as well as with data from recent laboratory experiments on overwash (Donnelly 2006). The simulation results demonstrated that the model could successfully reproduce the volume and shape of washover deposits on a variety of beach profiles and for a variety of beach profile change morphologies including a low barrier island, a barrier with a foredune, dune destruction, dune rollover, and barrier rollover. The new algorithm works better than the old algorithm for a horizontal slope because it is not dependent on the back barrier slope. The model failed to simulate crest accumulation and morphology changes on back barriers with significant changes in flow regime.

6.2   RECOMMENDATIONS

As discussed in previous chapters, coastal overwash can either benefit barrier islands or where development is present can be a coastal hazard. Therefore, the basic physics needs to be understood in terms of the governing overwash processes and the resulting washover morphologies. Also, the presently available overwash modelling capabilities should be improved. Although limited hydrodynamic data on overwash have been collected, some valuable observations have been made from these data; however, more comprehensive investigations are needed to improve knowledge and modelling approaches. Thus, further studies are recommended concerning the following:

- Laboratory experiments should be made at larger scale to avoid the problem of scaling up the results to the prototype. Laboratory measurements should include regular and irregular wave, and change in water level to allow for detailed study of overwash processes.

- Field investigations should be performed more in the future at different sites. The data collection should ideally include pre- and post-storm beach profiles, water level, wave height, wave period, and wave direction, as well as wind speed and wind direction in nearshore. These would allow for a better evaluation of predictive relationships and more realistic application of the predictive models for washover volumes. In practice, this is not always possible due to the difficulties in maintaining instrument platforms in storm conditions.

- Measurements of water depth at the crest and wave velocity on the beach face, at the beach crest, and at least one point on the landward side of the crest in order to support development of overwash algorithms (Donnelly et al. 2005)

- Descriptions of the effects of friction losses and vegetation on the flow on the backside of beach profile (Donnelly et al. 2005)

- Improvement and generalization of the SBEACH overwash algorithm to make it a more useful tool for local planning authorities. The SBEACH model simulates the beach profile change well along the centreline of a washover fan, but it cannot predict the two-dimensional evolution of the fan. Taking into account local back barrier slope.

- Landward and offshore boundary conditions in the SBEACH model have to be carefully considered.

- The effects of return flow on beach face require more investigation.

References

Baldock, T.E., Hughes, M.G., Day, K. and Louys, J., 2005. Swash Overtopping and Sediment Overwash on a Truncated Beach. Coastal Engineering 52, 2005, 633-645

Bradbury, A.P., and Powell, K.A., 1992. The short term profile response of shingle spits to storm wave action. *Proceedings 23rd Coastal Engineering Conference*, ASCE, 2694-2707.

Bymes, M.R., and Gingrerich, K.J., 1987. Cross-inland profile response to Hurricane Gloria, *Proceedings Coastal Sediments '87*, ASCE, 1486-1502.

Donnelly C. 2004; Kraus, N.C., and Larson, M., 2004. *Coastal overwash*. Part 1: *Overwash of Processes*. Regional Sediment Management Demonstration Program Technical Note, ERDC/RSM-TN-14, U.S. Army Engineer Researcher and Development Center, Vicksburg, MS.

Donnelly C., Kraus, N.C., and Larson, M., 2005 "State of knowledge on measurement and modelling of coasta overwash," *Journal of Coastal Research,* (accepted)

Donnelly, C., Ranasinghe, R., and Larson, M., 2005. "Numerical Modelling of Beach Profile Change Caused by Overwash" *Proceedings of Coastal Dynamics 2005 (in press)*.

Donnelly, C.; Walmsley, T. (2006); Data Report: Movable-Bed Experiments of Coastal Barrier Island Overwash and Breaching. *Coastal and Hydraulics Engineering Technical Report* ERDC/CHL   U.S. Army Engineer Research and Development Center, Vicksburg, MS. (in preparation)

Eiser, W.C., and Birkemeier, M., 1991. "Beach profile response to Hurricane Hugo," *Proceedings Coastal sediments' 91*, ASCE, 1681-1696.

Fisher J.S., and Stauble, D.K. 1978. Washover and dune interaction on a barrier Island. *Proceedings Coastal Zone '78*, ASCE, 1611-1618.

Fisher J.S., Letherman, S.P., and Perry, Cart F.C., 1974. "Overwash processes on Assateague Island," *Proceedings 14th Coastal Engineering Conference*, ASCE, 1194-1211.

Garcia, A.W., Jarvinen, B.R and Schuck-Kolben, R.E., 1990. "Storm surge observations and model hindcast comparison for Hurricane Hugo," *Shore and Beach, Journal of the American Shore & Beach Preservation Association*. Vol. 58 - No. 4, 1990, 15-23.

Hancock M.W., 1994. "Experiments on irregular wave overtopping and overwash of dune", Master thesis in Civil Engineering, Faculty of the University of Delaware, Network, DE.

Kana, T.W., 2004. "Hurricane Hugo – 15 years later, Beach Response and Relation to Damages". *Coastal Science and Engineering 2004*, p. 10.

Kraus, N.C., Smith, J.M., and Sollitt, C.K., 1992. SUPERTANK Laboratory Data Collection Project. *Proceedings 23rd Coastal Engineering Conference*, ASCE, 2191- 2204.

Larson , M.,and Kraus, N.C., 1989 "SBEACH. Numerical Model for Simulating Storm-induced Beach Change; Report 1: Empirical Foundation and Model Development," Technical Report CERC-89-9, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Larson, M., Donnelly, C., Hanson, H., 2005. "Analytical modelling of dune response due to wave impact and overwash", *Proceedings of Coastal Engineering.*

Larson, M., Kraus N.C., and Wise R.A., 1999 "Equilibrium beach profiles under breaking and non-breaking waves", *Coastal Engineering '36*, 59-85.

Larson, M., Wise, R.A., and Kraus, N.C., 2004 "*Coastal Overwash. Part 2: Upgrade to SBEACH,*" Regional Sediment Management Demonstration Technical Note, ERDC/RSM-TN-15, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Larson, M., Wise, R.A., and Kraus, N.C., 2005 "Modelling dune response due to overwash transport," Proceedings 29th *Coastal Engineering Conference*, World Scientific Press, 2133-2145.

Leadon, M.E., 1999. "Beach, dune and offshore profile response to a severe storm event", *Coastal Sediments '99*, ASCE, 2534-2549.

Leatherman, S.P., 1976. *Quantification of overwash processes.* Ph.D Dissertation, Department of Environment Sciences, University of Virginia, Charlottesville, VA.

Leatherman. S.P., 1977. Overwash hydraulics and sediment transport. *Proceedings Coastal Sediments* 1977, ASCE, 135-148.

Mayer, R.H., Kriebel, D.L., 1994 "Wave runup on composite-slope and concave beaches", *Coastal Engineering 1994*, 2325-2339.

Miller, J.K., Dean, R.G., 2004. "A simple new shoreline change model", *Coastal Engineering* 51, (2004), 531-556.

Morton, R.A., and Sallenger A.H., 2003. Morphological impacts of extreme storms on sand beaches and barriers. *Journal of Coastal Research* 19(3), 560-573.

Moton, R.A., Gonzalez, J.L., Lopez, G.I, and Correa, I.D., 2000. Frequent non-storm washover of barrier, Pacific Coast of Colombia. *Journal of Coastal Research* 16(1), 82-87.

Ozkan-Haller, T. and Brundidge, S., 2004 "*Equilibrium beach profiles for Dalaware Beaches and effects of the gravity term*", College of Oceanic and Atmospheric Sciences, Oregon Dtate University.

Parachure, T., Dean, R.G., and Srinivas, R., 1991. *Laboratory study of overwash on barrier islands.* Progress Report, Coastal and Oceanographic Department, University of Florida.

Randall A. Wise, S. Jarrell Smith, and Larson, M.,1996. "SBEACH. Numerical Model for Simulating Storm-induced Beach Change; Report 4: "*Cross-Shore Transport Under Random Waves and Model Validation with SUPERTANK and Field Data*," Technical Report CERC-89-9, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Schwartz, R.K., 1975. *Nature and genesis of some storm washover deposits*. Technical memorandum, 61, U.S. Army Corps of Engineers, Coastal Engineering Reseach Center Ft. Belvoir, VA, 99pp.

Shallenger, A.H.Jr., 2000. "Storm impact scale for barrier islands," *Journal of Coastal Research* 16(3), 890-895.

Stone, G.W., Liu, B., Pepper, D.A, and Wang, P., 2004. "Importance of extratropical and tropical cyclones on the short-term evolution of barrier islands along the northern Gulf of Mexico, USA", *International Journal of Marine Geology, Geochemistry and Geophysics. 63-78.*

Tanaka, H., Nagasawa, T. and Suntoyo: *Sediment intrusion into Gamo Lagoon by wave overtopping*, *Proceedings of 28th International Conference on Coastal Engineering*, pp.823-835, 2002.

Titus, J.G., 1998. *"Rising Seas".* Maryland Law Review, Volume 57, 1279-1399, 1998, U.S Environmental Protection Agency.

Tominaga, M., and Sakuma, N., 1971. Wave overtopping on coastal dikes. *Proceedings of the Japanese Conference on Coastal Engineering*, 17, 133-140 (in Japanese)

U.S. Army Corps of Engineers – 1984. *Shore Protection Manua*l, Coastal Engineering Research Center. Chapter 3: *Wave and Water Level Predictions.*

Wang, P., and Kraus, N.C., 2005, "Beach profile equilibrium and patterns of wave decay and energy dissipation across the surf zone elucidated in a large-scale laboratory experiment", *Journal of Coastal Research*, 522-534.

Williams, P.L., 1978. *Laboratory development of a predictive relationship for washover volume on barrier island coastalines.* Master thesis, Department of Civil Engineering, University of Delarware, Network, DE.

Wise, R.A., and Kraus. N.C., 1993. Simulation of beach fill response to multiple storms, Ocean City, Maryland. In: *Beach Nourishment Engineering and Management Considerations*, Stauble, D.K. and Kraus, N.C. (Volume Editors), *Proceedings Coastal Zone' 93*, ACSE, 133-147.

Zheng, J., Dean, R.G., 1997. "Numerical model and intercomparisons of beach profile evolution", *Coastal Engineering 30* (1997) 169-201.

Electronically Available Data:

U.S. Geographic Survey (USGS)

http://coastal.er.usgs.gov/hurricanes/ivan/photos/florida.html (20-06-2005)

Wave Information Studies (WIS) Hindcast:

http://frf.usace.army.mil/wis/wis_main.html (20-06-2005)

National Data Buoy Center:

http://www.ndbc.noaa.gov/index.shtml (20-06-2005)

Coastal and Marine Geology Program:

http://marine.usgs.gov/ (20-06-2005)

Google Maps:

http://maps.google.com/ (15-06-2005)

Retrieve Observed Water Levels and Associated Ancillary Data

http://tidesonline.nos.noaa.gov/monitor.html (20-06-2005)

National Oceanic & Atmosphere Administration (NOAA)

http://www.noaa.gov/ (15-06-2005)

National Aeronautics & Space Administration (NASA):

http://aol.wff.nasa.gov/aoltm/projects/beachmap/ (NASA). 20-10-2005