STUDIES ON BEACH MORPHOLOGICAL CHANGES USING NUMERICAL MODELS

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by

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THIRUVANANTHAPURAM

APRIL 2011

DECLARATION

I hereby declare that the thesis entitled "STUDIES ON BEACH MORPHOLOGICAL CHANGES USING NUMERICAL MODELS" is an authentic record of research work carried out by me under the supervision and guidance of Dr. N.P. Kurian, Director, Centre for Earth Science Studies, Thiruvananthapuram, in partial fulfillment of the requirements for the Ph.D. Degree of Cochin University of Science & Technology and that no part thereof has been presented for the award of any other degree in any University.

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CERTIFICATE

This is to certify that this thesis entitled "STUDIES ON BEACH MORPHOLOGICAL CHANGES USING NUMERICAL MODELS" is an authentic record of the research work carried out by Mr. V.R. Shamji (Reg. No. 3063), under my supervision and guidance, at the Marine Sciences Division, Centre for Earth Science Studies, Thiruvananthapuram, in partial fulfillment of the requirements for the Ph. D. Degree of Cochin University of Science and Technology under the Faculty of Marine Sciences and no part thereof has been presented for the award of any degree in any University.

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To
My beloved family

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01

Introduction

Chapter 1
INTRODUCTION

1.1 Beach

The zone of unconsolidated material that extends landward from the low water line to the place where there is a marked change in material or physiographic form or to the line of permanent vegetation (usually the effective limit of storm waves) is defined as beach. The seaward limit of a beach, unless otherwise specified, is the mean low water line. A beach includes foreshore and backshore (Shore Protection Manual, 1984). Beaches usually slope gently toward the body of water they border and the beach face has a concave shape. They extend landward from the low water line to the point where there is a distinct change in material (as in a line of vegetation) or in land features (as in a cliff). Shore is a word used as synonymous to beach.

Shoreline is an imaginary line which demarcates the land and water and it is a dynamic one. It is defined as the intersection of a specified plane of water with the shore or beach (e.g., the high water shoreline would be the intersection of the plane of mean high water with the shore or beach) (Shore Protection Manual, 1984). Shoreline definition is more complex and it should consider both temporal and spatial sense. Shoreline is a dynamic feature and this boundary is of importance to coastal engineers and scientists.

1.1.1 Beach morphology

A vertical section of the beach which is called beach profile is presented in Fig.1.1. Some of the terminologies associated with the beach and beach profile are reproduced below from Shore Protection Manual (1984).

Backshore: That zone of the shore or beach lying between the foreshore and the coastline and acted upon by waves only during severe storms, especially when combined with exceptionally high water.

Bar: A submerged or emerged embankment of sand, gravel or other unconsolidated material built on the seafloor in shallow water by waves and currents.

Berm crest: The seaward limit of berm

Berm: A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action.

Foreshore: The part of the shore lying between the crest of the seaward berm and the ordinary low water mark, that is ordinarily traversed by the uprush and backrush of the waves as the tides rise and fall.

Nearshore: The region seaward of the shore (from approximately the step at the base of the surf zone) extending offshore to the toe of the shoreface.

Surf zone: The area between the outermost breaker and the limit of high tide level.

Swash zone: The portion of the beach face alternately covered by the uprush of the wave swash and exposed by the backwash.

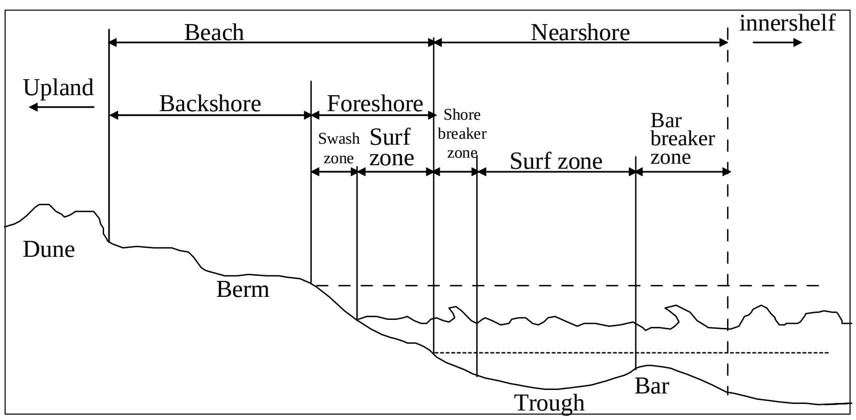


Fig. 1.1 A schematic diagram showing the different zones of the coast (Shore Protection Manual, 1984)

The terminologies ‘erosion’ and ‘accretion’ are used to describe beach profile changes over a period of time. Whenever there is a build-up of material in a temporal frame, the beach is said to accrete. Alternatively when there is loss of sediment from the beach, it is said to erode.

Another method of describing beach morphological changes is in terms of the advance or retreat of shoreline. An advance of shoreline is indicative of accretion while retreat is indicative of erosion.

Erosion/accretion or shoreline change can be both short-term and long-term depending on the time scale. Along the west coast of India, seasonal (short-term) erosion occurs during the southwest monsoon. This eroded beach is normally rebuilt during the fair weather period resulting in no net erosion or accretion over a period of one year. However, in some sectors of the coastline net erosion or accretion occurs on a longer time scale, mostly due to human interventions.

1.2 Causative Factors for Beach Morphological Changes

The processes affecting beach morphological changes can be broadly classified into (i) natural and (ii) man-made. A brief discussion of the causative factors is given in the following sections.

1.2.1 Natural factors

1.2.1.1 Waves

Waves are the principal source of input energy into the coastal zone. They comprise of the ‘sea’ which is generated locally and the swells that propagate into the area from other areas, where they are generated. In deep water, the water particle motion of waves is confined to the vicinity of the surface. As a consequence, the water particle velocity and pressure fluctuation are non-existent near the bottom. Therefore, neither bottom undulation nor bottom roughness appreciably affects the wave motion in deep waters. But in shallow waters, contrary to this, the waves undergo transformation under the influence of the bottom slope/steepness, bed characteristics, coastal structures, etc. Some of the important shallow water wave transformation processes are listed below.

Shoaling: As waves move into shallow water, the group velocity slightly increases and then decreases with decreasing water depth. Where group velocity increases wave crests move further apart leading to a reduction in wave height. Decreasing group velocity occurs for most of the nearshore region so that wave crests move closer together and wave heights increase. This process is called wave shoaling. If waves are incident normal to the beach with almost straight and parallel bottom contours, change in the wave profile is solely due to the change in water depth.

Refraction: A gradient in the wave celerity occurs along the crest of a wave moving at an angle to underwater contours because that part of the wave in deeper water is moving faster than the part in shallow water. This variation causes the wave crest to bend toward alignment with the contours. Such a kinematic process of wave transformation is referred to as wave refraction (Fig. 1.2). Refraction depends on the relation of water depth to wave length. Refraction coupled with shoaling, determines the wave height in any particular water depth for a given set of incident deep water wave. The change of direction of waves results in convergence or divergence of wave energy. Refraction therefore has a significant effect on the distribution of wave height and wave energy along a coast. This variation of energy is responsible for the beach morphological changes along the coastline.

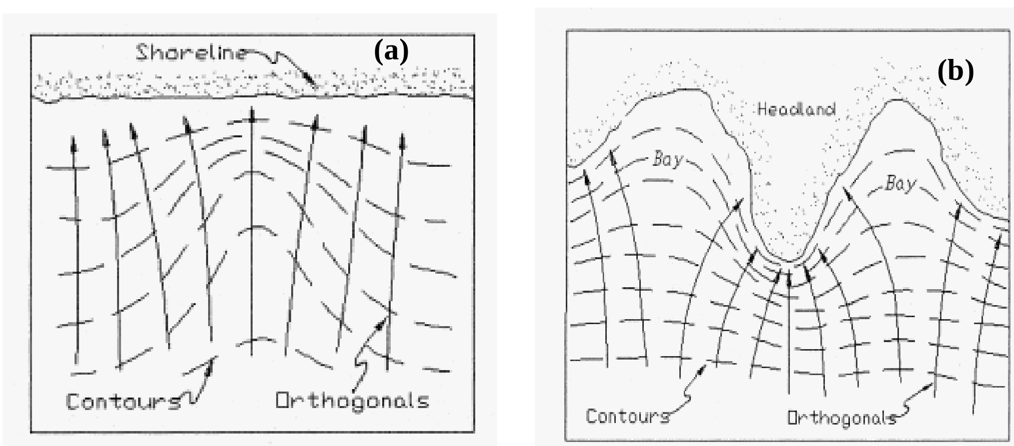


Fig. 1.2 Refraction of waves in the areas of (a) canyons (b) headland (Shore Protection Manual, 1984)

Diffraction: Waves can propagate into a sheltered basin, such as into the lee of a breakwater. This phenomenon of diffusion or transverse flow of wave energy is called wave diffraction. In diffraction, transfer of energy takes place laterally along a wave crest (Fig. 1.3).

1.2.1.2 Nearshore currents

Nearshore currents are responsible for sediment transport in the nearshore. The circulations as a result of waves breaking in the nearshore decide the sediment distribution in the nearshore area. The nearshore circulation system consists of longshore currents and a cell circulation system of rip currents.

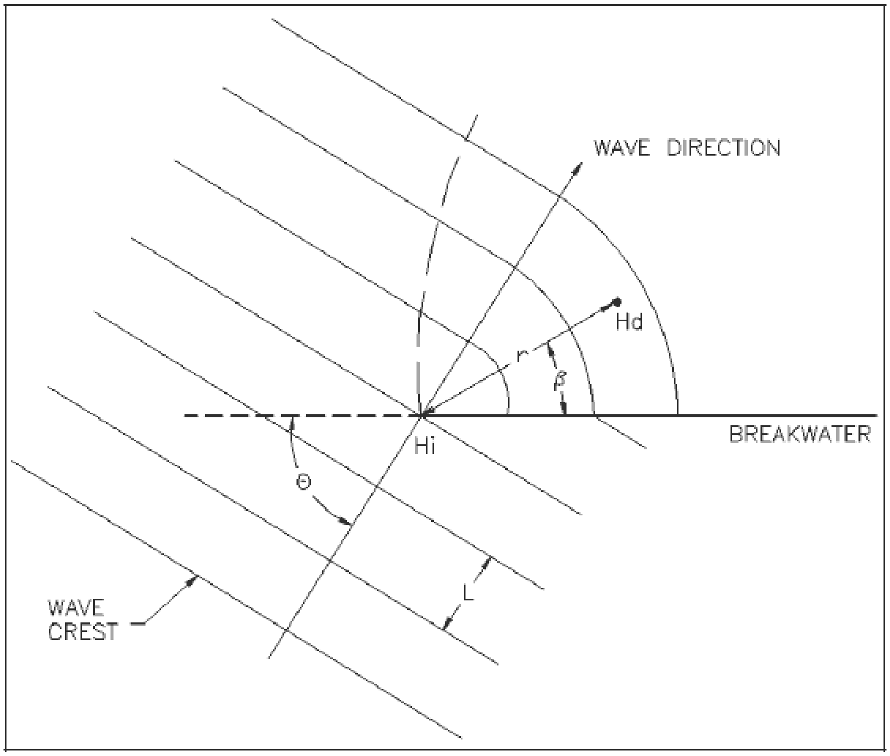


Fig. 1.3 Wave diffraction (Shore Protection Manual, 1984)

Longshore transport is an important mechanism of sediment transport caused by longshore currents that flow parallel to the coast in the surf zone. Longshore currents are generated by the longshore component of waves that obliquely approach the shoreline. The direction of longshore transport is related to the wave direction. Longshore current velocity depends mainly on two factors viz. angle between wave crest and the shoreline and breaker wave height. The longshore current velocity varies both across the surf zone and in the longshore direction. The volume of longshore sediment transport depends on five parameters: the breaker height, wave period, breaker angle with local shoreline, alongshore current velocity and the surf zone width. Rip currents are currents that flow perpendicular to the shoreline and are caused by water moving down slope (away from beach) as a result of wave setup (Fig. 1.4). Rip currents are fed by a system of longshore currents. The slow mass transport, the feeding longshore currents and the rip currents taken together form a cell circulation system in the nearshore zone.

1.2.1.3 Winds and wind-induced currents

Wind can influence the beach profile changes in two ways, directly by being a vehicle for transport of sand to and fro the beach and indirectly by influencing the hydrodynamic processes, which are the main vehicles of sediment transport in the nearshore environment. On beaches where strong seasonal wind blows, sand transport by wind is an important mechanism contributing to beach changes. If the wind speed at certain elevation reaches a critical value, sand grains on a loose sand surface begin to move. Once movement begins, winds of the same or higher speed can move the sand grains and cause their flow continuously to the downwind side. After the sand grain rises from the surface, it is acted on by the forces of gravity and force due to the wind (drag force), and when the force of gravity exceeds the drag force, the sand grain falls. In addition to this, direct effect of wind can cause onshore transport of sediments by influencing the hydrodynamic processes. Depending on the wind direction with respect to the shoreline, the wind can cause a seaward movement of the surface waters compensated by a landward near bottom current or a piling up of surface waters on the shore accompanied by an offshore undertow. In the former case accretions of the shore result while the latter leads to erosion. With offshore winds, the advancing waves tend to be reduced in height by the head winds, so that the waves reaching the shore are of lower steepness, resulting in beach accretion.

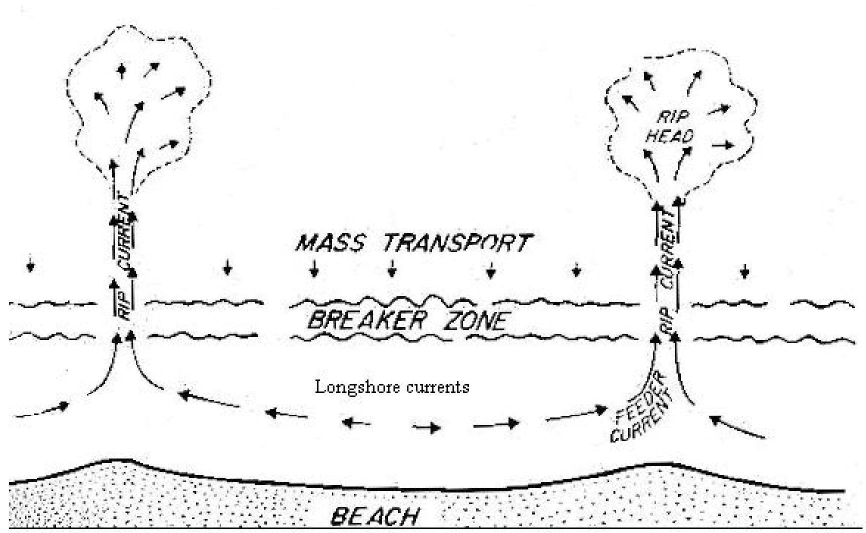


Fig. 1.4 Nearshore circulation systems (after Komar, 1976)

1.2.1.4 River/Estuary inputs

River/estuary may be a source of sediments depending upon the characteristics of the hinterland. Elevation, the types of rock, density of vegetation and the climate are the important factors determining the sediment supply. Damming of rivers has severely affected input of sediments from rivers. There are two distinct approaches to estimate the sediment supplied to the beach by the river. The first involves empirical correlation between the sediment supply, the drainage area of the river basin and the effective annual precipitation. The second approach is that of estimating the sand transport from measurements of the river discharge or velocity, by using appropriate mathematical formulations (Beer, 1998)

1.2.1.5 Geomorphologic factors

Geomorphology of coastal area may alter the positions of shoreline, which leads to erosion as well as accretion. The shoreline orientation depends on the geomorphology of the coastline. The natural geomorphologic features are bay, headland, cliff, rocky terrain, sand dunes, barrier beach etc. The nearshore processes become more complex because of the presence of these geomorphologic features. The presence of headlands and bay may change wave energy pattern in the nearshore area. Also, alongshore transport may be blocked by the presence of headland and it may cause accretion in the up-drift side and erosion in the down-drift side of the inlet.

1.2.1.6 Cliff erosion

Cliff erosion is an important source of sediments in some coastal areas. During storms, erosion of cliff occurs due to the hydrostatic pressure exerted by the waves. Rock type, orientations of rock formation, jointing and bedding pattern and wave exposure are some of the important factors that affect the sea cliff erosion by waves. In addition to wave action, ice wedging and rain-wash also contribute to sea cliff erosion. Cliff erosion rate can be determined by field surveys or by comparing aerial photos. Earlier, the principal source of sediment to any coast was rivers. But due to human interferences such as construction of dams, such sources get reduced and the principal source becomes erosion of adjacent shore and sea cliffs. Backshore erosion is a significant source where older coastal deposits, which contain a large fraction of sand, get eroded.

1.2.1.7 Sea level change

Due to global warming, the sea level is rising. Though the global sea level rise in the past one century is of the order of 10 to 20 cm, it is expected to increase exponentially in the coming century and is likely to be of the order of 1m over next 100 to 150 years. Sea-level rise causes the wave to progress/move to higher levels and thereby permitting larger waves to reach coast, through deepening of near-shore waters. Thus the sea level rise is expected to cause a significant retreat of the shoreline further hinterland. According to Unnikrishnan et al. (2006), the average rise in sea level in Indian coastline is 1.2 mm/year. The projected rise in sea level for the Indian coastaline in the next century is around 50cm and this in turn is expected to cause considerable retreat of the Indian shoreline.

1.2.2 Human-induced activities

Human-induced activities can cause shoreline changes. The civilization and instability of the coastline are very much linked. Man made structures in the coastline will alter/disturb the natural process. The hydrodynamic condition and sediment transport pattern may change due to man-made interventions leading to erosional or accretional processes. The existence of such instabilities, whether human induced or naturally occurring, whether sudden or spread over a long time, whether catastrophic or predictable do immensely affect the living stock of both humans and associated living forms. Some of the important human interventions that can cause beach morphological changes are listed below.

1.2.2.1 Coastal structures

Coastal structures are of different types. While some are intended for shore protection some others are built to facilitate harbours or other developmental activities. These structures by interfering with the natural coastal processes affect the beach morphology. A few of the coastal structures commonly seen along our coasts are listed below:

Seawalls: Seawalls are massive coastal structures constructed parallel to the shoreline mainly to protect the land adjoining the shoreline from wave action. The constructions of this massive structure alter natural processes of the beach and have some disadvantages also. Seawalls are less effective in preventing shoreline retreat. They do not protect the shore in front of them. Long-shore sediment transport will lead to toe erosion. The access to beach is affected. A major problem normally encountered in seawall protected coasts is the erosion observed in one end or both the ends of the seawall which usually is called as the “end effect”.

Groin: Groins are finger like structures usually perpendicular to the shoreline extending to the sea. Groins are classified as permeable or impermeable, high or low, fixed or adjustable. T-groin is the latest type which is intended to trap the sediment transported offshore. Groins are usually constructed in groups called groin fields. Their primary purpose is to trap littoral drift and thereby retard erosion of the shore by depositing the sediments in between the groin walls. But erosion is likely to occur on the downstream end or lee side. Transitional groins where the length of the groins tapers down towards the ends of the groin field are nowadays being built as an alternative to reduce erosion at the end.

Breakwater: Breakwaters are structures built offshore to dissipate the energy of incoming waves. Breakwaters may be either fixed or floating: the choice depends on normal water depth and tidal range. The disadvantages of breakwaters are its massive nature, negative impact on scenery and are not suitable for tourist spots. On rare occasions breakwaters reflect or diffract wave energy in destructive ways or lead to concentration of waves in local hot spots. Erosion problems and the scouring effects of the misdirected energy lead to the loss of beach/coastline and damage the structures.

Submerged breakwater is a breakwater with its top below the still water level. When waves strike this breakwater, part of the wave energy is reflected seaward and the remaining energy is largely dissipated in a breaker. This is further transmitted shoreward as a multiple crest system, or as a simple wave system. Permeable submerged breakwater is another variant of submerged breakwater which is partially permeable allowing the waves to pass through it.

1.2.2.2 Beach sand mining

Mining of beach material is done in many parts of the coast. This material is sometimes mined for the mineral it contains in some locations. In other places it is used for construction purposes. It is a direct loss, which can lead to erosion of the beach.

1.2.2.3 Reduction of sediment supply to the coastal zone

Reduction of sediment supply to the coastal waters can affect the beach morphology. In some areas the transport of sediment to the coast by rivers forms the major source of material to the coast. Dams constructed on these rivers not only trap the sediments but also reduce peak flood flows, thereby reducing the sediment supply to the coast. Dredging, leading to reclamation in many cases, is another activity which takes away sediment from the coastal environment.

1. 3 Methods for Estimation of Beach Morphological Changes

1.3.1 Beach profile measurements

The most accurate method of estimating shoreline change is by measurement of beach profiles by level and staff method. A shoreline can be compiled by interpolating between a series of discrete shore-normal beach profiles (Boak and Turner, 2005). Such beach profiling at regular time intervals can give accurate estimates of seasonal, annual and long-term shoreline change. However, the recourse to this method is constrained due to the exorbitant cost involved.

1.3.2 Aerial photography

Shoreline can be extracted from aerial photographs, preferably of scale 1:15,000 or more. Aerial photography is an old method and it provide good spatial coverage of the coast (Dollan et al., 1983). By definition, the ‘‘shoreline’ obtained from aerial photography is based on a visually discernible feature. This method has some drawbacks like distortion, which includes both radial and relief distortion, depending on the tilt and pitch of the aircraft, and scale variations caused by changes in altitude along a flight line (Anders and Byrnes, 1991). Modern technique of photogrammetry allows a digitally scanned pair of aerial photos to be converted into a three-dimensional digital terrain model and a georectified orthophoto (Hapke and Richmod, 2000; Overton and Fisher, 1996). The aerial photography is the most common data source for determining past shoreline positions. The extraction using this technique of mapping has to pass by two processes (Gibeaut et al., 2001). The first is to identify the shoreline and trace from the photograph, and the second is transfer on to a map with a common cartographic base.

1.3.3 Field survey using Global Positioning System (GPS)

Beach morphology can be mapped using GPS (Global Positioning System) and it is a more recent method of mapping shoreline. It is used to map shoreline positions as well as beach characteristics (berm, vegetation, scarp, etc.). While the hand held GPS can be used for the mapping, the accuracy of the mapping can be considerably enhanced by using Kinematic Differential GPS. The GPS survey can be effectively used to map the shoreline poison at regular time interval. The short-term as well as long-term shoreline change can be easily derived from the GPS surveyed data. This method is more accurate than aerial photography (Pajak and Leatherman, 2002).

1.3.4 Satellite remote sensing

Shoreline change can be monitored using satellite images. Images can be geo-referenced from base maps using GIS software and shoreline mapped. The advent of high resolution satellite sensors has increased the accuracy of this method in recent times. The advantage of this method is the high receptivity of the satellite data which enable the mapping of shoreline changes at a cheaper cost when compared to any other method.

1.3.5 Airborne Light Detection and Ranging Technology (LIDAR)

Airborne LIDAR surveys can be used for shoreline monitoring and it has the ability to cover hundreds of kilometers of coast in a relatively short period (Stockdon et al., 2002). This technique obtains highly accurate and detailed topographic measurements of the beach and hinterland. LIDAR can acquire data with vertical precision from 8 to 15cm and data-point less than 1m. From these data, a shoreline may be extracted for use in shoreline change analyses (Gibeaut et al., 2001). Tidal datum-based shorelines, such as MHW, can then be found by fitting a function to cross-shore profiles of LIDAR data. This data source is generally limited in its temporal and spatial availability because of high cost. The main advantage of LIDAR data is that it can be used to obtain a large coverage within a short period of time.

1.3.6 Video Imaging

Continuous monitoring of beach can be carried out by installing a video camera at a higher level overlooking the beach. By connecting the installed camera to a computer, the images at programmed intervals can be captured. By using appropriate image processing software, the shoreline or any other littoral environmental parameter can be derived. The advantage of this method is the facility to monitor shoreline changes in micro time scale.

1.3.7 Modelling

Basically there are two types of models viz. physical models and numerical models. The physical models are advantageous in correctly reproducing physical behavior. However, they have certain limitations like selecting the appropriate scale and the high cost. Hence numerical models have become more and more popular.

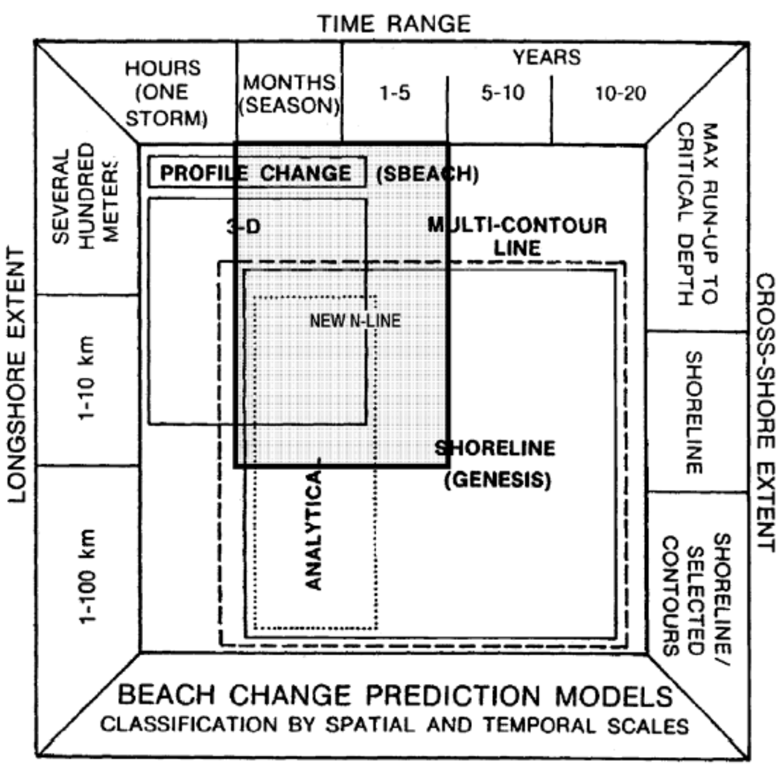


Fig. 1.5 New N-Line model and comparisons of beach change models in term of spatial and temporal scales (Hanson and Kraus, 1991a)

Numerical modelling is a powerful tool in understanding physical systems. It facilitates study of plan shape or profile changes particularly where both time and spatial scales are large. Numerical models are preferred not only due to the progressively increasing maturity of our knowledge in coastal processes, but also due to the advanced capacities of computer power for coastal morphological models (de Vriend, 1998; Komar, 1998). A classification of beach change prediction models by temporal and spatial scales as summarised by Hanson and Krauss (1991) is reproduced in Fig. 1.5

1.3.7.1 Advantages of numerical modeling

Numerical modelling is a powerful tool for study of physical systems. The models are able to examine systems to unravel the complexity of the multiple processes that may occur simultaneously. The beach and nearshore processes are very complex. Hence it is possible to understand and predict the behaviour of beach in response to hydrodynamic conditions by using numerical models. Beach erosion problem can be managed effectively by prudent use of numerical models. Designing of coastal structures and assessment of their environmental impact can be carried out effectively. The quantitative prediction of loss of beach material is one of the paramount tasks of coastal engineers. Numerical model helps to predict beach evolution quantitatively and the results can be used for many applications like estimating the quantity of beach nourishment, study of both long term and short term changes, etc.

Numerical model also has certain limitations. Most of the numerical models are based on large number of geological and oceanographic assumptions. Model assumptions should be examined collectively also in isolation. No numerical model can be an ideal representation of actual field conditions. There is a need for a theoretical re-examination of mathematical models used to predict any physical system (Robert Thieler et al., 2000).

1.4 Background of Present Investigation

The south-west coast of India is notable for severe erosion observed all along the coast during the southwest monsoon. While the beach is rebuilt in most of the sectors during the ensuing fair weather period, there are certain sectors of the coast which are not rebuilt fully or partially even during the fair weather period. Such sectors of the coast are said to be eroding on a long term basis. Muthalapozhi, north of Trivandrum, Valiazhikkal, north of Kayamkulam are examples of critically eroding coasts. In many of these cases man-made activities are the major contributing factors for the observed erosion.

Studies on beach morphological changes along the Indian coastline are mostly field based which are very laborious. Of late shoreline change studies using remote sensing techniques are also available. However, development of predictive capabilities, in spite of its immense application in coastal engineering and coastal zone management, has not received the due attention. Though there have been some isolated efforts towards development of predictive capability they were not successful. Even the commercially available models and free software that are available have not been calibrated/validated for our coast. The present investigation is undertaken in this context and aims at development of numerical model for prediction of long-term and short-term beach morphological changes using indigenously developed model instead of commercial available models which are very expensive.

1.5 Objectives

The investigation has been taken up with the following aim and objectives:

- Study the wave characteristics and beach processes of a few selected sites of SW coast of India
- Develop a profile change model to predict short-term profile changes due to episodic events
- Develop a shoreline change model to predict the long-term changes in the shoreline
- Apply these models for different coastal locations

1.6 Thesis structure

The thesis has been presented in 7 chapters including this introductory chapter.

The exhaustive literature review carried out is presented in the Chapter 2. The beach morphological change models are classified into two based on the time scale criterion: profile change (short-term) and shoreline change (long-term) models.

Chapter 3 describes the development of numerical models to predict short-term profile changes and long-term shoreline changes. To predict long-term shoreline change, a numerical model based on the approach of Kraus and Harikai (1983) is developed. Beach profile change model has been developed based on the concepts of Larson and Krauss (1989).

Chapter 4 describes the field measurements and collation of secondary data carried out in connection with the investigation for different coastal locations of the SW coast of India. The locations selected for study are Adimalathura, Valiathura, Muthalapozhi, Kayamkulam, Trikkunnapuzha, Mararikkulam and Calicut. The data on waves, beach profiles, littoral environmental characteristics and sediment characteristics have provided insight into beach morphodynamics at these locations in response to different forcing factors.

The processes of calibration/validation of the models utilising data from different locations are covered in Chapter 5. The chapter also deals with the performance assessment of the models by statistical and graphical methods in addition to comparison with a commercial model.

Chapter 6 presents the results of applications of the models for different coastal locations of the southwest coast of India. The Shoreline Change model is used to predict shoreline changes over a five year period at two locations of the south-west coast of India. The Profile Change model is used to predict profile changes in a couple of cases of episodic events.

Chapter 7 presents the summary and conclusions of the work. Recommendations for future work also are presented.

02

Review of Literature

Chapter 2
REVIEW OF LITERATURE

2.1 Introduction

The physical processes associated with sediment transport around the breaker zone are highly complex. The wave induced spatially varying currents and highly irregular flows make this environment extremely dynamic. Waves breaking near the coast mobilize the sediments around the breaker point and the currents generated by the waves transport the sediments along and across the coast. While steep waves due to episodic events lead to offshore transport and consequent erosion, long period waves cause onshore transport and beach build-up. A review of the available literature on the beach morphological changes and its modelling is presented in this Chapter. For convenience in presentation, the reviewed literature is grouped under the following sub-sections:

(a) Sediment transport computation

(b) Bar/berm profile criteria

(c) Bar and trough formation

(d) Equilibrium beach profile

(e) Experimental studies

(f) Numerical model studies

2.2 Sediment Transport Computation

Bagnold (1963, 1966) developed formulae for calculating cross-shore sediment transport rate based on a wave energy approach, distinguishing between bed load and suspended load. Bailard and Inman (1981) and Bailard (1981) used Bagnold's (1963) sediment transport relationships to develop a model for transport over a plain sloping beach. They determined the influence of the longshore current on the equilibrium profile slope. The beach profile was flattened in the area of the maximum longshore current and the slope increased with sand fall velocity and wave period. Sawaragi and Deguchi (1981) studied cross–shore transport and beach profile change in a small wave tank and distinguished three transport rate distributions. They developed an expression for the time variation of the maximum transport rate and discussed the relation between bed and suspended load.

Watanabe et al. (1981) calculated net cross-shore transport rate from the mass conservation equation and measured profiles in the laboratory, arriving at transport relationship. They introduced a critical '*shields stress'* below which no transport occur and assumed a linear dependence of the transport rate on the shields parameter. Shibayama (1984) investigated the role of vortices in sediment transport and derived transport formulae for bed and suspended load based on shields parameter. They observed that generation of vortices was not confined to plunging breakers but could occur under spilling breakers as well. Sunamura (1984) derived a formula to determine the cross-shore transport rate in the swash zone taken as an average over 1 hour. It was related to the near bottom orbital velocity and the transport equation predicted the net direction of sand movement. Nairn (1988) developed a cross-shore sediment transport model involving random wave transformation. Two different methods of wave height transformation were investigated, one using the root mean square wave height as a representative measure in the wave height calculations and the other a complete transfer of the probability density function based on the response of individual wave components.

2.3 Criteria for Delineating Bar and Berm Profile

Scott (1954) derived the wave steepness criterion for distinguishing between ordinary and storm profiles, based on his laboratory experiments. He found that the rate of profile change was greater if the initial profile was farther from equilibrium shape and he recognized the importance of the wave induced turbulence for promoting bar formation. Some analysis of sediment stratification and packing along the profile was carried out. Kemp (1961) introduced the concept of '*phase difference'* referring to the relation between time of up rush and wave period. He found the transition from a step (ordinary) to a bar (storm) profile to be a function of the phase difference and occur if the time of up rush was equal to the wave period.

Iwagaki and Noda (1963) derived graphically a criterion for predicting the appearance of bars based on two non-dimensional parameters − deepwater wave steepness and ratio between deepwater wave height and median grain size. They discuss the change in character of breaking waves due to profile evolution in time. The potential importance of suspended load was recognized and represented through the grain size which is emerging as a significant factor in beach profile change. Nayak (1970, 1971) performed small-scale laboratory experiments to investigate the shape of equilibrium beach profiles and their reflection characteristics. He developed a criterion for the generation of longshore bars that is similar to that of Iwagaki and Noda (1963) but included the specific gravity of the material. The slope at the still water level for the equilibrium profile was controlled more by specific gravity than grain size. Furthermore, he found that the slope decreased as the wave steepness at the beach toe or the dimensionless fall speed (wave height divided by fall speed and period) increased. The dimensionless fall speed was also found to be an important parameter for determining the reflection coefficient of the beach.

Dean (1973) assumed suspended load to be the dominant mode of transport in most surf zones and derived on physical grounds the dimensionless fall speed as governing parameter. Sand grains suspended by the breaking waves would be transported onshore or offshore depending on the relation between the fall speed of the grains and the wave period. A criterion for predicting the cross-shore transport direction, to be onshore or offshore, based on the non-dimensional quantities of deepwater wave steepness and fall speed divided by wave period and acceleration due to gravity was proposed. The criterion of transport direction was also used for predicting profile response (normal or storm profile). Sunamura and Horikawa (1974) classified beach profile shapes into three categories distinguished by the parameters of wave steepness, beach slope and grain size divided by wavelength. The criterion was applied to both laboratory and field data, only requiring a different value of an empirical coefficient to obtain division between the shapes.

Hattori and Kawamata (1981) developed a criterion for predicting the direction of cross-shore sediment transport similar to Dean (1973) by including the beach slope also as a parameter. The criterion was derived from the balance between gravitational and turbulent forces keeping the grains in suspension. Rushu and Liang (1986) proposed criteria for distinguishing between beach erosion and accretion involving a number of dimensionless quantities. A new parameter consisting of the bottom friction coefficient, critical velocity for incipient motion of the grains and the fall speed of the grains was introduced. Seymour and Castel (1988) evaluated studies on prediction of transport direction. Of the models studied, the one proposed by Hattori and Kawamata (1981) proved to have the highest predictive capability in applying to three different field sites. Most models were not considered successful at predicting transport direction.

2.4 Bar and Trough Formation

Evan (1940) studied bars and troughs along the eastern shore of Lake Michigan and concluded that these features are the result of plunging breakers. He regarded the bar and trough to form a unit, with the trough always located shoreward of the bar. He found that if slope was mild as indicated by the appearance of several breaker points, a series of bars and troughs would develop. Changes in wave conditions and water level were also found to influence the bar formation resulting a change in bar shape and migration of the bar seaward or shoreward. A decreasing water level would cause the innermost bar to migrate onshore and take the form of a sub-aqueous dune, whereas an increase in water level would allow a new bar system to develop inshore. The most seaward bars would then become inactive.

Keulegan (1945) experimentally obtained simple relations for predicting the depth-to-bar crest and the trough depth. He found that the ratio between trough and crest depths is approximately constant and independent of wave steepness. Keulegan (1948) through further laboratory experiments made significant contributions to the basic understanding of the physics of beach profile change. The objectives of his study were to determine the shape and other characteristics of the bar and the processes through which they were moulded by the incident waves. He recognized the surf zone as the most active area of beach profile changes and the breaking waves as cause of bar formation. The location of the maximum sand transport, measured by traps, was found to be close to the breaking point, and the transport rate showed a good correlation with the wave height envelope. He noted three distinct regions along the profile where the transport properties were different from a morphologic perspective. A gentler initial beach slope implied a longer time before the equilibrium profile was attained for fixed wave conditions. For constant wave steepness, an increase in wave height moved the bar seaward whereas for a constant wave height, an increase in wave steepness (decrease in wave period) moved the bar shoreward. He noted that two bars developed in the laboratory experiments were shorter and more peaked than the bars in the field and attributed this difference to the variability in the wave climate on natural beaches.

King and Williams (1949) distinguished between bars generated on non-tidal beaches and bars occurring on beaches with a marked tidal variation. They assumed that non-breaking waves moved sand seaward. Field observations from the Mediterranean also confirmed the main ideas of this conceptualization. In laboratory experiments, the cross-shore transport rates were measured with traps placed at different points, showing a maximum transport rate located around the break point. Further to this a term “breakpoint bar” was introduced to describe the bar formation, whereas berm formations were denoted as “swash bars”. The slope of the berm was related to the wavelength, where a longer wave period produced a gentler slope. They hypothesized that ridge and runnel systems were not formed as a result of breaking waves but were the result of swash processes.

Shepard (1950) made profile surveys along the pier at Scripps Institution of Oceanography, La Jolla, California, in 1937 and 1938, and discussed the origin of troughs and suggested that the combination of plunging breakers and longshore currents was the primary cause. He also showed that the trough and crest depths depend on breaker height. Large bars formed seaward of the plunge point of the larger breakers, and the ratio of the trough to crest depth were smaller than those found by Keulegan (1948) through laboratory experiments. He also observed the time scale of beach profile response to the incident wave climate and concluded that the profile change was better related to the existing wave height than to the greatest wave height from the preceding 5 days.

Watts (1954) studied the effect of varying wave period and water level, on the beach profile. A varying wave period reduced the bar and trough system as compared to waves of constant period, but only slightly affected beach slope in the foreshore and offshore. The influence of water level variation was small, producing essentially the same foreshore and offshore slopes. However, the active profile translated landward for the tidal variation, allowing the waves to attack at a higher level and thus activating a larger portion of the profile.

Mckee and Sterrett (1961) investigated cross-stratification patterns in bars by spreading layers of magnetite over the sand. Zenkovich (1967) presented a summary of a number of theories suggested by various authors for the formation of bars. Mothersill (1970) found evidence through grain size analysis that longshore bars are formed by plunging waves and rip currents. Sediment samples taken in troughs were coarser, having the properties of winnowed residue, whereas samples taken from bars were finer grained, having the characteristics of sediments that had been winnowed out and then re-deposited. Dyhr-Nielsen and Sorensen (1971) proposed that longshore bars were formed from breaking waves, which generated secondary currents directed toward the breaker line. On a tidal beach with a continuously moving break point, a distinct bar would not form unless severe wave conditions prevailed. Saylor and Hands (1971) studied the characteristics of longshore bars in the Great Lakes. The distance between bars increased nonlinearly with distance from the shoreline, whereas the depth to crest increased linearly. A rise in water level produced onshore movement of the bars. Carter et al. (1973) suggested that longshore bars could be generated by standing waves and associated reversal of the mass transport in the boundary layer, causing sand to accumulate at either nodes or antinodes of the wave. In order for flow reversal to occur, significant reflection had to be present.

Exon (1975) investigated bar fields in the western Baltic Sea, which were extremely regular due to evenly distributed wave energy alongshore. He noted that the presence of engineering structures reduced the size of the bar field. Greenwood and Davidson-Arnott (1975) performed field studies of a bar system in Kouchibouguac Bay, Canada and identified conditions for bar development as gentle offshore slope, small tidal range, availability of material and absence of long period swell. They distinguished between the inner and outer bar system and described in detail the characteristics of these features. The break point of the waves was located on the seaward side of the bar in most cases and not on the crest. Hands (1976) observed in field studies at Lake Michigan that plunging breakers were not essential for bar formation. A number of geometric bar properties were characterized in time and space for the field data.

Greenwood and Davidson-Arnott (1979) presented a classification of wave-formed bars and a review of proposed mechanisms for bar formation. Greenwood and Mittler (1979) found support in the studies of sedimentary structures of the bar system being in dynamic equilibrium from sediment movement in two opposite directions. An asymmetric wave field moved the sand landward and rip currents moved the material seaward.

Hunter et al. (1979) studied nearshore bars attached to the shoreline and migrated alongshore, in the Oregon coast. Rip currents were occasionally observed shoreward of the bar during field investigations. Bowen (1980) investigated bar formation by standing waves and presented analytical solutions for the standing waves on plane sloping beaches. He also derived slopes for beach profiles assuming simple flow variations. Davidson-Arnott and Pember (1980) compared bar systems at two locations in southern Georgian Bay, the Great Lakes and found them to be very similar despite large differences in fetch length. The similarity was attributed to the same type of breaking conditions prevailing, with spilling breakers occurring at multiple break points giving rise to multiple bar formation. Dolan and Dean (1984) investigated the origin of the longshore bar systems in Chesapeake Bay and concluded that multiple breaking was most likely the cause. Other possible mechanisms discussed were standing waves, edge waves, secondary waves and tidal current, but none of these could satisfactorily explain the formations. Sunamura and Takeda (1984) quantified onshore migration of bars from a 2-year series of profile data from a beach in Japan. They derived a criterion to determine the occurrence of onshore bar movement and an equation to estimate the migration speed. Onshore transport typically took place in the form of bed load shoreward in a hydraulic bore. Takeda (1984) studied the behaviour of beaches during accretion conditions. Based on field investigations from Naka Beach, Japan, he derived predictive relationships for determining average speed of onshore bar migration and berm formation, if onshore movement of bars occurs. He pointed out the rapid formation of berms in the field where the build-up may be completed in one or two days.

Birkemeier (1985a) analyzed the time scale of beach profile change from a data set comprising three and a half years of profile surveying at Duck, North Carolina. He found large bar movement with little change in the depth to crest. If low wave conditions prevail for a considerable time, a bar-less profile developed. Mason et al. (1985) summarized the field experiment conducted at Duck, North Carolina, where nearshore bar system was closely monitored during storm. Bar dynamics showed a clear dependence on wave height, the bar becoming better developed and migrating offshore as the wave height increased.

Mei (1985) mathematically analyzed resonant reflection from nearshore bars that can enhance the possibility for standing waves to generate bars. Sallenger et al. (1985) observed the rapid response of a natural beach profile at Duck, North Carolina, to changing wave conditions. Both offshore and onshore bar movement occurred at much higher speed than expected, and the ratio between the bar and trough depth was approximately constant during offshore bar movement but varied during onshore movement. Since bars appeared to be located well within the surf zone, they concluded that wave breaking was directly responsible for bar movement.

Thomas and Baba (1986) studied berm development produced by onshore migration of bars for a beach at Valiathura, southwest coast of India, and related the condition for onshore/offshore movement to wave steepness. Wright et al. (1986) concluded from field measurements that bar-trough morphology was favoured by moderate breaker heights combined with small tidal ranges. Short period waves were the main cause of sediment suspension in the surf zone, although the long period waves were believed to be important in the overall net drift pattern. Seymour (1987) summarized the results from the ‘Nearshore Sediment Transport Study’, a six-year program in which nearshore sediment transport equations were investigated. He pointed out the importance of bar formation for protecting the foreshore against wave action and the resulting rapid offshore movement of the bar on a beach exposed to storm waves. Takeda and Sunamura (1987) found from field studies in Japan the great influence of bar formations on the sub-aerial response of beaches with fine sand.

Guillén and Palanques (1993) investigated longshore bar and trough systems in a microtidal, storm-wave dominated coast. A descriptive model relating the shoreline displacement with the cross-shore migration and the longshore growth of the bar and trough systems is proposed.

2.5 Equilibrium Beach Profile

Brunn (1954) developed a predictive equation for the equilibrium beach profile by studying beaches along the Danish North Sea coast and the California coast. The equilibrium shape followed a power curve with distance offshore, with the power evaluated as 2/3. Rector (1954) investigated the shape of the of the equilibrium beach profile in a laboratory study. Equations were developed for profile shapes in two sections separated at the base of the foreshore. Coefficients in the equilibrium profile equations were a function of deep-water wave steepness and grain size normalized by the deep-water wavelength. An empirical relationship was derived for determining the maximum depth of the profile adjustment as a function of the two parameters. These parameters were also used to predict net sand transport direction. Brunn (1962) applied his empirical equation for an equilibrium beach profile to estimate the amount of erosion occurring along the Florida coast as a result of long-term sea level rise. Eagleson et al. (1963) studied the equilibrium profiles in the region seaward of the influence of breaking waves. They pointed out the importance of the of bed load for determining equilibrium conditions and used equations for particle stability to establish a classification of beach profile shapes. Kamphuis and Bridgeman (1975) performed wave tank experiments to evaluate the performance of artificial beach nourishment. They concluded that the inshore equilibrium profile was independent of the initial slope and is a function of beach material and wave climate. However, the time elapsed before equilibrium was attained, as well as the bar height, depended upon the initial slope. Van Hijum (1975, 1977) and Van Hijum and Pilarczyk (1982) investigated equilibrium beach profiles of gravel beaches in laboratory tests and derived empirical relationships for geometric properties of profiles. The net cross-shore sand transport rate was calculated from the mass conservation equation, and a criterion for the formation of bar/step profiles was proposed for incident waves approaching at an angle to the shoreline.

Dean (1976) discussed equilibrium profiles in the context of energy dissipation of wave breaking. Various causes of beach profile erosion were identified and analyzed from the point of view of the equilibrium concept. Dean (1977) analyzed beach profiles from the United States Atlantic and gulf coasts and arrived at a 2/3 power law as the optimal function to describe the profile shape, as previously suggested by

Brunn (1954). Dean (1977) proposed a physical explanation for the power shape assuming that the profile was in equilibrium if the energy dissipation per unit water volume from wave breaking was uniform across shore. Gourlay (1981) emphasized the significance of the dimensionless fall speed in describing equilibrium profile shape, relative surf zone width and relative up rush time.

Greenwood and Mittler (1984) inferred the volume flux of sediment over a bar by means of rods driven into the bed on which a freely moving fitting was mounted to indicate changes in bed elevation. Their study indicated an energetic approach to be reasonable for predicting equilibrium slopes involving the breaking wave height, wave period and grain size. Equations were developed and applied for laboratory and field conditions.

Zhi-Jun Dai et al. (2007) developed a new equation to predict the change in beach profile for sections above the water level and the adjacent nearshore portions. Moreover, fractal analysis is applied to predict types of equilibrium beach profile for the first time using the field data collected from Liao Zuikou and Nanwan beaches, South China.

Walton and Dean (2007) provided the necessary guidance for equilibrium beach profiles, although limited knowledge exists regarding the spatial and temporal variability of the equilibrium beach profile. Equilibrium beach profile theory is utilized to assess the spatial and temporal variability of beach profiles along the Florida Panhandle Coast to provide added empirical guidance on this subject.

2.6 Experimental Studies

Bagnold (1940) studied beach profile evolution in small-scale laboratory experiments using rather coarse material (0.5 – 0.7 mm), resulting in accretion of profiles with berm build-up. He found that the foreshore slope was independent of the wave height and mainly a function of grain size. However, the equilibrium height of the berm was linearly related to wave height. The effect of seawall on the beach profile was investigated by allowing waves to reach the end of the tank. In other experiments, a varying wave height was used. Saville (1957) was the first to employ a large wave tank capable of reproducing near-prototype wave and beach condition, and to study the equilibrium beach profiles and model scale effects. Waves with very low steepness were found to produce storm profiles, contrary to results from small–scale experiments. Comparisons were made between the large wave tank studies and small-scale experiments, but no reliable relationship between prototype and model was obtained.

Hattori and Kawakawa (1979) investigated the behaviour of beach profiles in front of a seawall by means of laboratory experiments. Their conclusion was that material eroded during a storm returned to the seawall during low wave conditions to form a new beach. Hughes and Chiu (1981) studied dune recession by means of small scale movable bed model experiments. The quantum of dune erosion was found by shifting the barred profile horizontally until eroded volume agreed with deposited volume. Geometric properties of the equilibrium bar profile were expressed in terms of dimensionless fall speed. Vellinga (1982, 1986) presented results of dune erosion from large wave tank studies and discussed scaling laws for movable-bed experiments. The dimensionless fall speed proved to provide a reasonable scaling parameter in movable-bed studies. He also emphasized the dependence of the sediment concentration on wave breaking. Kajima et al. (1983a, b) discussed beach profile evolution using data obtained in a large wave tank with waves of prototype size. Beach profile shapes and distributions of the net cross-shore transport rates were classified. A model of beach profile change was proposed based on a schematized transport rate distribution, which decayed exponentially with time. Seelig (1983) analyzed large wave tank data and developed a simple prediction method to estimate beach volume change above the still-water level.

Shimizu et al. (1985) analyzed data obtained with a large wave tank to investigate the characteristics of the cross-shore transport rate. The transport rate distribution was modeled by superimposing three separate curves representing the transport rate in the foreshore, surf zone and offshore zone. Kriebel et al. (1986) studied beach rebuilding after storm events both during laboratory and field conditions, noting the rapid process of berm formation. They could not find evidence for break-point bars moving offshore and welding onto the beach face during the recovery processes; instead, the berm was built from material originating farther inshore. Dally (1987) tested the possibility of generating bars by long period waves (surf beat) in a small wave tank, but he found little evidence for this mechanism. Instead, breaking waves in combination with undertow proved to be the cause of bar formation in the cases studied, irrespective of whether spilling or plunging breakers prevailed. Hallermeier (1987) stressed the importance of large wave tank experiments for providing valuable information of the beach response to storm conditions. He compared results from a large wave tank experiments with a natural erosion episode at Bethany Beach, Delaware and found similar erosive geometry. Beach recovery following the 1985 Hurricane Elena was discussed by Kriebel (1987), who concluded that the presence of a seawall did not significantly affect the process of beach recovery at the site. Kriebel, et al. (1987) performed laboratory experiments using a small wave tank and beach shapes designed with the dimensionless fall speed as the scaling parameter. They found marked differences in profile response depending on the initial shape being planar or equilibrium profile type. An initially plane beach produced a more pronounced bar and steeper offshore slopes. The fall speed parameter and the deep-water wave steepness were used to distinguish erosion/accretion profiles. Mimura et.al. (1987) performed laboratory experiments with a small wave tank to investigate the effect of irregular waves on the beach profile. They addressed the question of which representative wave height to use for describing profile response. The mean wave height represented macroscopic beach changes such as bar and berm development most satisfactorily, whereas microscopic phenomena such as threshold of transport and ripple formation were better described by use of significant wave height. Sunamura and Maruyama (1987) estimated migration speeds for seaward moving bars as given by large wave tank experiments using monochromatic waves. The bars were generated by breaking waves and located somewhat shoreward of the break point. They opined that spilling breakers could also form bars, although the approach to equilibrium was much slower than for bars formed by plunging breakers.

Uliczka and Dette (1987) and Dette and Uliczka (1987a, b) investigated beach profile evolution generated in large wave tank conditions. The tests were carried out with both monochromatic and irregular waves for a dune like foreshore both with and without a surf zone. For the case of a beach without a foreshore, monochromatic waves produced a bar, whereas irregular waves (represented by significant wave height and peak spectral period) did not. However, the incident wave energy was different in those two cases. Sediment concentration and cross-shore velocity were measured through the water column at selected points across the profile. Kraus and Larson (1988) described the large wave tank experiments on beach profile change performed by Saville (1957) and a similar experiment performed by the US Army Corps of Engineers, giving a list of all the data.

Teisson et al (1993) investigated the mechanisms of modeling cohesive sediment transport processes through laboratory experiments. Larson and Sunamura (1993) studied flow structure and sediment movement over the beach step, commonly present at the foot of the beach face in a laboratory wave tank. It was observed that most of the coarse sediments were eventually deposited either on the step or on the lower part of the beach face, and with only a small amount deposited farther up on the beach face.

Wang et al (2002) carried out experiments by generating typical sea conditions replaced by large-scale sediment transport to investigate its cross shore distribution pattern. Sunamura (2006) conducted a laboratory experiment using a two-dimensional wave tank designed to investigate the mechanism of erosion at a cliff base armed with rock fragments. The experiment was performed under constant wave conditions by systematically changing the amount of beach sand present at the foot of cliffs having the same slope and strength. The analysis of results indicated that the effect of the abrasion doubled when the cliff/beach junction was located above Still Water Level (SWL) as compared to that below SWL. The force of the sediment-laden water masses was found to be proportional to the square of the bore speed immediately in front of the cliff face. The factor of proportionality was related to the quantity of beach sand entrapped in the turbulent fluid. Turker and Kabdash (2006) conducted flume studies to study the effect of transport parameter on cross-shore sediment transport and arrived at an empirical relation to calculate the dislocation parameter from the wave height, wave period and fall velocity.

2.7 Numerical Models

Numerical models for beach morphological change prediction are broadly classified as Profile Change Models and Shoreline Change Models. Profile change models are used to model beach profile changes in response to episodic events which prevail for a short duration of one or two weeks while shoreline change model simulates the long term shoreline change in response to alongshore transport of sediment. The shoreline change models are used to predict the long-term change (time scales of several months) of the shoreline as a consequence of alteration in the wave climate and/or the rate of sediment transport along the shore introduced by the coastal constructions like groins, breakwaters, harbours and offshore dredging.

2.7.1 Profile change models

The study of beach profile change in the broad sense covers near-shore processes in spatial and temporal scales. Swart (1975, 1976) studied cross-shore sediment transport properties as characteristic shapes of beach profiles. A cross-shore sediment transport equation was proposed where the rate was proportional to a geometrically defined deviation from the equilibrium profile shape. A numerical model was proposed based on the derived empirical relationships and applied to a beach fill case. Wang et al. (1975) developed a computer intensive three-dimensional numerical model of beach change, assuming that cross-shore transport occurred largely in suspension. The transport rate was related to the energy dissipation across shore. Felder (1978) and Felder and Fisher (1980) divided the beach profile into various regions with specific transport relationships and developed a numerical model to stimulate bar response to wave action. In the surf zone, the transport rate depended on the velocity of a solitary wave. Nilsson (1979) assumed bars to be formed by partially reflected Stokes wave groups and developed a numerical model based on this mechanism. Sediment transport rates were calculated from the bottom stress distribution and an offshore-directed mean current was superimposed on the velocity field generated by the standing waves.

Dally (1980) and Dally and Dean (1984) developed numerical model of profile change based on the assumption that suspended transport is dominant in the surf zone. The cross-shore broken wave height distribution determined by the numerical model supplied the driving mechanism for profile change. An exponentially shaped profile was assumed for the sediment concentration through the water column. Shibayama and Horikawa (1980a, b) proposed sediment transport equation for bed load and suspended load based on the Shields parameter. A numerical beach profile model was applied using these equations, which worked well in the offshore region but failed to describe profile change in the surf zone. Davidson-Arnott (1981) developed a numerical model to simulate multiple longshore bar formation. The model qualitatively produced offshore bar movement, but no comparison with measurements was made.

Moore (1982) developed a numerical model to predict beach profile change produced by breaking waves. He assumed the transport rate to be proportional to the energy dissipation from breaking waves per unit water volume above an equilibrium value. An equation was given which related this equilibrium energy dissipation to grain size. The beach profile calculated with the model approached an equilibrium shape in accordance with the observations of Brunn, if exposed to the same wave conditions for a sufficiently long time.

Kriebel (1982, 1986) and Kriebel and Dean (1984, 1985) developed a numerical model to predict beach and dune erosion using the same transport relationship as Moore (1982). The amount of erosion was determined primarily by water level variation, and breaking wave height was entered only to determine the width of the surf zone. The model was verified both against data from large wave tank experiments and from natural beaches taken before and after Hurricane Eloise. The model was applied to predict erosion rates at Ocean City, Maryland, caused by storm activity and sea level rise (Kriebel and Dean 1985). Watanabe (1982, 1985) introduced a cross-shore transport rate, which was a function of the Shields parameter to the 3/2 power in a three-dimensional model of beach change. The model simulated the effects of both waves and nearshore currents on the beach profile. The transport direction was obtained from an empirical criterion.

Sunamura (1983) developed a simple numerical model of beach morphological change caused by short-term events and described both erosion and accretion phases of a beach in the field. Exponential response functions were used to calculate the magnitude of beach morphological change. Balsillie (1984) related longshore bar formation to breaking waves from field data and developed a numerical model to predict profile recession produced by storm and hurricane activity. In a numerical model developed by Stive and Battjes (1985), offshore sand transport was assumed to occur through the undertow and as bed load only. They verified the model against the laboratory measurements of profile evolution produced by random waves. Stive (1987) extended the model to include effects of asymmetry in the velocity field from the waves. Boczar-Karakiewicz and Davidson-Arnott (1987) proposed the nonlinear interaction between shallow-water waves as a possible cause of bar formation. A mathematical model was developed to predict the generation of bars and the model results were compared with field data. In the model, the mass transport velocities associated with the primary wave and the second harmonic are used to calculate net sediment flux across a two-dimensional profile. Model predictions of bar number and spacing, starting with an initially planar slope, correlate well with the field measurements, for the two sets of wave conditions and mean slopes. In addition the model is able to predict profile changes reflecting seasonal changes in wave climate.

Broker Hedegaard et al.(1991) and Skou et al.(1991) developed deterministic model for predicting morphological evolution of a coastal profile. The model established the interaction between hydrodynamic conditions and bed-level evolution. The cross-shore profile changes are described by solution of the bottom sediment continuity equation based on the sediment transport rates.

Zheng and Dean (1997) developed a model called CROSS based on equilibrium concepts and is calibrated using wave flume tests on profile evolution. Miller and Dean (2004) developed a numerical model to predict the beach profile changes. In this model several possible forms of rate parameters incorporating local wave and sediment properties were considered and evaluated. The model was used to predict the changes for some field results and predictions were successful.

In summary, all available models regarding beach profiles are reviewed under different categories. The models differ in their estimation of cross-shore transport rate. The degree of modification varies from model to model. The equilibrium profile models are site specific and have to be recalibrated while applying to different field conditions. For deterministic models, the computational time is relatively long and unsuitable for long-term simulations. A number of models have been developed but they need further calibration and validation.

2.7.2 Shoreline change models

Though studies on shoreline change by alongshore transport of sediments are plenty, it has not attracted as much attention as the profile changes due to cross-shore transport. This may be due to the fact that short-term events cause erosion in most cases which attracts more attention, whereas the long-term changes are very slow. Many of the existing models for predicting the shoreline variation assume that the sediment balance associated with the shoreline evolution process could be described in terms of the variations in the shoreline geometry i.e. the variations in the zero contour line. One-line model assumptions are rarely satisfied by short time scale variations in the shoreline and evidently models based on these assumptions cannot predict short-term beach profile variations. The one-line concept rests on a common observation that the beach profile maintains an average shape that is characteristic of the particular coast, apart from times of extreme change as produced by storms. A second geometrical-type assumption is that sand is transported alongshore between two well-defined limiting elevations on the profile.

All the existing investigations on modelling of shoreline changes, except that of Bakker (1968), are based on one-line simulation which essentially equates the change of beach volume to the material transported out of a given area, but differ from one another in the manner in which the longshore material transport is computed. But in most of the models, only the longshore drift due to longshore current is considered and the transport of beach material by wind, loss of silt to offshore and loss of beach material due to rip currents (during storms) are not included, since generally they are not significant and very little information is available on them. In a situation where movement of beach material in an offshore or onshore direction is important, Bakker (1968) described a more sophisticated model where the beach is represented by two contours which in general are not parallel. This two-line theory allows for onshore-offshore sediment transport and change in beach slope. The model has the disadvantage that it is very difficult to quantify such effects and hence to realise the full potential of this method.

Using a simple one line model, Price et al. (1972) attempted to predict the changes in plan shape of a beach following the construction of groins. They used the Scripps equation as modified by Komar (1998) to quantify the longshore sediment transport and applied to simulate the shore changes during an experiment conducted in the wave basin. They reported good agreement between prediction and measurement. Sasaki (1973) developed two simulation models on nearshore environments − the first model predicts shoreline deformation behind a detached breakwater placed parallel to the shoreline and the second simulates current in the nearshore zone under the influence of an arbitrary bottom topography. These models were tested, the former in laboratory and the latter in the field.

LeMhaute and Soldate (1977) presented a critical review of literature on mathematical modelling of shoreline changes with emphasis on long-term evolution rather than seasonal or episodic. LeMhaute and Soldate (1978, 1980) developed mathematical model for long-term shoreline evolution combining the effects of variation in sea level, wave refraction, wave diffraction, loss of sand by density currents, rip currents and bluff erosion and berm accretion as well as the effects of manmade structures such as groins or navigational structures and beach nourishment. A computer program was developed with facility to permit modifications as the state of art progresses. The program was applied to a test case of Holland Harbour, Michigan and model has reasonably simulated the shoreline change at Holland Harbour, Michigan.

Perlin and Dean (1978) described three numerical models − (a) one-line implicit model, (b) one-line explicit model and (c) two-line explicit model − representing shoreline response and applied to a number of problems in coastal engineering. Simplified refraction and diffraction schemes are incorporated into the models. The one-line implicit model is applied to predict the shoreline response in the vicinity of an offshore breakwater at Channel Islands Harbour, California where sediment is accumulated and dredged periodically. Perlin (1979) presented a numerical model to predict beach plan forms in the lee of detached offshore breakwaters. Non-dimensional theoretical situations were also investigated. Using the one-line model, Ozasa and Brampton (1980) attempted to simulate the evolution of a beach backed by sea walls; the shoreline changes were measured during a physical model study of a bay in Japan. The equation for longshore sediment transport used by them takes into account the sediment transport due to oblique breaking of waves and that due to the longshore wave height gradients. Mizura and Shiraishi (1981) investigated the effects of the presence of an airport to be constructed offshore of Sennan coast on coastal processes using hydraulic model tests.

Kraus and Harikari (1983) using the one-line model assumptions, numerically simulated the long term shoreline changes on sandy beaches adjacent to Oarai Harbour, Japan. They first calibrated the model using available data over a period of 7½ months and then simulated the shoreline change over three years to verify the model's predictive capability. The calculation procedure used for the estimation of the breaking wave height and angle along the beach under combined diffraction and refraction was also verified with field measurements. Perlin and Dean (1983) developed a one-line numerical model to predict bathymetric changes in the vicinity of coastal structures. The wave field transformation includes shoaling, refraction and diffraction. The model is capable of simulating one or more shore-perpendicular structures, movement of offshore disposal mounds and beach fill. The length of structure, shoreline geometry, sediment properties, equilibrium beach profile, etc. is user specified along with wave climate.

The general formulation of shoreline evolution of LeMehaute and Soldate (1980) includes most of the important agents those transport material along the coast. But in the case study reported, they considered only the longshore transport due to the littoral currents, which was assumed to depend solely on the longshore energy flux P_1. The longshore transport Q in cubic metres per day is expressed by the empirical relationship. Most studies of shoreline simulation are based on the one-dimensional sediment balance equation which is referred to as one-line model equation. In general one-line model equation is nonlinear and environmental conditions are complex due to variations in the wave field and consequent longshore littoral transport. The simulation of shoreline evolution requires numerical methods for its solution.

Larson et al. (1997) intensively surveyed 25 previous analytical models for simulating the evolution of a sand beach. They developed a general solution using Laplace transformation techniques for beach evolution with and without coastal structures. Depending on the boundary conditions, these solutions cover a number of cases ranging from beach fill of arbitrary initial shape, sand mining, river discharges, groins and jetties, detached breakwaters and seawalls.

Leontyev (1997) proposed a one-line model to simulate the short-term shoreline changes in the vicinity of cross-shore structures during a storm event. The Hanson and Larson (1999) model was applied to Duck, North Carolina, USA, to predict the shoreline change for a time period of 11 years (1981-1991). To simulate the impact of structures, a simple formulation of diffraction in the Steetzel et al. (2000) model considering the modifications of the wave directions and wave height based on formula from Kamphuis (1992) has been developed.

Ravens and Sitanggang (2002) developed the shoreline change model (GENESIS) of the Galveston shoreline. The model performance was verified in the presence of structure like T-groins, off-shore breakwater. Statistical and stochastic models using a moment equation for shoreline evolutions have been developed by Dong and Chen (1999) and Spivack and Reeve (2002). Ashton et al. (2003) developed a numerical model to examine the shoreline instability subjected to high angle waves along the North Carolina Outer Banks, USA.

GENESIS (Hanson and Kraus, 1991a), UNIBEST (Delft Hydraulics, 1993), LITPACK (DHI, 2001), ONELINE (Dabees, 2000), SAND94 (Szmytkiewicz et al., 2000) are all based on one- line theory and are all widely recognized numerical modelling systems used in coastal engineering practice.

2.8 Summary

There are comprehensive studies on beach changes due to cross-shore as well as longshore sediment transport model. Although most of the major factors affecting beach morphology have been considered, there are still many factors that are poorly understood which restrict the extension of models in the application to simulate regional and long-term processes. Due to this uncertainty, the constants and coefficients become semi-empirical and assume a wide range of application in various sites.

Cross-shore sediment transport models are mainly used to model beach profile changes. Models are reviewed based on their theoretical basis (mainly sediment transport) and the extent to which they were verified. Still many models are undergoing constant development and improvement and each model may be the best for a specific case (purpose) or under specific conditions.

Most studies on shoreline change models are based on the one-dimensional sediment balance equation which is referred to as one-line model equation. These models have the advantage of being very fast and they can predict long-term shoreline changes very well after suitable calibration. However, they cannot accurately predict the impact of morphological changes in the vicinity of coastal structures that occurs due to short-term events. A typical coastal area model consists of several modules describing the wave field, the spatial distribution of wave-induced currents, the associated sediment transport fluxes and finally the resulting spatial and temporal changes of the bed level.

A number of beach morphological change models have been developed in the global scenario, but no such specific model has been developed, calibrated and verified in the field conditions of the country. Since the vast coastline of the country is subjected to high erosion and accretion during different seasons, there is a need to develop morphological change models applicable for our coast. The development of such models assumes greater importance in view of the increasing developmental activities along the vast coastal zone of the country.

03

Numerical Modelling

Chapter 3
NUMERICAL MODELLING

3.1 Introduction

As seen already in Chapter 2, a number of models on beach morphological change are available in literature but for Indian scenario no such specific model has been developed, calibrated and verified in the field conditions. There is a need to develop beach morphological change models and these models have to be calibrated and validated for field conditions in this scenario. There are two modes of sediment transport viz. alongshore and onshore/offshore transport. Based on this two numerical models have been developed to simulate long-term and short-term beach morphological changes. The process of development of the models is discussed in this Chapter.

3.2 Shoreline Change Model (Long-term Model)

Many of the existing models for predicting the shoreline variation assume that the sediment balance associated with the shoreline evolution process could be described in terms of the variations in the shoreline geometry. Further, the beach profiles between the underwater contour defining the limiting depth of the sediment transport and the foreshore end of the berm are assumed to be similar along the coast in an average sense. These assumptions are referred to as one-line model assumptions. The models based on these assumptions are used to predict the long-term evolution (time scales of several months) of the shoreline as a consequence of alteration in the wave climate and/or the rate of sediment transport along the shore introduced by the coastal constructions like groins, breakwaters, harbours and offshore dredging.

In long term shoreline evolution problems, the irregular cross-shore beach profile is approximated by a representative profile with a uniform slope. The one-line model assumes, the average slope of the yearly mean beach profile remains nearly constant as long as there is no significant change in the beach material characteristics and the wave climate from year-to-year and this mean profile could well serve the role of the representative profile. These one-line models equate the changes of beach volume to the sediments transported out of a given area by different agencies, and among them the spatial gradient of longshore sediment transport is the major contributor in the sediment balance relation. Hence, to simulate the long term shoreline evolution adjacent to coastal structures accurate estimates of the longshore transport in the coastal regions become essential. It should also be noted that the one-line model assumptions are rarely satisfied by short time scale variations in the shoreline, and evidently models based on these assumptions cannot predict short term beach profiles variations.

3.2.1 Longshore sediment transport

There are two modes of longshore sediment transport viz. sediment transport in suspension called suspended and that along the bed called bed load transport. The finer portion of the sediment is generally transported as suspended load and the coarser portion as bed load. Because of the turbulence generated by the breaking waves, much of the finer sediments are placed in suspension. Hence the suspended load transport rate is relatively high in the breaker zone. Wave induced orbital motion sets some of the finer sediment seaward of the breaker zone into suspension. The sand in suspension is then moved down coast by the wave induced longshore current. The bed load transport occurs over the entire zone of the longshore current and the maximum transport occurs near the breaker zone where longshore current velocities are maximum. Sediments also move along a zigzag path over the beach face, forced by the oblique wave and the return gravity flow.

Due to the complexity of the wave induced sediment transport processes and due to the limitation on the information available on this phenomenon, the procedures for estimating the transport rate are mostly based on semi-empirical relationships. These relationships correlate the rate of longshore transport 'Q' with the wave energy flux 'E' in the longshore direction. These relations are of the form,

$$Q = A\,E^n \tag{3.1}$$

Such a commonly used relationship in engineering applications is (Shore Protection Manual, 1984),

$$Q = 7500\,P_1 \tag{3.2}$$

where Q is the longshore sediment transport rate (m^3/year) and P_1 is the longshore energy flux at the breaker point given by,

$$P_1 = E_b C_{gb} \sin \alpha_b \cos \alpha_b \quad (3.2\ a)$$

where the energy density of waves $E_b = (1/8)\rho g H_b^2$ per unit length along the wave front, C_{gb} is the wave group velocity and α_b is the angle of the wave propagation direction with respect to the normal to the shoreline. The suffix 'b' refers to variables at the break point. Assuming that the wave energy flux, in the absence of diffraction, remains constant along a wave ray up to the wave break point and noting that the deep water group velocity $C_{go} = g\, T/4\pi$, P_1 in terms of the deep water wave height (H_o) is given by,

$$P_1 = (1/32\pi)\, \rho g^2\, H_o^2\, T\, K_r^2 \sin \alpha_b \cos \alpha_b \quad (3.3)$$

where K_r is the refraction coefficient. If the coastal region under consideration is in the diffracted wave region, K_r in Eqn.3.3 is replaced by the product of the refraction and the diffraction coefficients, ' $K_r\, K_d$'. Then the longshore transport,

$$Q = A\, K_r^2\, K_d^2 \sin \alpha_b \cos \alpha_b \quad (3.4)$$

where $A = (1290/32\pi)\ \rho g^2\ H_o^2\ T$ (m^3/year). When the coastal region under consideration is not in the diffracted wave region, the diffraction coefficient $K_d = 1$.

The above relation (Eqn.3.4) does not consider the variables such as beach slope and breaker type, and sediment characteristics. The breaker angle α_b is usually small and difficult to measure precisely by visual observation. Hence it is better to use measured wave data in the field than visually observed data, which has a high degree of uncertainty.

3.2.2 Problem formulation

In the one-line model, the orientation of the co-ordinate system is chosen such that the x-axis lies roughly parallel to the beach. The shoreline is represented by $y_s(x,t)$, where 't' is the time (Fig. 3.1 b). Fig. 3.1 a shows the co-ordinate system and the orientation of the coast. y_c in Fig. 3.1a is the deep-water limit of the beach or the limit of active sand transport beyond which beach profile changes can be assumed negligible. This deep water limit of active sand transport is called depth of closure

(D_c). The water depth D_c at $y = y_c$ is one of the important parameters in one-line models and it depends upon the wave characteristics.

Due to the random nature of the wave conditions, it is difficult to define the position of y_c and to estimate D_c. Wills and Price (1975) estimated D_c as twice the average breaking height. Sunamura and Horikawa (1977) assigned a value somewhat larger than the maximum breaker height, and Walton and Chiu (1979) recommended a value 1.3 times the breaker height. Kraus and Harikai (1983) first suggested the use of Hallermier's (1978, 1981) equation:

$$(D_c/H_s)=2.28- 68.5\ (H_s/gT_s) \tag{3.5}$$

where H_s and T_s are respectively the significant height and period of incident waves.

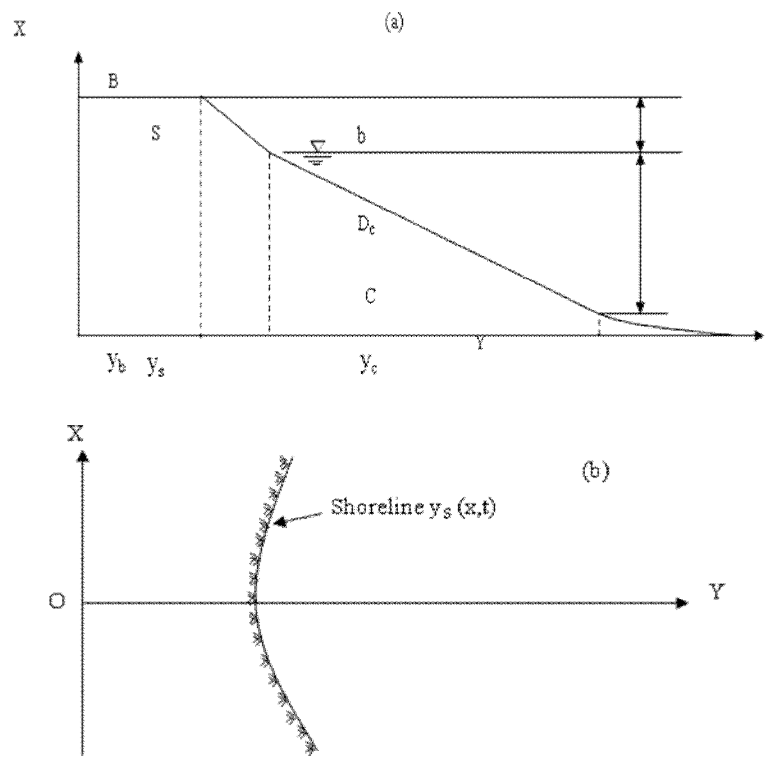


Fig. 3.1 Coordinate system for the description of shoreline evolution: (a) Sectional view and (b) Plan view of the beach

Finally, Kraus and Harikai (1983) recommended the use of a constant limiting depth for long-term simulation extending to several months, since wave forecasting over a long time period is not possible with the available wave forecasting techniques. The value is to be fixed by field calibration processes.

The numerical modelling of shoreline change essentially relates the change of beach volume to the rate of material transported from/to the beach by different agents. Fig. 3.2 shows the changes in beach volume due to the translation of the coastline (from S to S_1). We shall consider that the beach profile BSC changes to a profile $B_1S_1C_1$ during a time interval δt. Then the change in the profile can be considered to be due to a horizontal displacement δy_s (i.e. S_2S) and a vertical displacement δD (i.e. S_2S_1) which specifies the change in the sea level over a time interval, δt. Then the change in the volume over time δt of the beach material over a length δx of the beach due to the horizontal displacement of the shoreline is given by,

$$\delta V_y = (b + D_c)\, \delta y_s\, \delta x \qquad (3.6)$$

where b is the height of the berm. The change in volume due to vertical displacement is given by,

$$\delta V_D = (y_c - y_b)\, \delta D\, \delta x \qquad (3.7)$$

In the above equations a few terms which are very small are ignorable. The contribution from the area CC_1C_2 is very small compared to the sediment volume defined in Eqn. 3.7. Similarly the beach volume changes due to the change in mean sea level δD is also negligible.

It has been observed that the mean beach slope changes when the wave condition favouring deposition of beach material changes to the condition favouring erosion and vice-versa. Although the one-line models are based on the assumption of constant beach slope, it is possible to include the effects of changes in beach slope in the general formulation of shoreline evolution. If the beach slope changes from θ_1 to θ_2 over a time δt (Fig. 3.3), the volume of sediment transport δV_S, due to change in the beach slope becomes,

$$\delta V_S = 1/2\, D^2_c\, (\cot \theta_2 - \cot \theta_1)\, \delta x \qquad (3.8)$$

Then the total change in beach volume δV over a time interval δt is given by

$$\delta V = \delta V_y - \delta V_D + \delta V_S$$

$$= (b+D_c)\,\delta y_s\,\delta x - (y_c - y_b)\,\delta D\,\delta x + D_c^2\,(\cot\theta_2 - \cot\theta_1)\,\delta x/2 \quad (3.9)$$

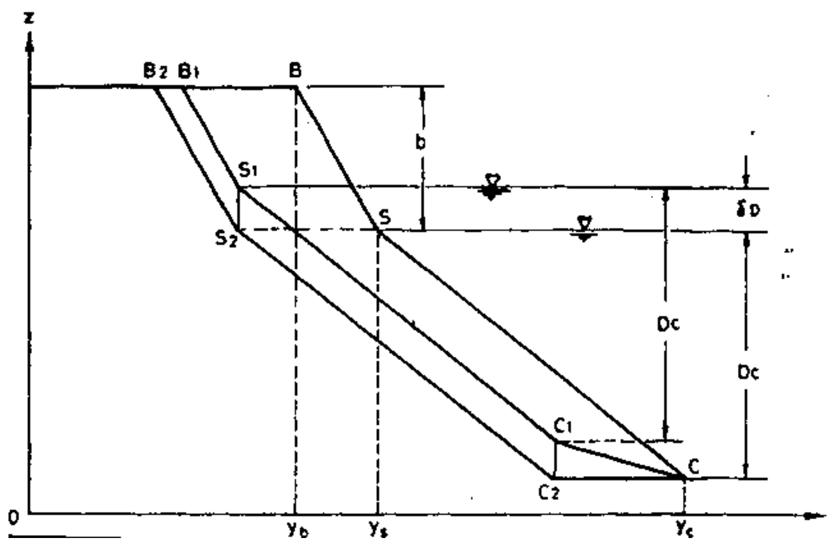


Fig. 3.2 Change in beach volume due to translation of beach profile

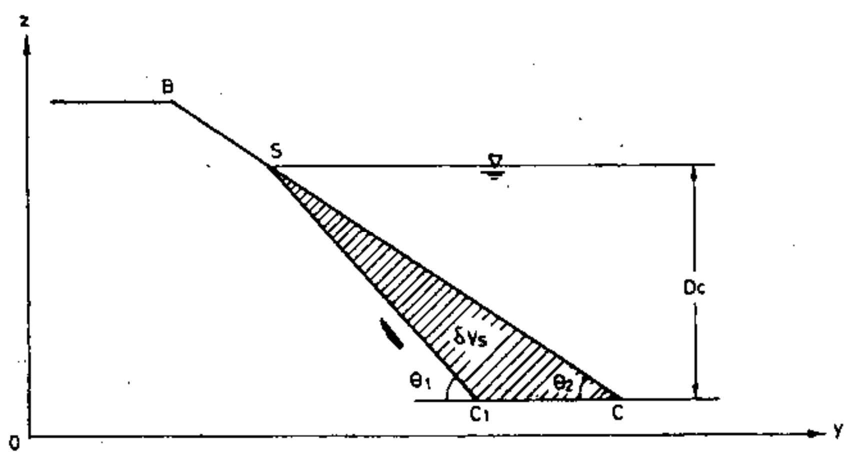


Fig. 3.3 Change in beach volume due to change in beach slope

which should be equal to the spatial variation in longshore transport '$(\partial Q/\partial x)\,\delta x\,\delta t$' (Fig. 3.4) together with,

a) the loss/gain of sand by wind

b) the loss of silt contained in the bluff which tends to move offshore as suspended sediment when there is erosion of beaches

c) the loss of sand by currents during storms and loss by rip currents

d) the quantity of sand dredged and deposited during beach nourishment.

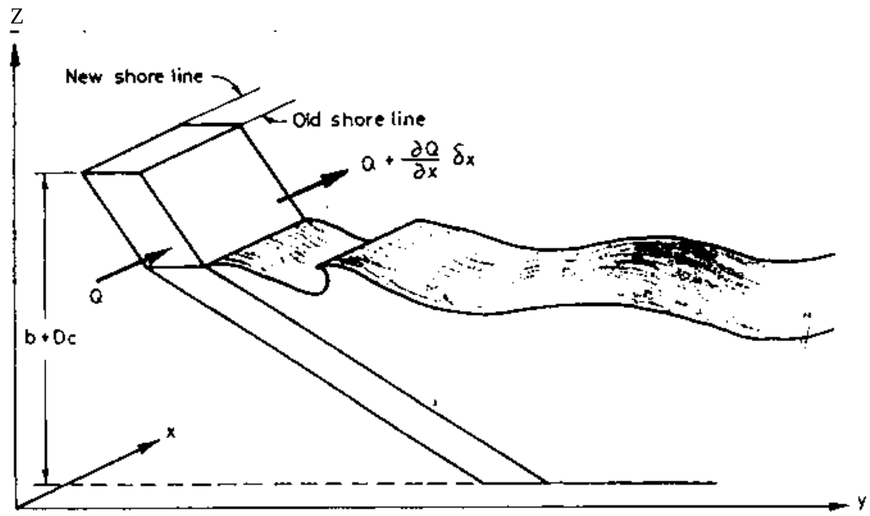


Fig. 3.4 Schematic diagrams showing the beach material balance

Thus over a time interval δt we get,

$$(b + D_c)\ \delta y_s + (y_c - y_b)\ \delta D + (\cot\theta_2 - \cot\theta_1)\ D_c^2/2 = [-\ \partial Q/\partial x + q(x)]\ \delta t \qquad (3.10)$$

where Q stands for the rate of material transported along the beach and q(x) is the sum of the sediment added per unit length and unit time to the beach by the agents listed above. When the change in the mean sea level and beach slope is not significant, the shoreline evolution Eqn.3.10 in the limit of $\delta t \rightarrow 0$ becomes,

$$(b + D_c)\ \partial y_s / \partial t = -\ \partial Q/\partial x + q(x) \qquad (3.11)$$

Defining the non-dimensional quantities (quantities with under caps) as follows,

$$x = \hat{x}(b + D_c) \quad ; \quad y_c = \hat{y}(b + D_c)$$

$$t = \hat{t}(b + D_c)^3 / A \qquad Q = \hat{Q}A$$

$$q = \hat{q}A/(b + D_c)$$

$$dy/dx = (d\hat{y}/d\hat{x}) \qquad (3.12)$$

the shoreline evolution equation and the longshore transport equation in non-dimensional form become,

$$\partial\hat{y} / \partial\hat{t} = -(\partial\hat{Q} / \partial\hat{x}) + \hat{q}(x) \tag{3.13}$$

$$\hat{Q} = K_r^{\ 2}\ K_d^{\ 2}\ \mathrm{Cos}\ \alpha_b \mathrm{Sin}\ \alpha_b \tag{3.14}$$

In Eqns. 3.12 the dimension of A is the same as that of Q (see Eqn.3.4). In subsequent discussions, the caps over the non-dimensional variables are omitted for the sake of convenience.

3.2.3 Analytical solution

Le Mahaute and Soldate (1980) (hereafter referred to as LMS) constructed an analytical solution to a simplified shoreline evolution problem. The assumptions in this simplification are that during the study period, there is no change in the mean sea level and the beach slope. And the beach has an initially straight coastline bounded on one side by a breakwater perpendicular to the coast with wave incidence as shown in Fig. 3.5. Then the diffraction coefficient,

$$K_d = 1 \tag{3.15}$$

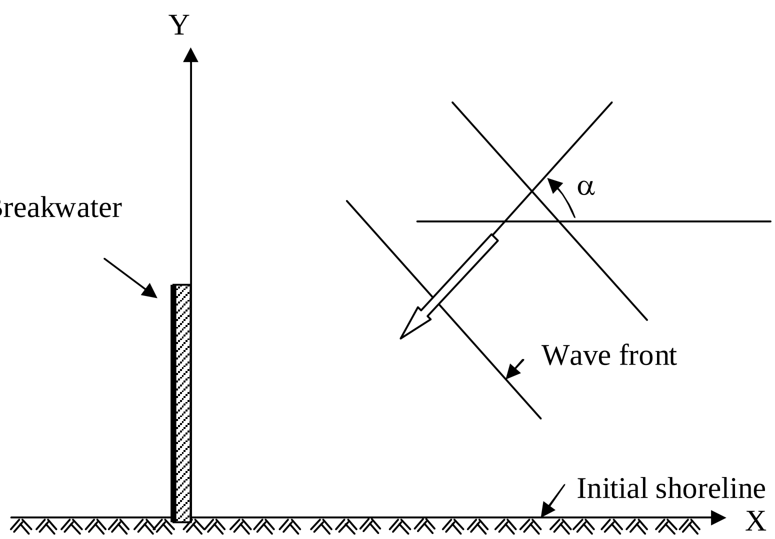


Fig. 3.5 Schematic diagram showing breakwater and initial position of shoreline

Assuming that the nearshore depth contours are nearly parallel, the refraction coefficient is approximated as,

$$K_r^2 = \mathrm{Cos}\ \alpha_o / \mathrm{Cos}\ \alpha_b \tag{3.16}$$

where 'α_o', the deep water wave direction with respect to the seaward normal to the shoreline, is given by,

$$\alpha_o = \alpha - \alpha_{sp}$$

$$= \alpha - \tan^{-1}(-\,dx/dy_s) = \alpha - \tan^{-1}(dy_s/dx) + \pi/2 \qquad (3.17)$$

α and α_{sp} are the angles that the direction of waves in deep water and the seaward normal of the shoreline make with the x-axis, respectively (Fig. 3.6).

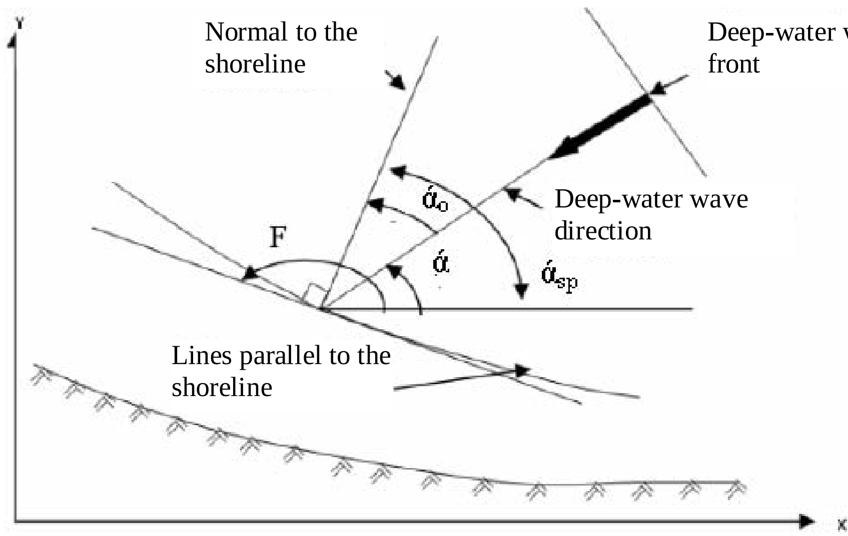


Fig. 3.6 Schematic diagram defining angles α_0, α, and α_{sp}

In the case of parallel bottom contours, Le Mehaute and Koh (1967) approximated the breaker angle 'α_b' as,

$$\alpha_b = \beta\,\alpha_o \qquad (3.18)$$

where, $\beta = 0.25 + 5.5\,(H_o/L_o)$ (3.19)

L_o is the deep water wave length. Using Eqns.3.13 to 3.19 and noting that $(d\hat{y}/d\hat{x}) = (dy/dx)$ the shoreline evolution Eqn. 3.13 can be written in the form,

$$(\partial y/\partial t) = a\,(\partial^2 y/\partial x^2) + q(x) \qquad (3.20)$$

where $a = (\beta \cos\alpha_o \cos\alpha_b - \sin\alpha_o \sin\alpha_b)/[1 + (dy/dx)^2]$. In the above equation, caps over the non-dimensional variables are omitted, and the equation is non-linear, since 'a' is a function of (dy/dx).

LMS further assumes that only the longshore transport due to wave induced longshore current exists and the beach material transport by wind, rip currents and currents generated by environmental forcing other than waves i.e. $q(x) = 0$ in Eqn. 3.20. They linearise Eqn.3.20 by assuming 'a' as a constant and approximate it by its value at $x=0$.

For a beach with an initially straight coastline bounded on one side by a breakwater perpendicular to the coast (Fig. 3.5), the boundary conditions are,

$$(dy/dx)_{x=0} = \tan(\alpha + \pi/2) \qquad (3.21)$$

and $$y(x, t) \rightarrow 0 \text{ as } x \rightarrow \infty \qquad (3.22)$$

The boundary condition, Eqn.3.21 and 3.22, requires that the wave front be parallel to the shoreline, i.e. $\alpha_b = 0$ and $a = \beta / [1 + \tan^2(\alpha + \pi/2)]$, at $x = 0$. When $q(x) = 0$, the solution to this problem becomes,

$$y(x, t) = \tan(\alpha + \pi/2) [x \,\mathrm{erfc}\{x /\sqrt{(4at)}\} - 2\sqrt{(at/\pi)} \exp (- x^2/4at)] \qquad (3.23)$$

where $\mathrm{erfc}\{x\} = 1-\mathrm{erf}\{x\}$ and $\mathrm{erf}\{x\} = (2/\sqrt{\pi}) \int_0^x \exp(-u^2)\, du$. This solution does not conserve mass, since the beach material deposited on the up drift side of the breakwater per unit time given by,

$$[\partial \{\textstyle\int_0^\infty y dx\} /\partial t] = \int_0^\infty (\partial y/\partial t)\, dx = \beta \cos\alpha \sin\alpha$$

is not equal to the sediment entering the coastal region per unit time, i.e., the longshore transport rate Q as $x \rightarrow \infty$, given by $Q_\infty = \sin\alpha \sin[\beta(\alpha - \pi /2)]$ as $x \rightarrow \infty$, $\alpha_o = \alpha-\pi /2$.

In order to satisfy the conservation of mass, LMS reformulate the problem in terms of longshore transport Q as follows.

Differentiating the governing equation (3.13) with respect to x we get,

$$[\partial (\partial y/\partial t) / \partial x] = - \partial^2 Q/\partial x^2 + (\partial q/\partial x) \qquad (3.24)$$

Then from Eqn.3.17 we have,

$$[\partial (\partial y/\partial t)/ \partial x] = [\partial (\partial y/\partial x)/ \partial t] = - \sec^2(\alpha - \alpha_o + \pi/2) (d\alpha_o/dt) \qquad (3.25)$$

Noting that $(\partial Q/\partial t) = (dQ/d\alpha_o) (d\alpha_o/dt)$, we have from Eqns. 3.24 and 3.25,

$$(\partial Q/\partial t) = a\,(\partial^2 Q/\partial x^2) - a\,(\partial q/\partial x) \qquad (3.26)$$

When $q(x) = 0$, $\alpha < \pi/2$, treating 'a' as a constant and approximating it at $x = 0$ (i.e. $a \equiv \beta / [1 + \tan^2(\alpha + \pi/2)]$), LMS obtained an analytical solution to the Eqn. 3.26 subject to boundary conditions, $Q(0, t) = 0$ and $Q(x, t)\big|_{x \to \infty} = Q_\infty = \cos \alpha_o \sin\alpha_b$ as,

$$Q(x, t) = Q_\infty \,\mathrm{erf}\{x / \sqrt{(4\,a\,t)}\} \qquad (3.27)$$

Substituting Q(x, t) in Eqn.3.13 and integrating over t, we get,

$$y(x, t) = (Q_\infty / a)[x\,\mathrm{erfc}\{x/\sqrt{(4\,a\,t)}\} - 2\sqrt{(a\,t / \pi)}\,\exp\{-x^2 / (4\,a\,t)\}] \qquad (3.28)$$

It can be seen that at $x = 0$ (i.e. at the breakwater) shoreline varies as $t^{1/2}$ and the time required for the shoreline to advance from its initial undisturbed position, along the breakwater through a distance 'L' towards its seaward end is given by

$$t = \pi\,a\,L^2 / (4\,Q_\infty{}^2) \qquad (3.28\ a)$$

This is used in Eqn. 3.6.

In analytical solutions though it is relatively easy to formulate the equations, it is much too complicated to obtain solutions. More realistic models of greater complexity can be investigated using numerical techniques.

3.2.4 Numerical scheme

The numerical scheme of Kraus and Harikai (1983) is used to solve longshore sediment continuity equation. When the changes in the mean sea level and beach slope are not significant, the governing equations describing the shoreline change in non-dimensional form is given by,

$$(\partial y/\partial t) = -(\partial Q/\partial x) + q(x) \qquad (3.29a)$$

and

$$Q = K_r^2\,K_d^2 \sin\alpha_b \cos\alpha_b \qquad (3.29b)$$

where, Q is the non-dimensional rate of longshore transport and q(x) is the source term non-dimensionalised with respect to $A/(b+D_c)$. Note that the caps over the non-dimensional variables are omitted. For a given initial and boundary conditions and wave characteristics, Eqn.3.29a is numerically solved, using the Crank - Nicholson's implicit finite difference scheme in a staggered grid system.

$$Y_{n,\,t+1} = B\,(Q_{n,\,t+1} - Q_{n+1,\,t+1}) + C_n \tag{3.30}$$

where $B = \delta t / (2\,\delta x)$

and $C_n = Y_{n,\,t} + B\,(Q_{n,\,t} - Q_{n+1,\,t} + 2\,\delta x\, q_{n,\,t})$

The set $\{Q_n\}$ is specified at the grid points and the sets $\{q_n\}$ and $\{Y_n\}$ at the centre of the grid spacing (Fig. 3.7). δx is the distance between two consecutive grid points and δt is the time interval chosen for the integration . Subscript 't' denotes the number of the time step. Eqn.3.30 expresses the sets of unknown quantities $\{Y_{n,\,t+1}\}$ and $\{Q_{n,\,t+1}\}$ at time levels (t+1) in terms of the known sets $\{Y_{n,\,t}\}$ and $\{Q_{n,\,t}\}$ at time t.

The implicit schemes compared to the explicit schemes permit the use of longer time steps of integration. The shoreline change problem being non-linear, the maximum allowable time step to preserve stability can only be determined by trial and error procedure. However, a useful approximate stability criterion can be obtained for the linearized Eqns.3.20 and 3.26. Since these linearized equations are of the form of a diffusion equation, the limit on time step to preserve stability in an explicit numerical integration scheme is given by,

$$r \equiv \delta t / \delta x^2 \leq 1/2a \tag{3.31}$$

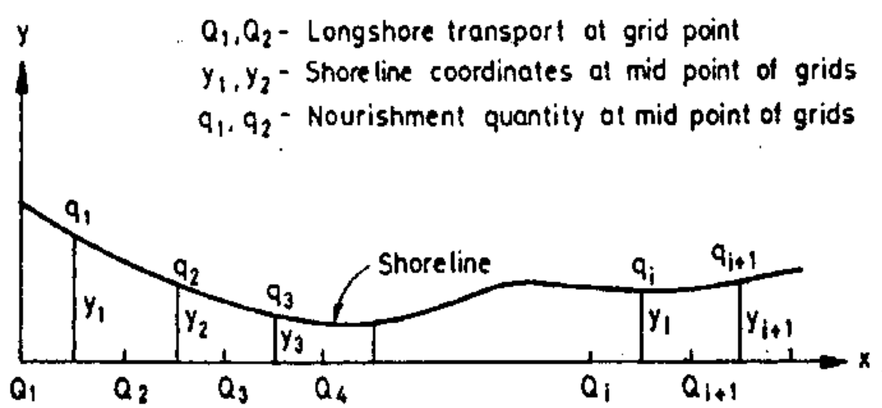


Fig. 3.7 Grid specifications for finite difference scheme (*Kraus and Harikai, 1983)*

For an implicit scheme the integration procedure is supposed to be stable for all finite values of r, but a large value will yield inaccurate estimates. It is to be noted that the shoreline evolution problem being nonlinear, the Eqn.3.31 can only provide a useful guideline for choosing the time step.

Following Kraus and Harikai (1983), the long-shore transport Q at the time step (t+1) in Eqn.3.29b is expressed in terms of the shoreline co-ordinate y, by first isolating the term involving α_{sp} using trigonometric identities, i.e.,

$$Q = K_r^2 K_d^2 \cos(\alpha_b - \alpha_{sp}) \sin\alpha_b$$

$$= K_r^2 K_d^2 (\cos\alpha_b \sin\alpha_{sp} \cot\alpha_{sp} + \sin\alpha_b \sin\alpha_{sp}) \sin\alpha_b \qquad (3.32)$$

Then Q at time level (t+1) is approximated by expressing Cot α_{sp} in the above equation in terms of y at the time level (t+1) and the remaining terms at the time level t as,

$$Q_{n,\,t+1} = E_n\,(Y_{n-1,t+1} - Y_{n,\,t+1}) + F_n \qquad (3.33)$$

where, $E_n = K_d^2 \cos\alpha_{b,t}\ \sin\alpha_{sp,\,t}\ \sin\alpha_{b,t} / \delta x$

$F_n = K_d^2\ \sin\alpha_{b,t}\ \sin\alpha_{sp,t}\ \sin\alpha_{b,t}$

Eqns. 3.30 and 3.33 contain two sets of unknowns $\{Y_{n,\ t+1}\}$ and $\{Q_{n,\ t+1}\}$. Since boundary conditions are expressed in terms of Q, we first solve for $\{Q_{n,\ t+1}\}$. Substitution of Eqn.3.30 into Eqn.3.33 gives,

$$BE_n\, Q_{n-1,\,t+1} - (1+2B\,E_n)\, Q_{n,\,t+1} + B\,E_n\, Q_{n+1,\,t+1} = E_n\,(C_n - C_{n-1}) - F_n \qquad (3.34)$$

For n = 2 to N, the above equation represents a set of (N-1) linear equations in (N-1) unknowns $\{Q_{n,\ t+1}\}$. The end values Q_1 and Q_{N+1} are specified from boundary conditions. For example, if a breakwater which prevent the longshore transport is located at the grid point 1, $Q_1 = 0$. At the other boundary it may be assumed that the transport is steady $Q_{N+1} = Q_N$, i.e $(\partial Q/\partial x) = 0$. Now the linear system of equations defined by Eqn. 3.34 is in a tridiagonal form and can be solved using a standard procedure. Then the set $\{Y_{n,\ t+1}\}$ can be determined using the Eqn.3.30. This procedure is repeated to simulate the evolution of the shoreline with time.

To solve the Eqn. 3.34 we need to specify α_b, K_r and K_d at grid points. When the grid point is not in the diffracted wave region, K_d =1 and α_b and K_r are obtained by solving the refraction equations once in every few time steps using the approximate method suggested by Le Mehaute and Koh (1967).

In the geometric shadow region of the breakwater, the following approximate procedure of Dean and Darlymple (1984) is used to compute K_d. A new co-ordinate system (x', y') is defined with its origin at the tip of the breakwater (Fig. 3.8). The distance y' is measured along the wave ray passing through the tip of the breakwater, i.e. along the line separating the geometric shadow region and illuminated region, and y' is positive towards the coast. x' is measured perpendicular to this line and it is positive on the illuminated side of the diffracted region. The diffraction coefficient at any point (x', y') in the geometric shadow region is given by the approximate relation,

$$K_d = f(u) \tag{3.35}$$

where $u = [(4/\lambda)\{(x'^2 + y'^2)^{0.5} - y'\}]^{0.5}$; u and f(u) are tabulated in Table 3.1. 'λ' is the wave length of the incident wave at the tip of the breakwater. The angle of the deflected wave front α_d in the diffracted wave region is given by

$$\alpha_d = \pi/2 + \mathrm{Tan}^{-1}[x/(L - y)] \tag{3.36}$$

where y and x are respectively the distances along and perpendicular to the breakwater (Fig. 3.8) and L is the length of the breakwater.

Table 3.1 Values of K_D for different values of u

K_D = f (u)	u	K_D = f(u)	u
0.1	2.25	0.80	0.49
0.15	1.44	0.90	0.63
0.2	1.02	1.00	0.78
0.3	0.53	1.17	1.22
0.4	0.23	1.00	1.61
0.5	0.00	0.88	1.88
0.6	0.18	1.00	2.12
0.7	0.24		

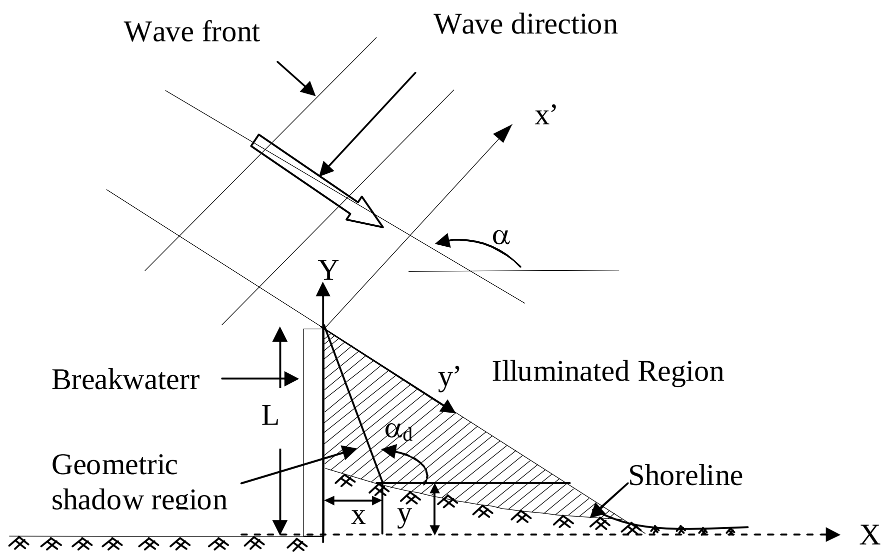


Fig. 3.8 Schematic diagram of diffraction region

A computer programme was developed in FORTRAN to facilitate faster computations to solve the equations. The numerical model called "Shoreline Change Model" is proposed for the prediction of long term shoreline change. This model was calibrated and validated using field data. The results are presented in the Chapter 5.

3.3 Profile Change Model (Short-term Model)

The physical processes associated with the cross-shore sediment transport in the breaker zone are highly complex, due to wave induced spatially varying oscillatory currents and highly irregular flow field associated with breaking waves. Modelling these changes requires quantitative description of the relation between the sediment transport rate and the physical mechanism responsible for this transport. In spite of numerous studies in this area, our knowledge on cross-shore transport processes is limited. Hence, it is impossible at the present state to follow the first principle approach to compute the sediment transport rate and the resultant beach profile changes. Beach profile prediction models have to be based on empirical relations derived from experiments carried out in laboratories and field. This requires both field and laboratory data on evolution of beach profiles under varying conditions of waves, sediment characteristics and profile shape.

Since Indian coastline is subjected to high erosion due to episodic events like, storm, monsoon waves, cyclones etc., the need for models are badly felt. Since beach profiles are mainly bar/berm type, a profile change model has been developed, calibrated and validated based on the approach of Larson and Kraus (1989), which is mainly focused on bar/berm type profile changes. They, after an extensive review of literature (laboratory scale and field experimental results) on wave induced sediment movement in the surf zone, proposed a method for simulating the beach profile changes using empirical results derived from experimental data.

3.3.1 Problem formulation

The equation governing the beach profile evolution model in the absence of sand sources and sinks, is the conservation law of sediments in the cross-shore direction,

$$\partial d/\partial t = \partial q/\partial x \tag{3.37}$$

where 'd' is the water depth from the undisturbed sea surface and 't' is time. The cross-shore co-ordinate 'x' is directed positive shoreward. 'q' is the time-averaged cross-shore sediment transport rate. The boundary conditions to Eqn. 3.37 are zero transport at the run-up limit and insignificant cross-shore transport at some seaward point far from the coast.

To quantify the transport rate, Larson and Kraus (1989) divided the beach profile into four transport zones according to the wave characteristics across the profile. The cross-shore transport rates in different zones having distinct wave characteristics are then calculated using empirical relations (Larson and Kraus, 1989) derived from field and laboratory experiments. This requires the computation of wave height distribution across the shore for the wave condition given at the seaward end of the beach profile, which forms the first step in the estimation of profile evolution. Next step is the computation of transport rates in different transport zones. Once the transport rates are known, the profile change is calculated by numerically solving the mass conservation equation (Eqn. 3.37). The basic assumptions made in this model are:

a) Coastal region is straight with parallel depth contours

b) Waves are monochromatic and approach normal to the coast

c) Ocean surface waves breaking in the surf zone are responsible for beach profile changes

d) Small amplitude wave theory is applicable to estimate wave height distribution up to the point of breaking

e) The breaker decay model of Dally (1980) is applicable to estimate the wave height distribution in surf zone

3.3.2 Wave heights in shoaling region

In a coastal region with straight parallel depth contours, waves approaching normal to the coast propagate towards the coast without refraction, and only shoaling of waves need to be considered to estimate their heights in the nearshore region. Wave height distribution from the seaward end of the profile up to the break point is estimated using the linear wave theory. In this region the wave height 'H' according to linear wave theory is given by,

$$H = H_{off} * (C_{goff} / C_g)^{0.5} \quad (3.38)$$

where the subscript 'off ' denotes the variable at the seaward end of the profile and the group velocity C_g is given by,

$$C_g = n\, C \quad (3.39a)$$

The wave phase speed,

$$C = (gT/2\pi) \tanh(2\pi h/\lambda) \quad (3.39b)$$

and

$$n = (1/2)\, [1 + (4\pi h/\lambda)/\mathrm{Sinh}(4\pi h/\lambda)] \quad (3.39c)$$

Here g is acceleration due to gravity, T is the wave period, $h = d + \eta$ is the total water depth from the mean free surface, adjusted for wave setup/setdown, 'η' and 'd' is the water depth from the undisturbed sea surface. The wavelength λ is estimated using the dispersion relation,

$$\lambda = (g\, T^2/2\pi) \tanh(2\pi h/\lambda) \quad (3.40)$$

3.3.3 Breaker decay model

After the breaker point, in the wave decay region, several models have been proposed to describe the wave energy flux variation and wave height distribution (*e.g.*, Batjes and Janssen, 1979; Dally, 1980 & Svendsen, 1984). The model proposed by Dally (1980) and subsequently used by Dally et al.(1985a,b) has been chosen here to estimate the wave energy flux and wave height distribution in the wave decay region, since it has been verified with both laboratory (Dally, 1980) and field data (Ebersole, 1987). Furthermore, the breaker decay model allows for wave reformation to occur, which is an essential feature for modeling beach profiles with multiple bars.

The governing equation of the breaker decay model in the wave decay region is given by,

$$(dF/dx) = -(\kappa/h)(F - F_s) \tag{3.41}$$

In this equation, the wave energy flux 'F', based on linear wave theory, is given by,

$$F = E\,C_g \tag{3.42}$$

where the wave energy density $E = (1/8)\,\rho\, g\, H^2$, ρ is the density of water and κ is the empirical wave decay coefficient. The stable energy flux ' F_s ' is a function of the wave height ' H_s ' which in turn is generally considered to be a linear function of the total water depth h (Horikawa and Kuo 1967),

$$H_s = \Gamma\, h \tag{3.43}$$

where Γ is the stable wave height coefficient. Then F_s is becomes,

$$F_s = (1/8)\,\rho\, g\,(\Gamma\, h)^2\, C_g \tag{3.44}$$

The assumption behind the Eqn. 3.41 is that the energy dissipation per unit plane beach area is proportional to the difference between the existing energy flux and a stable energy flux below which wave will not decay.

In the wave decay region, Eqn. 3.41 is solved to determine wave energy flux and wave height distributions. The initial condition for this equation is specified at the break point and the initial value of F is computed using Eqn. 3.42 with the estimated wave height at the break point. On the basis of experiments and field tests, Dally et al. (1985 b) recommended the values of $\kappa = 0.15$ and $\Gamma = 0.4$, for this decay model.

3.3.4 Cross-shore sediment transport

3.3.4.1 Transport zones

As a wave from deep water region propagates shoreward, it undergoes transformation due to shoaling (i.e. change in wave length and wave height due to change in water depth). When the wave height reaches certain limiting value, the wave breaks and starts dissipating energy rapidly, i.e. enters the wave decay region.

With reference to waves propagating shoreward, Larson and Kraus (1989) divided the shallow coastal region into four zones (Fig. 3.9) having different hydrodynamic properties leading to different transport relationships:

Zone - 1. Pre-breaking Zone - From the seaward depth of significant sand transport to the break point.

Zone - 2. Breaker transition Zone - From the break point to the plunge point

Zone - 3. Broken wave Zone - From the plunge point to the point of wave reformation or to the off-shore end of the swash zone

Zone - 4. Swash Zone - From the shoreward boundary of the surf zone to the shoreward limit of the run up

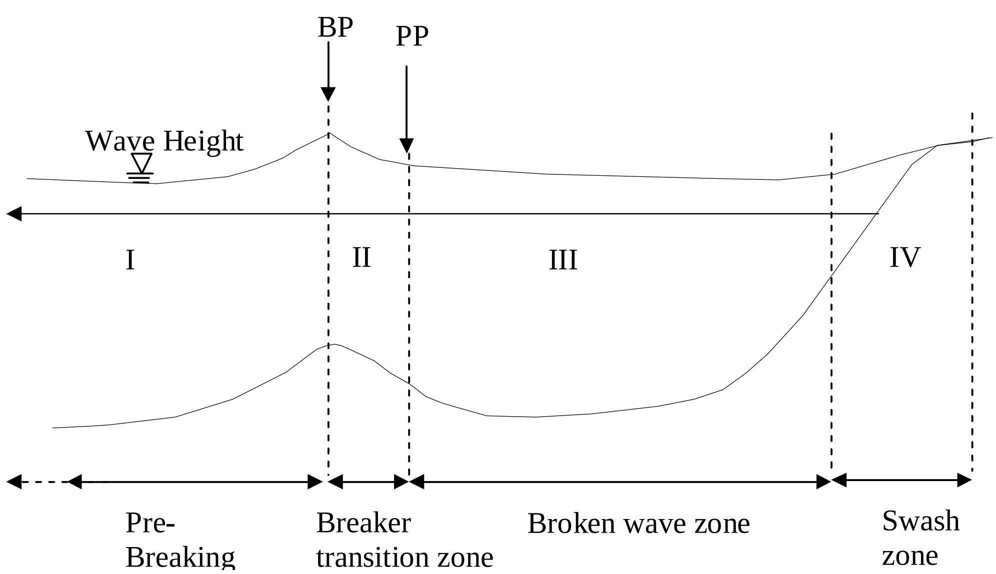


Fig. 3.9 Principal zones of cross-shore transport

Sometimes the energy of the broken wave reaches a stable energy state beyond which there will be no significant reduction in energy and the wave reformation occurs, *i.e.* the translatory broken wave form reverts to an oscillatory wave. The reformed wave will go again through shoaling, breaking and decay. If several break points occur with intermediate wave reformation, several zones of type 1, 2 and 3 will be present along the profile.

3.3.4.2 Transport direction

Depending on, the wave conditions, existing profile shape and sand properties, the cross-shore sand transport rate will be generally either offshore or onshore over the entire profile. Offshore transport results in erosion at the landward end of the beach profile and formation of a bar near the break point, whereas onshore transport leads to accretion of beach on the foreshore and berm build-up, and the gradual disappearance of the bar near the breakpoint. These two types of profile response forming two distinctly different beach shapes are commonly observed in both laboratory and field studies (Thomas and Baba, 1986), and are known as bar and berm profiles. As the formation of bar and berm profiles are related to the direction of cross-shore sediment transport, the criterion used for delineating bar and berm profile could be used to determine cross-shore transport direction. The berm profile corresponds to the onshore transport of sediments and a bar profile correspond to the offshore transport. The bar/berm profile configurations are also referred as erosional/accretional, winter/summer or storm/normal or dissipative/reflective profiles.

A large number of criteria have been developed for predicting the general response of a beach profile (a bar or berm profile) to incident waves (e.g. Dean, 1973; Sunamura and Horikawa, 1974; Larson and Kraus, 1989). Table 3.2 gives a summary of criteria developed for distinguishing bar / berm profile response of a beach, when the incident waves on the beach are regular. The deep water wave steepness 'H_o/L_o' (the ratio between the wave height and the wave length in deep water,) appears in all criteria. Other parameters appearing in these criteria are, the sediment characteristics, such as average grain size or sediment fall velocity (the fall velocity incorporates both grain size and fluid viscosity) and or the beach slope.

Table 3.2 Criteria for classifying bar (erosional) and berm (accretionary) profiles

Author	**Parameter**	**Comments**
Waters(1939)	H_o/L_o	$H_o/L_o > 0.025$, bar < 0.025, berm
Rector (1954)	H_o/L_o , D/L_o	$H_o/L_o < 0.0146(H_o/L_o)^{1.25}$, bar $> 0.0146(H_o/L_o)^{1.25}$, berm
Iwagaki and Noda (1963)	H_o/L_o , D/L_o	Graphically determined
Nayak (1970)	H_o/L_o, H_o/SD	Graphically determined
Dean (1973) Kriebel, Dally and Dean (1987)	H_o/L_o, $\pi w/gT$	$H_o/L_o > A\ \pi w/gT$, bar $< A\ \pi w/gT$, berm A=1.7,mainly lab scale A= 4 - 5 prototype
Sunamura and Horikawa (1974) Sunamura (1980)	H_o/L_o, D/L_o, $\tan\beta$	$H_o/L_o > C\ (\tan\beta)^{-0.27}\ (D/L_0)^{0.67}$, bar $H0/L_0 < C\ (\tan\beta)^{-0.27}\ (D/L_0)^{0.67}$, berm (C= 4, small-scale lab. Regular waves; C=13, field)
Hattori and Kawamata (1981)	$(H_o/L_o)\tan\beta$, π/gT	$(H_o/L_o)\tan\beta > 0.5\ \pi/gT$, bar $< 0.5\ \pi/gT$, berm
Wright and Short (1984)	H_b/wT	$H_b/wT > 6$, bar H_b/wT = 1-6, mixed bar and berm $H_b/wT < 1$ berm
Larson and Kraus (1989)	H_o/L_o , H_o/wT	$H_o/L_o < M(H_o/wT)^3$,bar $> M(H_o/wT)^3$,berm (M= 0.007 for regular waves in lab. Or = mean wave height for field data)

The dimensionless parameters appearing in these criteria have distinct physical meaning. The deep-water wave steepness (H_o/L_o) is a measure of the wave asymmetry, which influences the direction of the flow field in the water column. The dimensionless fall speed 'H_o/wT' used by Larson & Kraus (1989) in their criterion is a measure of the time that a sediment grain remains suspended in the water column. Also the wave height entering in the dimensionless fall speed directly introduces the magnitude of the wave height into the description of sediment motion. And a representative beach slope is implicitly contained in the dimensionless fall speed because the equilibrium beach profile depends on this quantity.

Larson & Kraus (1989) concluded that the parameters H_o/L_o and H_o/wT are the most basic and general ones for prediction of occurrence of bar and berm profiles, and the criteria used by them for fixing the direction of transport is,

$$(H_o / L_o) \quad < M\,(H_o/wT)^3, \qquad \text{bar profile or offshore transport}$$

$$\qquad\qquad > M\,(H_o/wT)^3, \qquad \text{berm profile or on-shore transport} \qquad (3.45)$$

where, M = 0.0007 is a constant, H_o is the incident wave height (m) in the case of laboratory experiments or significant wave height in deep water in the case of field observations, w is the sediment fall velocity (m/sec) and T is the zero crossing wave period (sec)

3.3.4.3 Transport rates

Quantitative knowledge of cross-shore transport provides the necessary foundation for numerical modelling. It is related to wave parameters, sand characteristics and beach profile shape. Larson and Kraus (1989) used empirical relationships, derived from Laboratory Wave Tank (LWT) experiments and verified in the field conditions, for the estimation of the magnitude of the transport rates in different transport zones.

The transport rate relations in different zones are:

Zone - 1: $$q = q_b \exp\{-\lambda_1(x-x_b)\} \qquad (3.46)$$

Zone - 2: $$q = q_p \exp\{-\lambda_2(x-x_p)\} \qquad (3.47)$$

where, q is the net cross-shore transport rate (m^3/m./sec),

λ_1 and λ_2 are the spatial decay coefficients (m^{-1}) in transport Zones 1 and 2, and x is the cross-shore co-ordinate from the seaward end of the beach profile directed positive towards the coast (m). The subscripts ' b and p' in Eqns. 3.46 & 3.47 stand for quantities evaluated at the break point and the plunge point respectively. Empirically estimated spatial decay coefficients λ_1 and λ_2 (Larson and Kraus, 1989) in the transport rate Eqns. 3.46 & 3.47 are given by,

$$\lambda_1 = 0.4\,(D_{50}/H_b)^{0.47} \quad \text{and} \quad \lambda_2 = 0.2\,\lambda_1 \qquad (3.48\ a,b)$$

where D_{50} (mm) is the median grain size and H_b (m) is the breaking wave height.

While calculating the transport rates in Zones 1 and 2, q is determined first at the plunge point using the transport rate formula Eqn. 3.47 defined for Zone 3 and at the break point using the formula defined for Zone 2. Then the transport rates at other grid points seaward of plunge point are computed using the exponential decay rates applicable in the respective Zones.

Breaking and broken waves produce turbulent conditions that put grains into suspension and make them available for transport across the profile. In Zone 3, wave energy dissipation is large and cross-shore transport is more than that in non-breaking zones. The details of sediment transport under broken waves at microscopic level are too complex and models developed to predict the rate of transport in this zone at macro-level employ simplified approaches.

Since the energy dissipation model of Kriebel and Dean (1985) requires a minimum input data for transport rate estimation, it is generally used for engineering applications. Kriebel and Dean (1985) assume that the cross-shore transport rate in fully broken waves is proportional to the excess energy dissipation per unit volume over a certain equilibrium value, which was defined by the amount of energy dissipation per unit volume that a beach with a specific grain size could withstand without generating significant sediment transport. That is, the cross-shore transport rate 'q' in fully broken wave region with horizontal sea floor, is expressed as,

$$q = K\,(D - D_{eq}) \tag{3.49}$$

where D is the wave energy dissipation per unit volume of water (Nm/m^3 /sec), which is given in terms of the spatial gradient of the wave energy flux 'F' as,

$$D = (1/h)\,(dF/dx) \tag{3.50}$$

where K is the transport rate coefficient (m^4 / N), D_{eq} is the equilibrium energy dissipation per unit volume of water (Nm/m^3 /sec).

As the transport rate also depends on the local slope of the sea floor, an extra term is added to Eqn. 3.49 to account for the effect of the local slope. With this modification the transport rate becomes,

$$q = K\,[D - D_{eq} + (\varepsilon/K)\,(dd/dx)], \qquad D > D_{eq} - (\varepsilon/K)\,(dd/dx)$$

$$= 0, D < D_{eq} - (\varepsilon/K)\,(dd/dx) \qquad (3.51)$$

where ε is the slope related transport rate coefficient (m^2/sec).

The equilibrium value of energy dissipation is defined as the amount of energy dissipation per unit volume that a beach profile with a specific grain size could withstand without generating significant sediment transport. If a beach profile is not in equilibrium with the existing wave climate, sediment will be redistributed along the profile to produce an equilibrium profile shape, in which state the incident wave energy will be dissipated without causing further significant net sediment movement.

Dean (1977) derived a simple analytical expression for the equilibrium beach profile shape in the surf zone based on the concept that, the wave energy dissipation per unit water volume will be a constant when the beach profile reaches an equilibrium; i.e.,

$$D = (1/h)(dF/dx) = \text{constant}$$

$$= (1/8)\,\rho\, g^{3/2}\,(1/h)\,[d(H^2\ h^{1/2})/dx] = \text{constant} \qquad (3.52)$$

Assuming that $H = \Gamma h$ near equilibrium, we get

$$D = (5/16)\rho g^{3/2}\Gamma^2 h^{1/2}(dh/dx) = \text{constant} \qquad (3.53)$$

Integrating the above equation with respect to x, we get

$$h = A\, x^{2/3} \qquad (3.54)$$

3.3.4.4 Total water depth

While determining the break point and integrating the Dally's equation we need to use the total water depth with respect to the mean water level which is adjusted for the wave setup or setdown at the chosen location. As waves propagate towards the shore, a flux of momentum (radiation stress) due to shoaling or breaking of waves arises in the nearshore region, and causes displacement of the mean water level. Seaward of the break point, shoaling produces an increase in wave height, which causes a corresponding increase in the momentum flux. This flux increase is balanced by lowering of the mean water elevation, called the setdown. Inside the surf zone waves dissipate energy and the momentum flux decreases producing an increase in water surface elevation called the wave setup. The wave setup/setdown 'η' is determined from the momentum equation,

$$\rho\ g\ h\ (d\ \eta/dx) = -\ (dS_{xx}\ /dx) \qquad (3.60)$$

where the radiation stress S_{xx} is given by,

$$S_{xx} = E\ (Cg\ /C - 1/2\} \qquad (3.61)$$

The wave setup/setdown 'η' is generally small, but in shallow depths near the coast, the contribution of η to the total water depth becomes important in the estimation of wave parameters in that region.

The transport rate in swash zone is assumed to decrease linearly from the end of the surf zone (Zone 3) to the run-up limit given by,

$$q = q_z\ \{(x - x_r\ )/(x_z - x_r)\} \qquad (3.62)$$

where subscripts ' z ' and ' r ' stand for quantities evaluated at the end of the surf zone and run-up limit respectively. The surf zone is arbitrarily terminated at a point where the depth of water from the undisturbed sea surface is 0.3-0.5 m. The linear transport rate also implies that the foreshore recedes or accretes uniformly along its full length. This property of the foreshore was observed occurring in many LWT experiments, in both erosional and accretionary cases. Similar characteristics have also been noted in natural beaches. However, no direct measurements on the transport rates have been made in the foreshore.

The active subaerial profile height 'z_r' or z-coordinate of the run-up limit of waves is determined using an empirical relation derived from LWT experiments, given by,

$$z_r/Ho = 1.47\ \ \xi^{0.79} \qquad (3.63)$$

where ξ is the surf similarity parameter = tan $\beta/(H_o/L_o)^{1/2}$. The slope to be used in this formula is the average beach slope in the surf zone. The corresponding x-coordinate 'x_r' of the run-up limit in Eqn. 3.62 is interpolated from the profile depths defined at grid points.

3.3.4.5 Avalanching

As the profile evolves due to accretion or erosion, the profile slope may exceed a limiting value, known as the *angle of initial yield*, beyond which the seabed becomes unstable. When the profile slope exceeds this limiting value the profile shears off and the slope decreases to a lower stable value known as the *residual angle*. This phenomenon is referred to as the avalanching. Avalanching in LWT experiments was observed if the profile slope exceeds 28^o on an average and the residual angle is generally found to be nearly 10^o- 15^o lower (Allen, 1970) than the angle of initial yield. The field test carried out by Christopher et.al.(2008) observed maximum beach slope is up to 30^o. Computationally this is performed by checking the slope between adjacent grid points over the entire profile starting from one end of the profile. If the slope between any two adjacent grid points exceeds the limiting value, the depths at the grid points are redefined by adding or subtracting half the product of the residual slope and the grid size to the average of the depths at the two grid points, i.e.,

$$
\begin{aligned}
d_{ave} &= (d_1 + d_2)/2 \\
d_1' &= d_{ave} - \mathrm{Tan}\,(18)\,\Delta x/2 \\
d_2' &= d_{ave} + \mathrm{Tan}\,(18)\,\Delta x/2
\end{aligned}
\tag{3.64}
$$

where d_1 and d_2 are the depths of the adjacent grid points (Fig. 3.10) from the free surface and d_1' and d_2' are the redefined depths.

The above relation ensures that (a) the slope between these grid points after readjustment become equal to the chosen residual angle and (b) the volume of bed material is conserved during this readjustment. Then this check-readjustment procedure is continued up to the end of the profile and repeated till the slope condition favouring avalanching is not encountered even once over the entire profile. This checking procedure has to be performed at the end of each time step.

3.3.5 Numerical scheme

Computational steps involved in the beach profile simulation model are:

a) Estimation of the wave height distribution across the beach profile.

b) Computation of sediment transport rate distribution across the beach profile.

c) Solution of sediment continuity equation across the beach profile

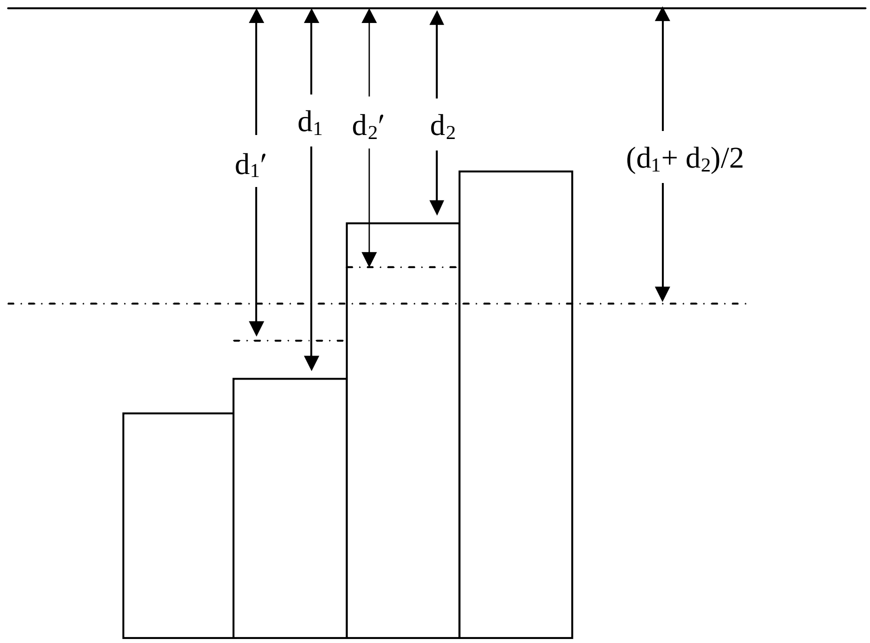


Fig. 3.10. Definition sketch describing avalanching

3.3.5.1 Wave height distribution

Wave height distribution from the seaward end of the profile up to the break point is estimated using linear wave theory (Equations (3.38 – 3.40). The break point is fixed when one of the following criteria for Breaker Index "H_b/h_b" is satisfied (Larson and Kraus, 1989):

$$H_b/h_b = 1.14\,(\tan\theta) / \{\sqrt{(H_o/L_o)}\}^{0.21} \quad (3.65a)$$

$$H_b/h_b = 0.78 \quad (3.65b)$$

$$H_b/h_b = 0.53\,(H_o/L_o)^{-0.24} \quad (3.65c)$$

In the present model, criteria (b) is used to determining the break point and the plunge point is fixed at a distance equal to three times the breaker wave height inshore of the break point (Larson and Kraus, 1989).

In the wave decay region, Eqn. 3.41 is solved using Runge-Kutta-Gill scheme to determine wave energy flux, and the wave height distribution is given by the relation,

$$H = [8\,F/(\rho\, g^{3/2}\, h^{1/2})]^{1/2} \qquad (3.66)$$

The initial condition for the Eqn. 3.41 is specified at the break point and the initial value of F is computed using Eqn. 3.42 with the estimated wave height at the break point. Integration of breaker decay model is continued up to the point where, either the ratio between F and F_s becomes equal to 1.1 (this value is considered to define the point of wave reformation) or the minimum profile depth point is reached. The minimum profile depth is fixed between 0.3 and 0.5 m after field verification. Dally et al. (1985) recommended the values of $\kappa = 0.15$ and $\Gamma = 0.4$ for this decay model.

3.3.5.2 Total water depth

To compute the total water depth h (= d +η), the variation in η (wave set-up) in the nearshore region is estimated by solving the Eqn. 3.60 from the seaward end of the beach profile. The boundary condition at the seaward end of the beach profile is given by the analytical solution of the Eqn. 3.60 as,

$$\eta = -(\pi H^2 / 4\lambda) / \mathrm{Sinh}(4\pi h / \lambda) \qquad (3.67)$$

3.3.5.3 Sediment transport rate

Steps involved in the computation of cross-shore transport rate in the transport zones 1 and 2 are:

Step 1: The transport rate at the plunge point is computed using the transport rate formula (Eqn. 3.51) defined for Zone -3.

Step 2: The transport rate at the break point is computed using the relation (Eqn. 3.47) defined for Zone -2

Step 3: Then the transport rates at other grid points seaward of plunge point are computed using relations (Eqns. 3.46 & 3.47) with the exponential decay rates (Eqns.3.48) applicable in the respective zones.

In Zone -3, the transport rate is computed using the Eqn. 3.51. In Zone -4, the run-up height is determined using the empirical expressions (Eqn. 3.63) and the transport rate computed using the Eqn. 3.62. In the present study, the direction of the cross-shore sediment transport is determined using the criteria given by Eqn. 3.45.

3.3.5.4 Solution of sediment continuity equation

Once the cross-shore transport rates (magnitude and direction) are determined, the profile change is calculated using the sediment conservation equation (Eqn. 3.37). The boundary conditions to this equation are, zero transport at the run-up limit and insignificant cross-shore transport at some seaward point far from the coastline. Then the sediment conservation equation (Eqn. 3.37) is solved using an explicit finite difference scheme. Fig. 3.11 illustrates the staggered grid system used in this numerical scheme, in which the water depths from the undisturbed sea level are specified.

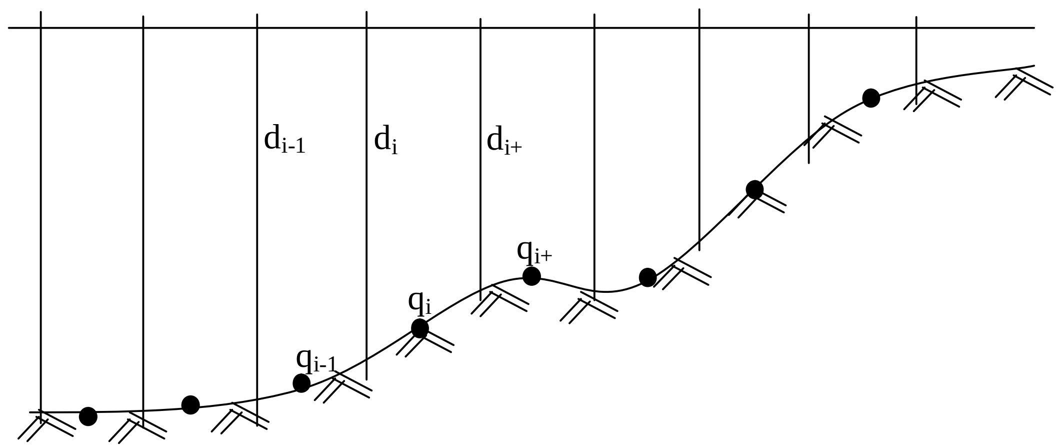


Fig. 3.11 Staggered grid system used in the numerical scheme

at the grid points and the transport rates at the centre of the grid spacing. The Eqn. 3.37 is written in explicit difference form as,

$$d_i^{t+1} = d_i^t + (q_{i+1}^t - q_i^t)\,(\delta t / \delta x) \tag{3.68}$$

where the superscript 't' denotes time level and the subscript i the grid number. δt is the time step and δx is the spatial grid spacing. Typical length and time steps used in the model for numerical stability are $\Delta x = 0.5 - 5.0$m and $\Delta t = 5$ to 10 min. A shorter length step required a shorter time step to maintain numerical stability.

To obtain a realistic description of wave height distribution across irregular profiles a three point moving average is used to obtain representative depth values:

$$d_{s,i} = 0.25\, d_{i-1} + 0.5\, d_i + 0.25\, d_{i+1} \tag{3.69}$$

The subscript 's' denotes the smoothed depth and i the grid number. The depths at the first and the last points are not smoothed. If the wave calculations are not based on a smoothed beach profile, the wave heights will respond in an unrealistic manner to small changes in the profile.

Similarly a three point moving average is used for smoothing the cross-shore transport rates. But when solving the continuity equation the exact profile depths are used.

A computer programme was developed in FORTRAN to facilitate faster computations to solve the equations. This numerical model called 'Profile Change Model' is proposed for the prediction of short-term beach profile change. This model was calibrated and validated using comprehensive field data and the results are presented in the Chapter 5.

3.4 Summary

Based on different types of sediment transport in the nearshore area, two numerical models have been developed to predict short-term as well as long-term shoreline changes. To predict long-term change of shoreline, a numerical model developed based on the approach of Kraus and Harikai (1983) is proposed. This model can be used to predict shoreline change in the vicinity of coastal structure. Beach profile change model has been developed based on the concepts of Larson and Krauss (1989). The model can be used to predict short-term profile change due to episodic events.

04

Field Data Used

Chapter 4
FIELD DATA USED

4.1 Introduction

The south-west coast of India is typical of a micro-tidal tropical environment. As in any other tropical coastal setting, waves and currents are the driving forces. In addition to the temporal variations in the hydrodynamic regime which is dominated by the south-west monsoon, there are spatial variations too. Kurian (1987) classifies the nearshore of SW coast of India into different zones of wave energy regime according to which the Trivandrum coast has the highest energy followed by a decrease towards north reaching the lowest off Tellichery. North of Tellichery, the energy level is seen to increase towards Mangalore. In the context of these spatial variations in the hydrodynamic regime, any attempt on development, calibration and validation of numerical models for beach morphological changes has to make use of comprehensive field data pertaining to different coastal locations representing different energy regime and geomorphological characteristics. In accordance with the above requirement, field data pertaining to different coastal locations have been used for the present investigation. An analysis of the field data used for the investigation is presented in this chapter.

4.2 Field Locations

Based on energy regime and environmental characteristics, three representative coastal stretches viz, Trivandrum (Valiathura, Muthalapozhi), Alleppey (Kayamkulam, Thrikkunnapuzha, Mararikkulam) and Calicut of southwest coast of India (Fig. 4.1) were selected for the study. While field measurements including offshore deployment of equipments were carried out in a couple of locations as part of this study, secondary data available at CESS were used in the case of other locations. The details of the field measurements and results of analysis of the extensive field data are presented location-wise in the following sections.

4.3 Valiathura

The Valiathura beach (Fig. 4.2) is almost straight with NW-SE orientation. The isobaths of the shelf are nearly straight and parallel, and the shelf has an average width of 45km. The inner shelf (30/20 m contour) is very steep with a slope of about 0.002. This coast like other parts of west coast of India is under the spell of southwest monsoon during June-September. Sea wall is constructed in many stretches of the Valiathura coast.

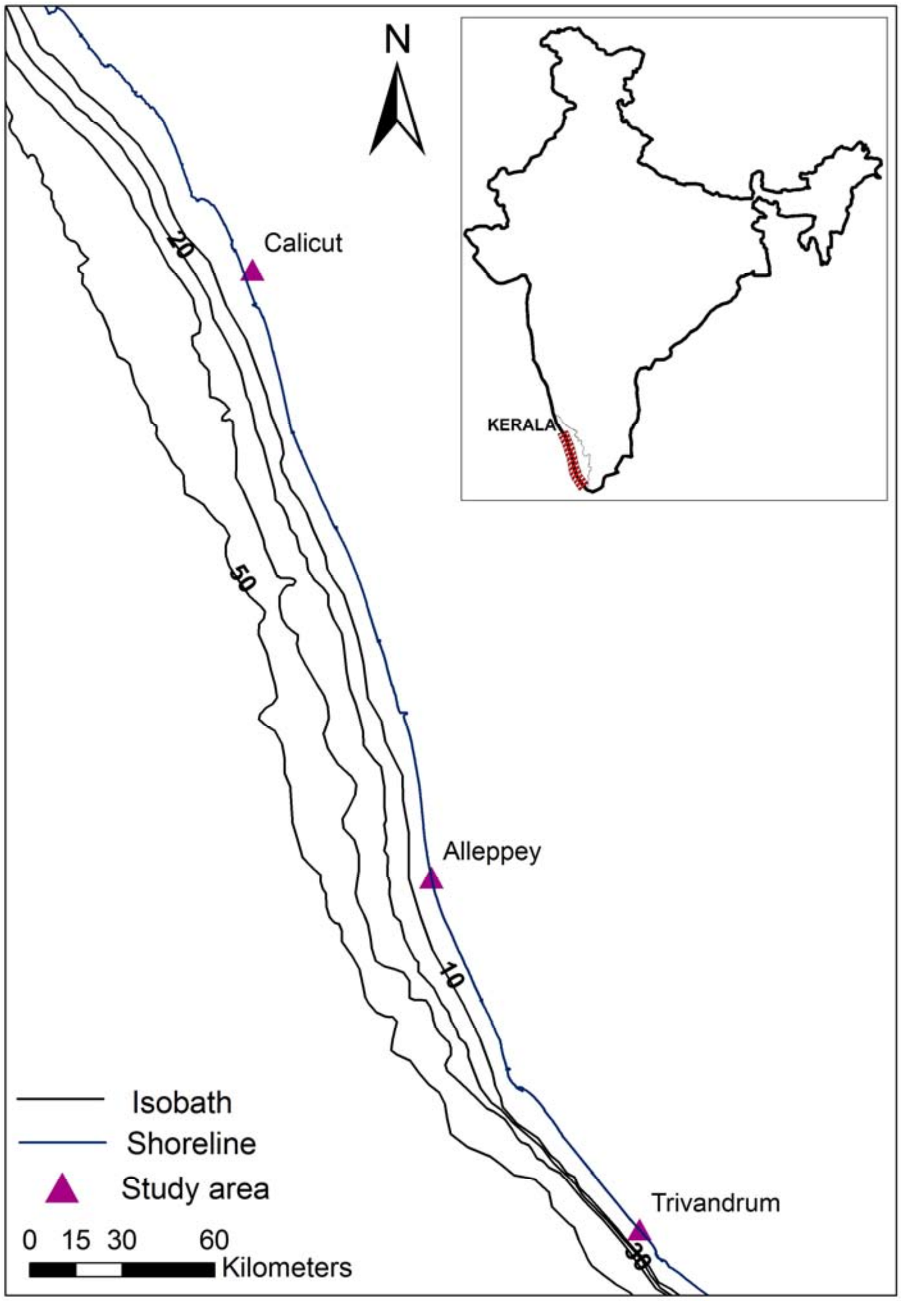


Fig. 4.1 Location map showing the general study area

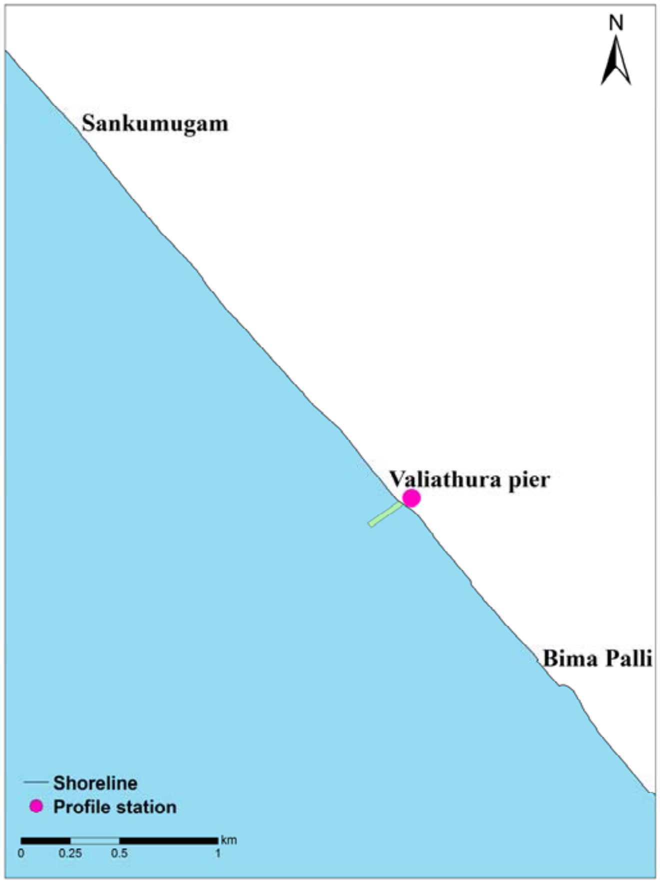


Fig. 4.2 Valiathura coast

4.3.1 Wave characteristics

Among the data requirements for model running, the first was the hydrodynamic data, which were collected using sophisticated instruments. The wave and current in the nearshore were measured by deploying a Valeport wave gauge and an Acoustic Doppler Current Profiler (ADCP) off Valiathura and nearby area at a depth of 8 m (Fig. 4.3) during 5 - 26 June 2005. Beach profile measurements using dumpy level and staff, together with surficial sediment sampling were carried out just after the offshore deployment and just before the retrieval of the equipments. The foreshore slope was derived from the profiles of the beach. The statistics of the wave for the measured period is given in the Table 4.1.

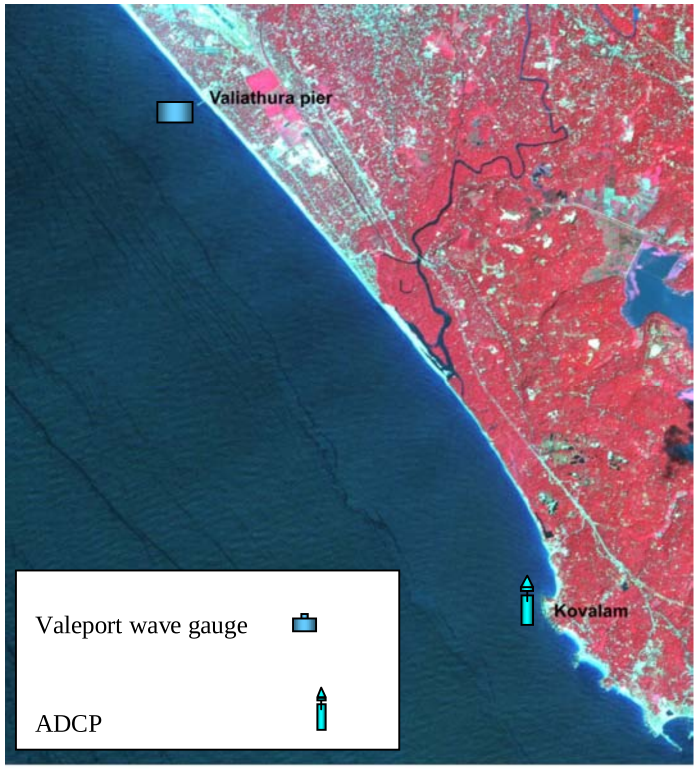
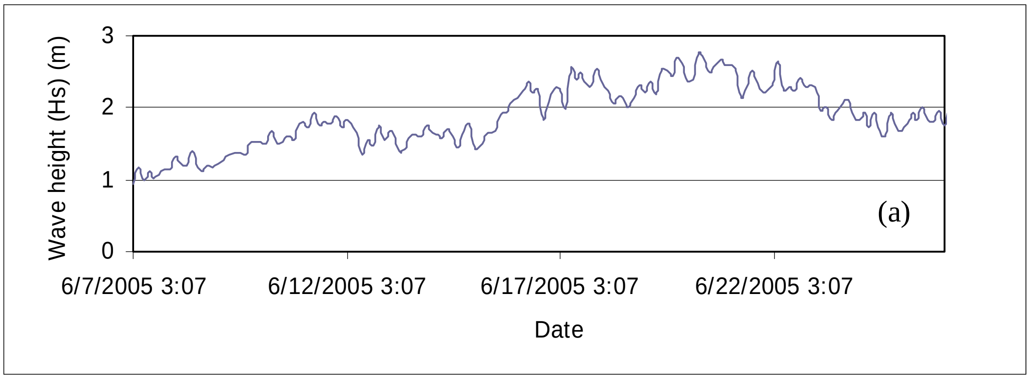


Fig. 4.3 Inshore deployment locations off the Valiathura coast

Secondary data available at CESS was also collated for the detailed modelling study. The data was collected as part of major Wave Project implemented by CESS at selected locations of south-west coast of India. The waves in the nearshore site were measured by using a pressure gauge deployed off Valiathura at a depth of 8 m during May-1980-1985. The wave data recorded during May1981 - May 1982 were taken for analysis and calibration of models. The wave characteristics derived for the area of study from the primary and secondary data are presented in the following sections.

4.3.1.1 Wave Height (H_s)

Out of the different wave height parameters, the significant wave height (H_s) is commonly used to represent wave heights. Hence the H_s is used here to present the wave height characteristics. The time series of H_s during the study period 06-06-2005 to 26-06-2005, which is representative of peak monsoonal period are presented in Fig. 4.4 (a). The peak wave intensity is seen by mid-June.



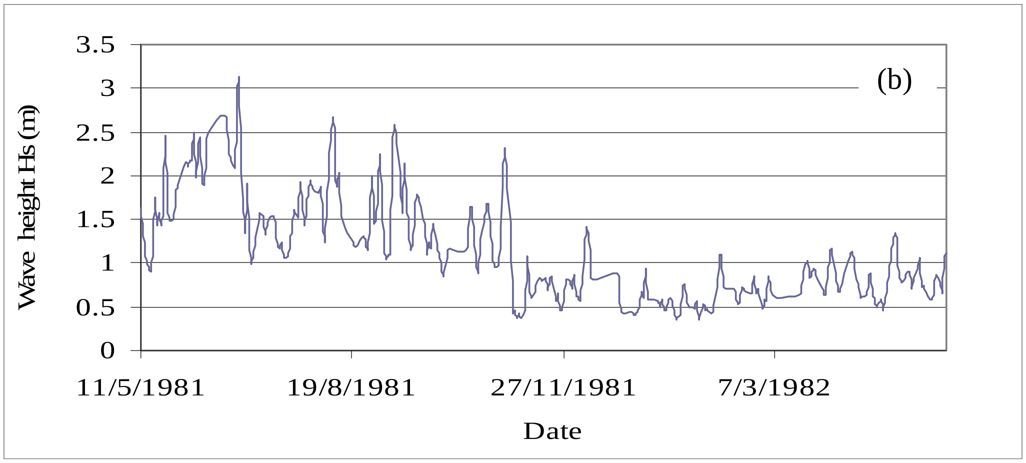


Fig. 4.4 Time series of H_s off Valiathura during (a) mid-June 2005 and (b) May1981 – May 1982

The wave statistics for June 2005 shows that H_s is in the range 0.75 – 2.66 m, with mean of 1.58 m and standard deviation of 0.48 (Table 4.1). The time series of Hs for the period May 1981– May 1982 are presented in Fig. 4.4 (b) and Hs statistics are presented in Table 4.1. The H_s observed in June-September 1981 are in the ranges 1.0-3.5 m, with mean of 1.7 m and standard deviation of 0.50**.** The mean significant wave heights occurring during monsoon of 1981 and 2005 are almost of the same order of magnitudes.

During the post-monsoon (October 1981 – January 1982), the H_s are in the range 0.36-2.3 m (Table 4.1). The maximum H_s value observed is 2.31 m, with a mean of 0.82 m and standard deviation of 0.39. During the pre-monsoon season (February – mid-May 1982), the wave intensity starts to increase significantly (Fig. 4.4 (b)). During this period the H_s values are in the range 0.42 to 1.34 m, with a mean of 0.75

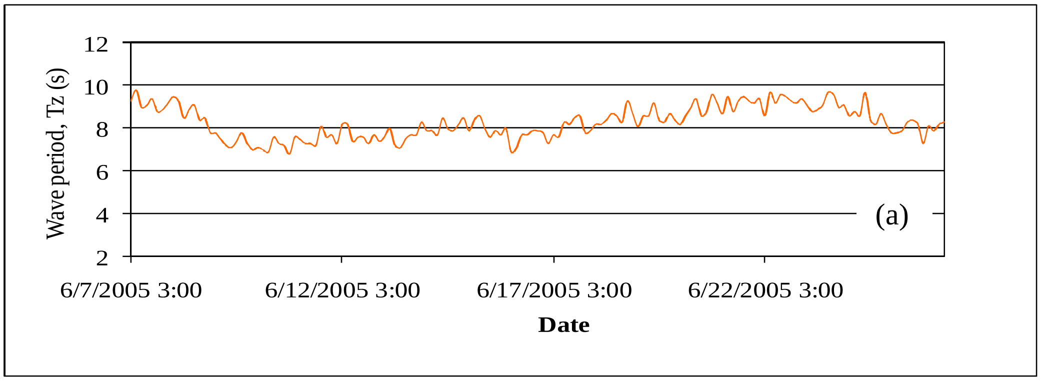
m and standard deviation 0.18. The highest wave activity was recorded in the second half of May. The H_s are in the range 1.0-1.4 m during the 3rd week of May but it increased to 1.2–1.8 m by the last week of May.

Table 4.1 Nearshore wave statistics off Valiathura during different seasons

Parameters	Period	Min.	Max.	Mean	Standard Div.
Wave Height (H_s), m	Monsoon (06.06.2005 – 6.06.2005)	0.75	2.66	1.58	0.48
Wave Period, (T_z) s		6.8	9.8	8.3	0.75
Wave direction, °N		185	298	234	24.8
Wave Height (H_s), m	Monsoon (Mid-May – September, 1981)	1.0	3.5	1.7	0.50
Wave Period, (T_z) sec		6.1	11.8	8.3	1.44
Wave direction, °N		200	270	235	20.46
Wave Height (H_s), m	Post-monsoon (October 1981– January 1982)	0.36	2.30	0.82	0.39
Wave Period, (T_z) sec		7.3	13.6	10.3	1.28
Wave direction, °N		200	205	200.3	1.31
Wave Height (H_s), m	Pre-monsoon (February – May 1982)	0.42	1.34	0.75	0.18
Wave Period, (T_z) sec		7.72	12.8	10.2	1.07
Wave direction, °N		190	210	199.7	2.16

4.3.1.2 Wave period (T_z)

The different wave period parameters generally referred to are the peak period (T_p) which is the period corresponding to the maximum energy density in the wave spectrum, the zero-crossing period (T_z) which is the period of up-crossing/down-crossing the average zero level about which the crest and trough of the waves are calculated and the average period (T_{avg}). Out of these parameters the zero-crossing period (T_z) is the commonly used parameter to represent wave periods and hence it is used in this study to represent wave period. During the monsoon period of June 2005, most of the waves are of short period with T_z in the range 6.8–9.8s. The time series plot of T_z for this period is given in the Fig. 4.5 (a).



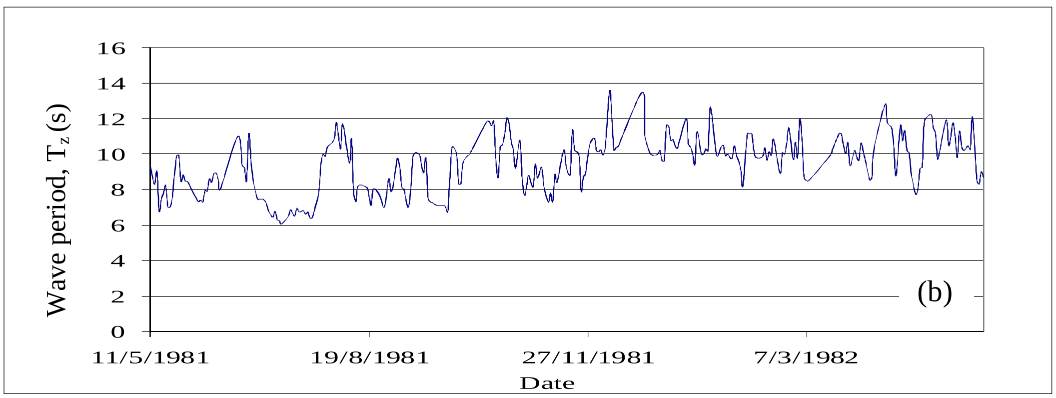


Fig. 4.5 Time series of T_z *off Valiathura during (a) June 2005 (b) May1981-May 1982*

The time series plot of T_z for the period May 1981- May 1982 is presented in Fig. 4.5b and T_z statistics in Table 4.1. Again the period of monsoon shows a narrow range of 6.05-11.8 s with a mean T_z of 8.31s. The lowest periods are observed in the first week of June. During the post monsoon period, wave period increases and lies in the range of 7.27-13.56 s with a mean of 10.31s. During the pre-monsoon period, the characteristics of T_z remain more or less the same as in the post-monsoon period (see Table 4.1).

4.3.1.3 Wave direction

The wave direction varies in accordance with seasons. During the monsoon period June 2005, the wave directions fall in the range 180–298^{o}N with an average value of

239 °N i.e.3° south of shore normal. It can be seen from Fig. 4.6 (a), that the westerlies dominate the peak wave intensity period starting from mid-June.

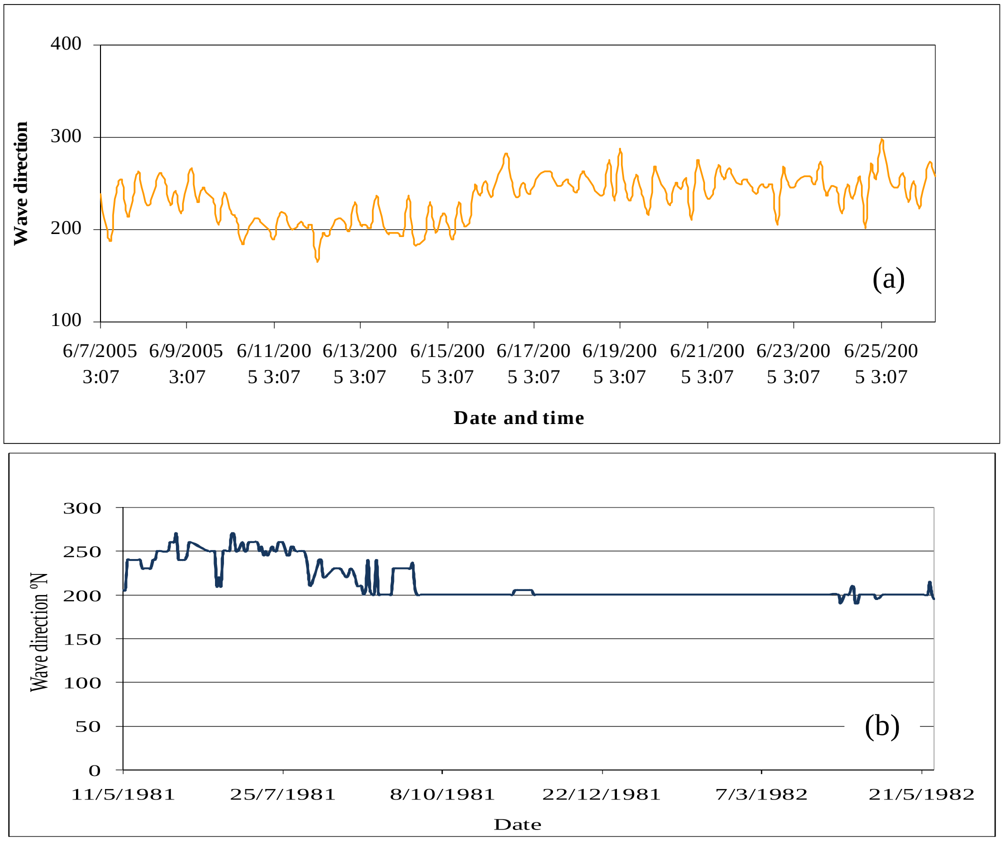


Fig. 4.6 Time series of wave direction off Valiathura during (a) mid-June 2005 and (b) May 1981 – May 1982

The time sere of wave direction during May 1981- May 1982 is presented in Fig. 4.6 (b) and the seasonal statistics in Table 4.1. Wave direction is dominated by waves from 200 to 210 ºN during post-monsoon season. During monsoon, the prevailing wave direction is westerly, due to the action of south-west monsoon and also high waves are observed during this season. During pre–monsoon season wave directions fall in the range 190–210 ºN (Fig. 4.6 (b)**).**

4.3.2 Beach profiles

As in the case of waves, primary data were collected in June 2005 and secondary data collated for the period 1980-1984. The beach profiles were measured following the dumpy level and staff method. The elevations were measured at 5 m interval with respect to fixed bench mark. The positions of berms, scarps, shoreline, etc. were also noted. The shoreline positions were also located from the profile survey. Berm height is a measurable value from the beach profile and is used as one of the input parameter in the modelling study. The beach profiles were measured at different stages of monsoon in 2005, 2007 and 2008. Each year, the profiles show high erosion at beach face followed by bar formation in the nearshore during the early stages of monsoon as seen in Fig. 4.7.

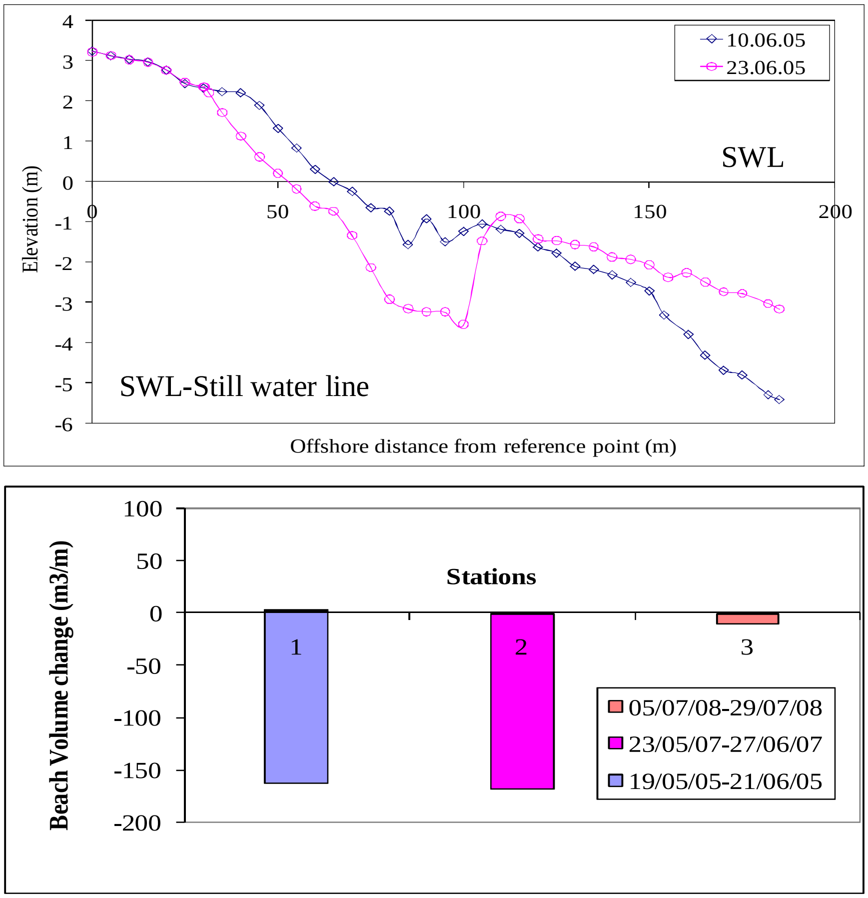


Fig. 4.7 Measured beach profiles at Valiathura pier during monsoon season of different years

4.3.2.1 Beach volume changes

The beach volume changes between successive measurement dates were calculated from the beach profile data using a computer program developed by Hameed (unpublished). The results are presented in Fig. 4.8 and Table 4.2. In general, significant erosional tendency is observed during the monsoon onset month of mid May and June, with maximum horizontal retreat of 38 m during May- June of 2007.

Table 4.2. Beach volume changes at Valiathura during monsoon season of different years

Sl. No.	Stations	Period	Beach Volume change (m^3/m)	Remarks
1.	Pier	19/05/05-21/06/05	-163.45	Erosion
2.	Pier	23/05/07-27/06/07	-167.95	Erosion
3.	Pier	05/07/08-29/07/08	-10.20	Erosion

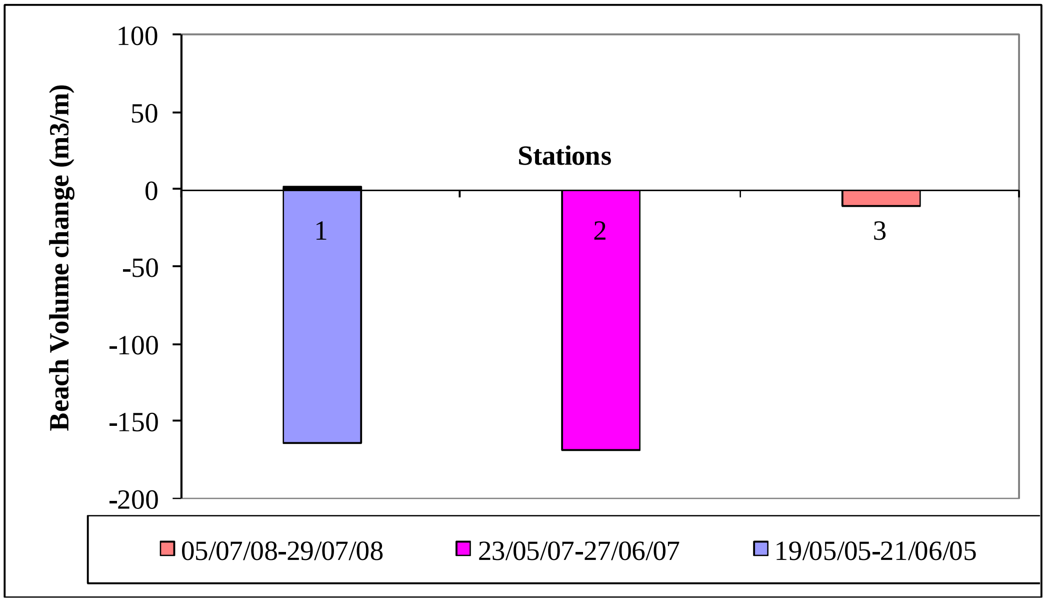


Fig.4.8 Beach volume changes at Valiathura during monsoon season of different years

4.3.3 Sediment characteristics

Sediment size is an important parameter in profile change modelling and hence textural analysis was carried out for sediment samples collected during monsoon. The size characteristics (Table 4.3) reveal that the berm sediments during monsoon are medium sized sand with a mean size of 0.25 mm, moderately well sorted, symmetrical and platykurtic in nature. Along the beach face, the sediments are medium sand with a size of 0.37 mm, well sorted, fine skewed, and leptokurtic in nature. The size characteristics can be attributed to the high wave energy conditions prevailing in the region.

Table 4.3 Statistical parameters of the sediment samples from Valiathura during monsoon 2008

Parameters	Beach Face	Berm
Mean (mm)	0.37	0.25
Sorting (mm)	0.74	0.65

4.4 Muthalapozhi

Muthalapozhi coast is part of an almost straight, NW-SE trending coast between Veli inlet and Varkala cliff (Fig. 4.9) with almost the same hydrodynamic, beach morphological and sedimentological characteristics as Valiathura. Breakwaters have been constructed for a fishing harbour at the inlet. Anchuthengu, northern sector of Muthalapozhi inlet experiences severe erosion. Some modifications to the breakwater are now being undertaken. The beach profiles at stations on either side of the Muthalapozhi breakwater were measured for study of accretion/erosion pattern and deriving shoreline positions for modelling (Fig. 4.10).

4.4.1 Wave characteristics

Kurian (1987) has categorized the coast into different wave energy zones. According to that categorization, the area of the present study belongs to high energy level. Since the energy level and geomorphological conditions of Muthalapozhi is same as Trivandrum coast, the wave characteristics off Valiathura is used for modelling study at Muthalapozhi coastal sector.

4.4.2 Beach profiles

Beach profiles were measured in the sectors on both the sides of Muthalapozhi inlet to identify critical eroding and accreting sectors and to derive shoreline positions for modelling.

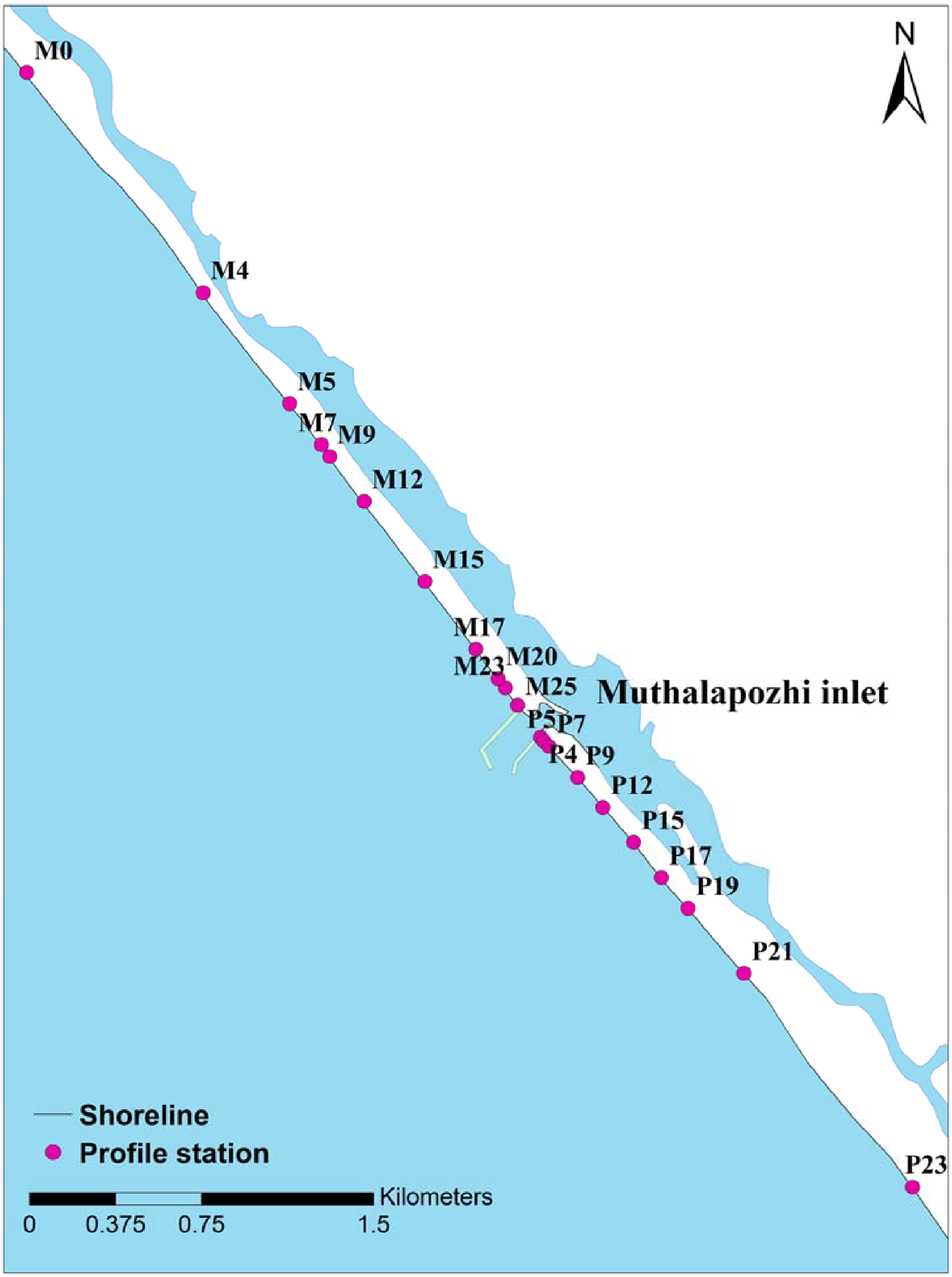


Fig. 4.9 Location map and beach profile stations of Muthalapozhi study area.

Fig. 4.10 Erosion and accretion pattern of Muthalapozhi coastal sector: (a) critical eroding sector north of inlet and (b) intensively accreting sector south of inlet

4.4.2.1 Muthalapozhi north

The sector north of Muthalapozhi inlet is subjected to severe erosion during monsoon, in addition to long term erosion. Most part of the northern coastal stretch is protected by seawall. The scarcity of sediment exists all over the season and become more severe during monsoon season. This may due to the groin effect of the breakwater which traps the sediments south of the inlet. No frontal beach is available during monsoon season and hence no profile data were taken during this season. Beach profile data were collected at regular intervals from 2002 to 2004 (Fig. 4.9). The elevations were measured at 5 m interval with respect to fixed bench mark as explained in the previous section. The positions of berms, scarps, shoreline, etc. were also noted. Beach profiles of selected locations are presented in Fig. 4.11. The changes in volume of beach material in temporal scale were calculated at different stations (M0, M4, M20 and M23). No beach is available during monsoon season in a few stations and hence no profile data were taken at these stations. The profiles show moderate accretion during pre-monsoon and severe erosion during monsoon season (Table 4.4).

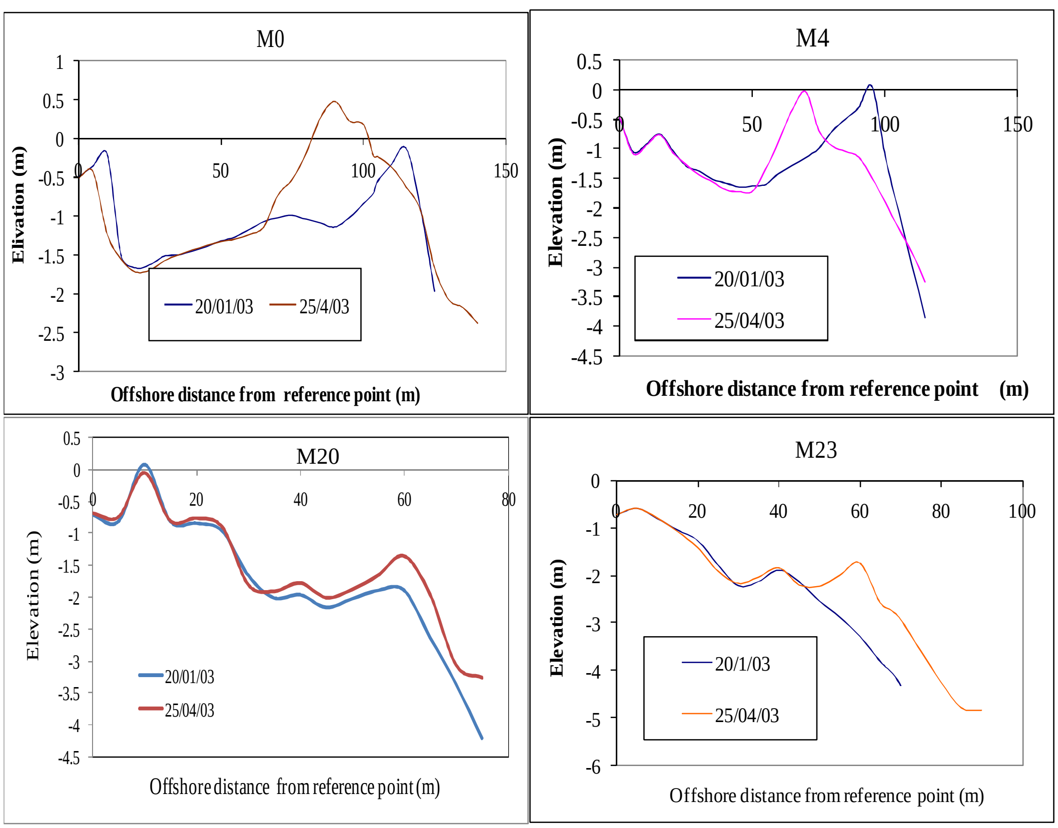


Fig. 4.11 Beach profiles in the northern sector of Muthalapozhi inlet

Table 4.4 Beach volume changes at different seasons of 2004 in the northern sector of Muthalapozhi

Sl. No.	Stations	Period	Beach Volume change (m^3/m)	Remarks
1.	M_0	20/01/03-25/4/03	32.40	Accretion
		20/01/03-30/08/03	–35.08	Erosion
2.	M_4	20/01/03-25/04/03	No data	–
		20/01/03-30/08/03	No data	–
3.	M_{20}	20/01/03-25/04/03	13.37	Accretion
		20/01/03-30/08/03	No data	–
4.	M_{23}	20/01/03-25/04/03	30.08	Accretion
		20/01/03-30/08/03	No data	–

4.4.2.2 Muthalapozhi south

Accretion is prevailing in the southern sector near to the breakwater (Sajeev, 1993), which may due to the groin effect of the breakwater. No frontal beach is available at a

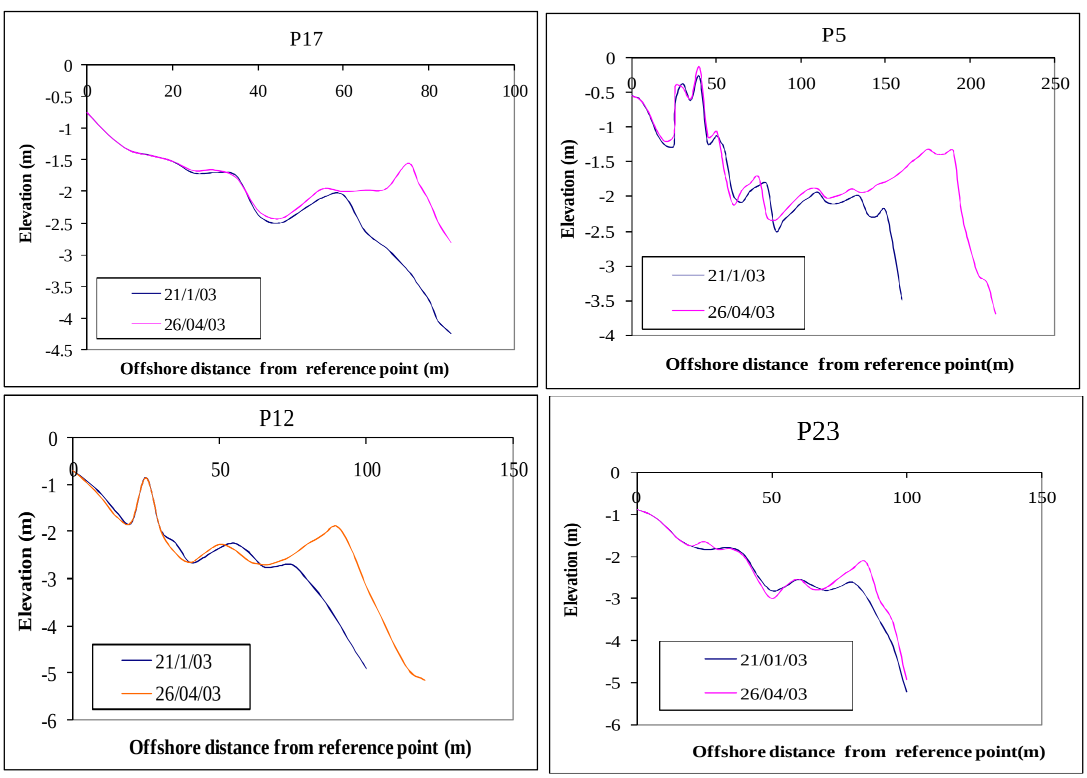


Fig. 4.12 Beach profiles in the southern sector of Muthalapozhi inlet

few stations during monsoon season and hence no profile data were taken at these stations during this season. Beach profiles of selected locations are presented in Fig. 4.12. The beach volume changes were computed for selected stations (P5, P12, P17 and P23) and presented in the Table 4.5.

Table 4.5 Beach volume changes at different seasons of 2004 in the southern sector of Muthalapozhi

Sl. No.	Stations	Period	Beach Volume change (m^3/m)	Remarks
1.	P5	21/01/03-26/04/03	8.98	Accretion
		21/01/03-30/08/03	No data	–
2.	P12	21/01/03-26/04/03	33.40	Accretion
		21/01/03-30/08/03	No data	–
3.	P17	21/01/03-26/04/03	46.20	Accretion
		21/01/03-30/08/03	No data	–
4.	P23	21/01/03-26/04/03	11.20	Accretion
		21/01/03-30/08/03	No data	–

4.4.3 Sediment characteristics

Since geomorphological and hydrodynamic conditions of Muthalapozhi coast are similar to Trivandrum coast, the sediment characteristics are also similar. The sediments are composed of medium sand, moderately well sorted; fine skewed and leptokurtic in nature.

Table 4.6 Average beach sediment characteristics at different locations of Muthalapozhi coast

Sl. No.	Locations	Period	Beach Location	Mean (mm)
1.	Muthalapozhi south	April 2008	Berm crest	0.33
			Beach face	0.31
2.	Muthalapozhi north	April 2008	Berm crest	0.43
			Beach face	0.73

4.5 Kayamkulam Inlet

The study area on both the sides of the Kayamkulam inlet covers about 16 km. The narrow strip of land that separates the sea from the backwater (Fig. 4.13) is undergoing several environmental problems. Hydrodynamic measurements, beach profiling, sediment data collection and littoral environmental observations pertaining to different seasons have been carried out during January 2004 – December 2004.

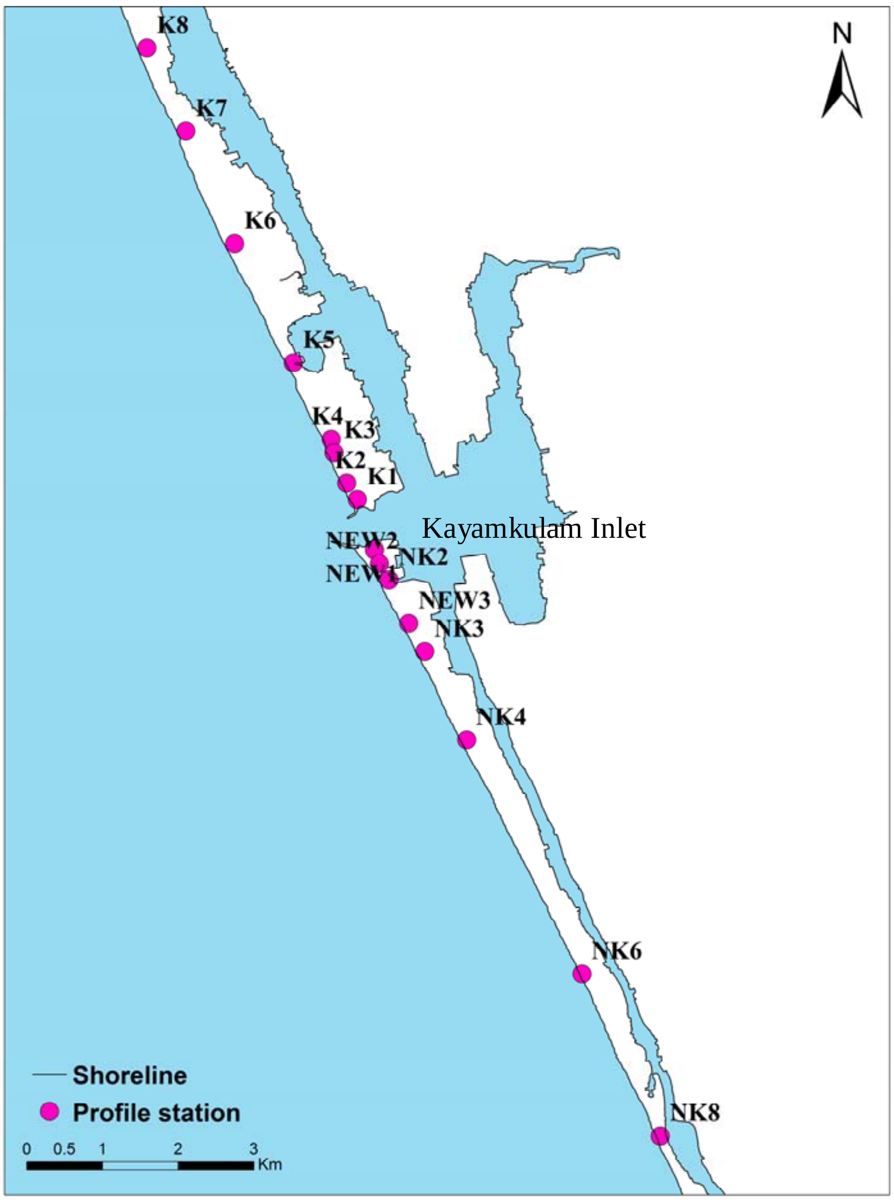


Fig. 4.13 Location map with beach profile stations on both the sides of Kayamkulam inlet

4.5.1 Wave characteristics

A schematic diagram showing the offshore instrumentation for measurement of waves is given in Fig. 4.14. The deployments were carried out at a station of 8 m depth off Srayikkad on the southern side of the Kayamkulam inlet in the pre-monsoon season. The statistics of wave parameters for the pre-monsoon are discussed in the sections below.

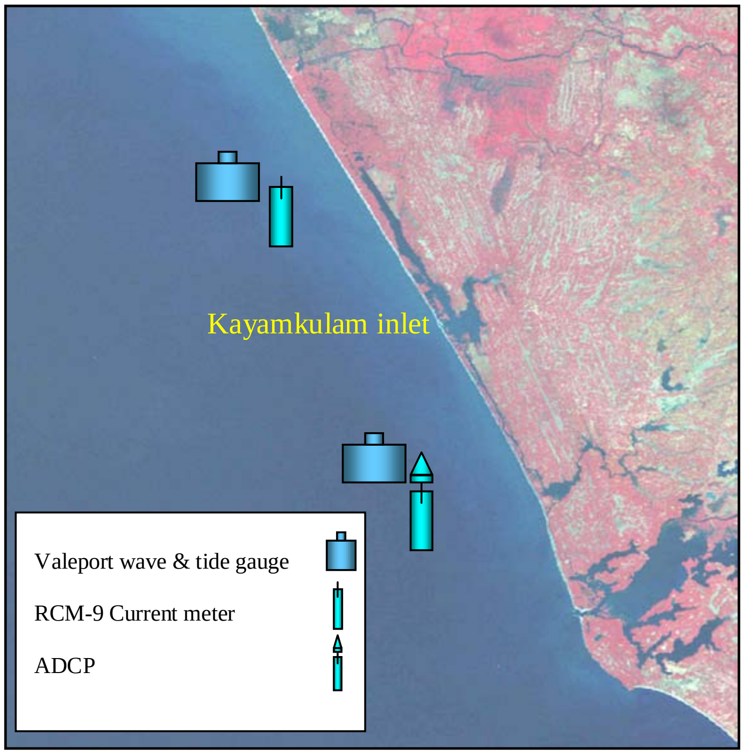


Fig. 4.14 Sites of offshore deployments off Kayamkulam-Thrikkunnapuzha coast

4.5.1.1 Wave height (H_s)

The time series of H_s during the measurement period 27-02-04 to 19-03-04, which is representative of pre-monsoon season are presented in Fig. 4.15. In this period, moderate wave activity which is characteristic of pre-monsoon is observed. The H_s observed in this period is in the range 0.38-0.80 m.

4.5.1.2 Wave period (T_z)

As discussed in the earlier sections zero crossing periods (T_z) is used for numerical modelling study. T_z observed during the period (27-02-04 to 19-03-04) is represented as time series plot in Fig. 4.18. The T_z falls in the range 3.5 to 9.5s with standard deviation of 1.33. Since the measurement period pertained to the early period of pre-monsooon season, long period swells typical of pre-monsoon are absent.

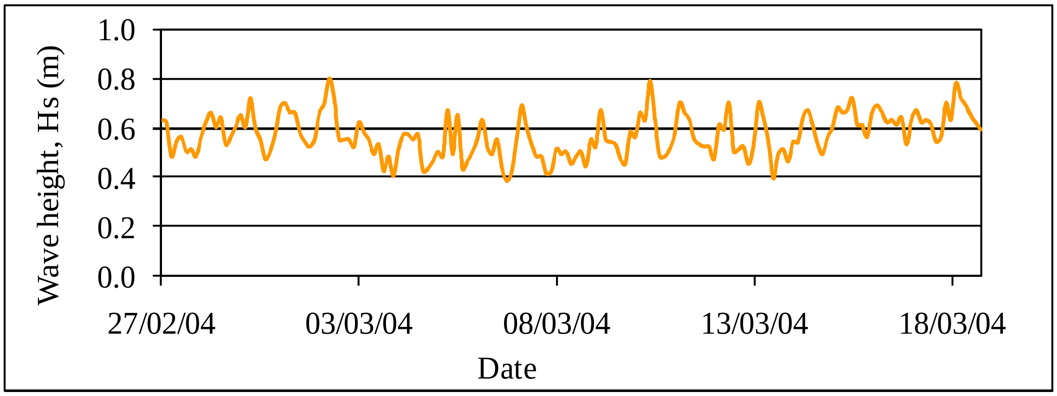


Fig.4.15 Time series of significant wave height off Kayamkulam during March 2004

Table 4.7 Nearshore wave statistics off Kayamkulam during pre-monsoon

Parameters	Period	Min.	Max.	Mean	Standard Div.
Wave Height (H_s), m	Pre-monsoon (27-02-04 to 19-03-04)	0.38	0.80	0.57	0.08
Wave Period, (T_z) sec		3.5	9.5	5.92	1.33
Wave direction, °N		117	342	232.4	32.0

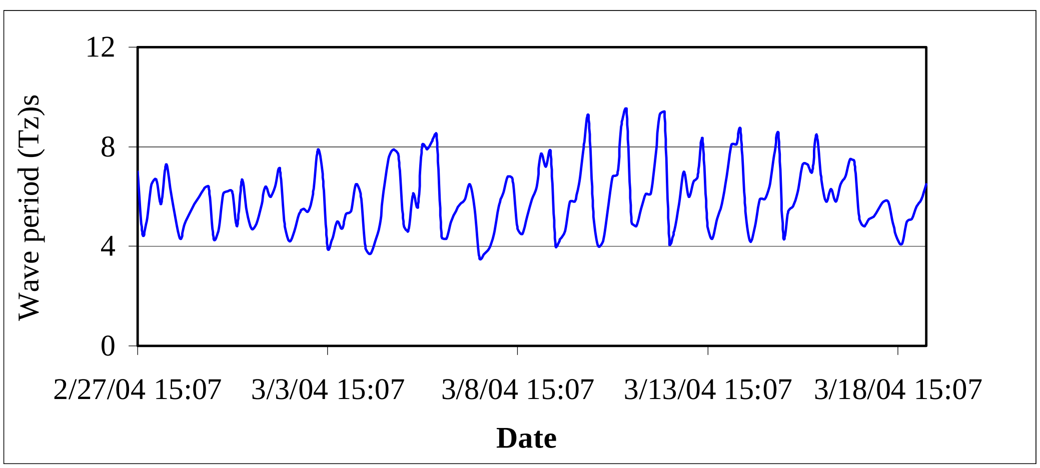


Fig. 4.16 Time series of wave period (T_z) off Kayamkulam during March 2004

4.5.1.3 Wave direction

The time series of the peak mean directions are presented in Fig. 4.17. It can be seen from the time series that during pre monsoon, the average value of mean direction is 225^0 N (south west). The shoreline normal for this coast being around 240^0 N, it can be inferred that the predominant waves are south of the shore normal, indicating predominant northerly transport.

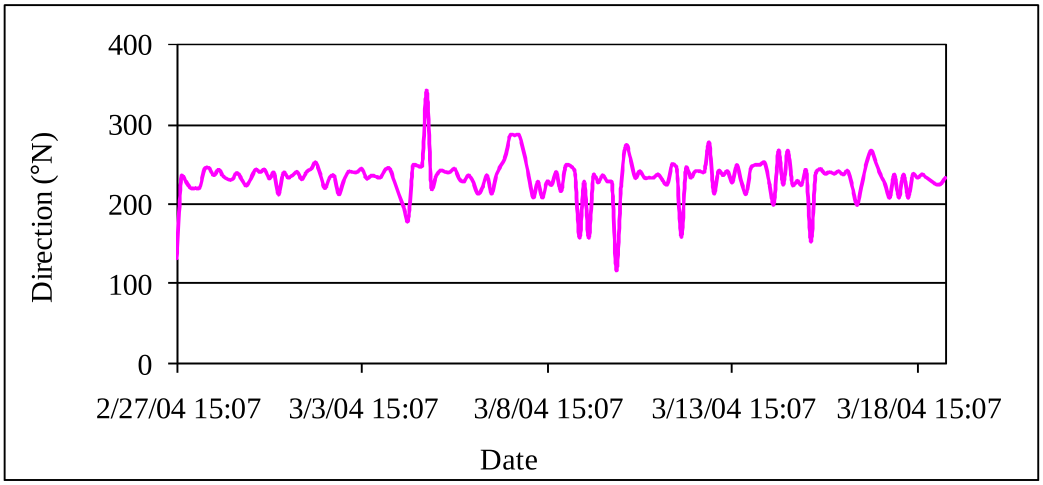


Fig. 4.17 Time series of peak mean wave direction off Kayamkulam during March 2004

4.5.2 Beach profiles

A major portion of the shoreline was fronted by sea wall with no frontal beach during monsoon. As per the field observation the beach has maximum width during the fair weather months. High erosion is observed during the months mid-May-June-July, when the wave intensity is at its maximum

There were 8 stations located at regular interval on the southern side of the inlet. During the study period construction of the breakwater was in progresses. During fair weather period the beach was having maximum width close to the southern breakwater, which tapered down further south. This was also reflected in beach profiles. Some of the profiles of selected stations are given in Fig. 4.28.

Northern sector of Kayamkulam coast is identified as one of the critically eroding sectors of southwest coast of India. Due to the construction of breakwaters this sector of the coast is deprived of sediments transported north ward by the predominant northerly longshore current and hence severe erosion is prevailing in the northern side of the Kayamkulam inlet. Beach profiles measured at 8 stations in this sector reflect the eroding nature of the coast. Some of the profiles of selected stations are given in Fig. 4.19.

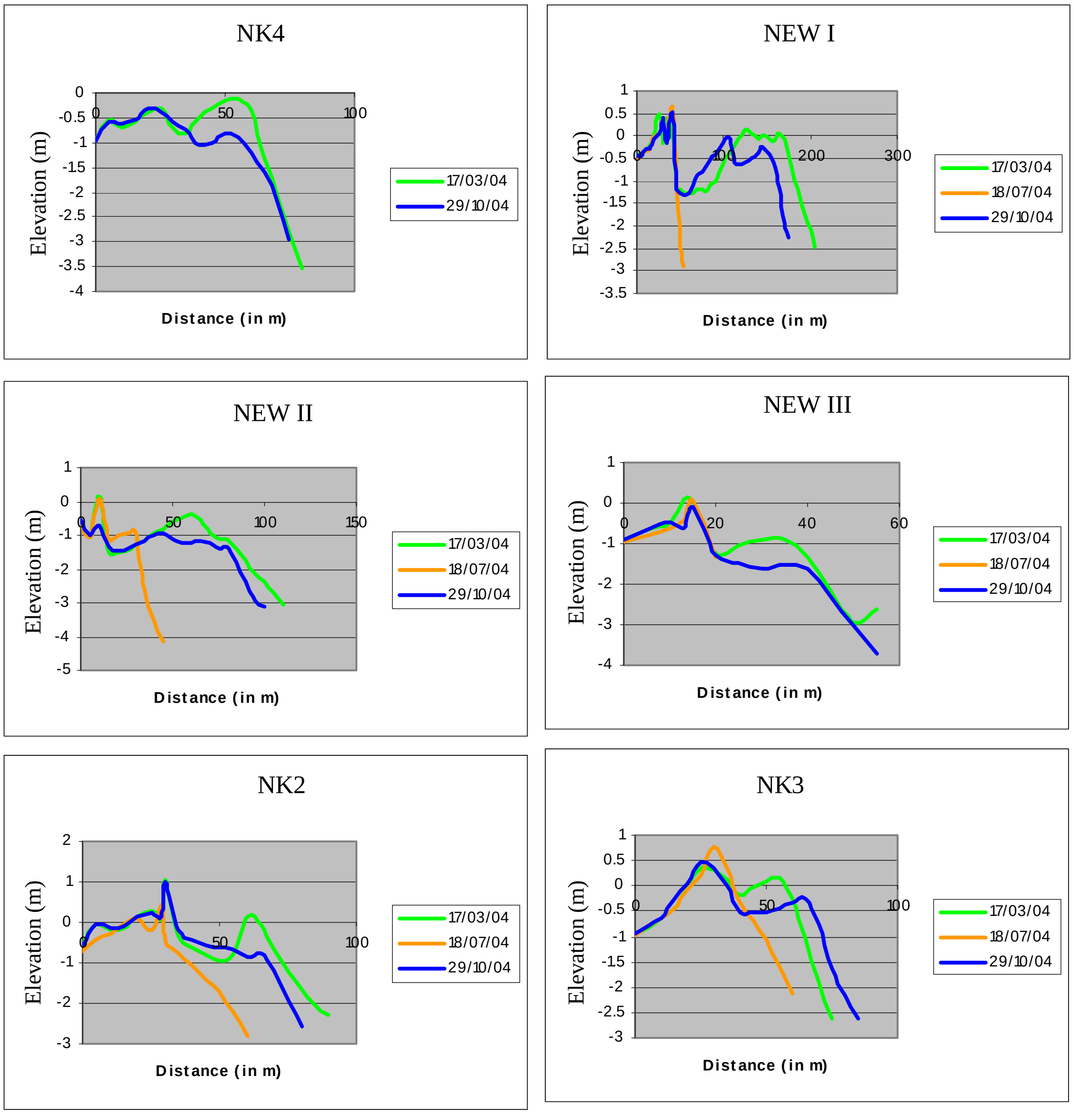


Fig. 4.18 Beach profiles at stations in the southern sector of Kayamkulam inlet during different seasons

The beach volume changes during different seasons are computed and given in the Table 4.8 and Fig. 4.20. In general, significant erosional tendency is observed with the onset of monsoon in June. Though beach build up tendency starts in the month of

July, it is often interrupted during the later period of monsoon. However, accretion starts in the post-monsoon and continues till March. As expected, there is noteworthy fluctuation between the stations and not all stations behave identically, which can be explained by the effects of nearshore circulation, presence of sea walls and the influence of construction of breakwater.

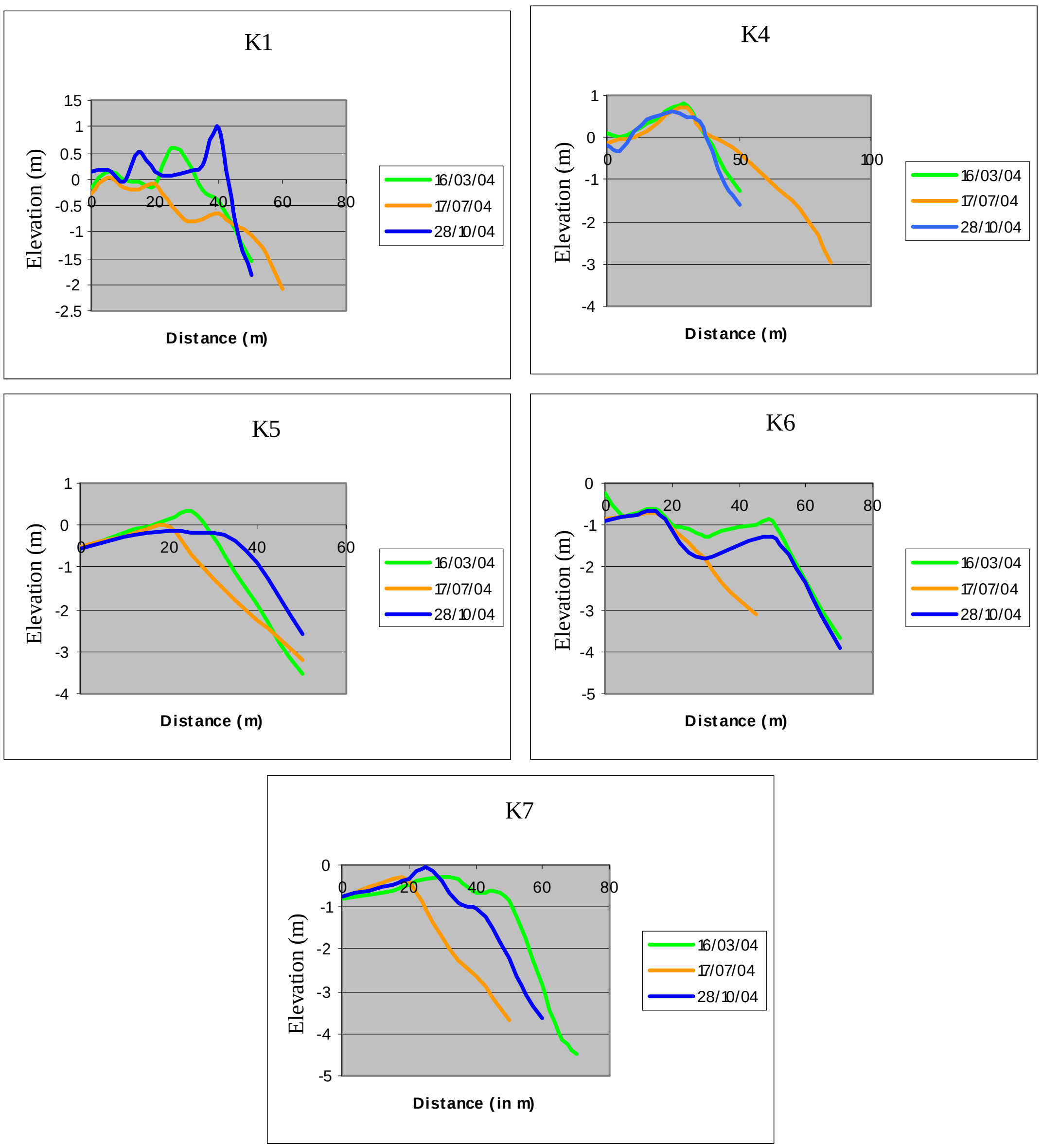


Fig. 4.19 Beach profiles at stations in the northern sector of Kayamkulam inlet during different seasons

Table 4.8 Beach volume changes at different stations in the sectors north and south of Kayamkulam inlet during different seasons of 2004

Sl. No.	Stations	Volume Changes			
		During 16.03.04-17.07.04 (m^3/m)	Remarks	During 16.03.04-28.10.04 (m^3/m)	Remarks
1	K 1	-5.53	Erosion	-1.43	Erosion
2	K 4	54.81	Deposition	-14.93	Erosion
3	K 5	-14.5	Erosion	15.29	Deposition
4	K 6	-73.36	Erosion	-19.43	Erosion
5	K 7	-85.36	Erosion	-25.83	Erosion
6	NEW I	-1810.35	Erosion	-79.70	Erosion
7	NEW II	-458.06	Erosion	-38.77	Erosion
8	NEW III	-157.25	Erosion	-16.76	Erosion
9	NK 2	-137.09	Erosion	-23.12	Erosion
10	NK 3	-53.05	Erosion	22.26	Deposition
11	NK 4	-19.64	Erosion	No data	No data

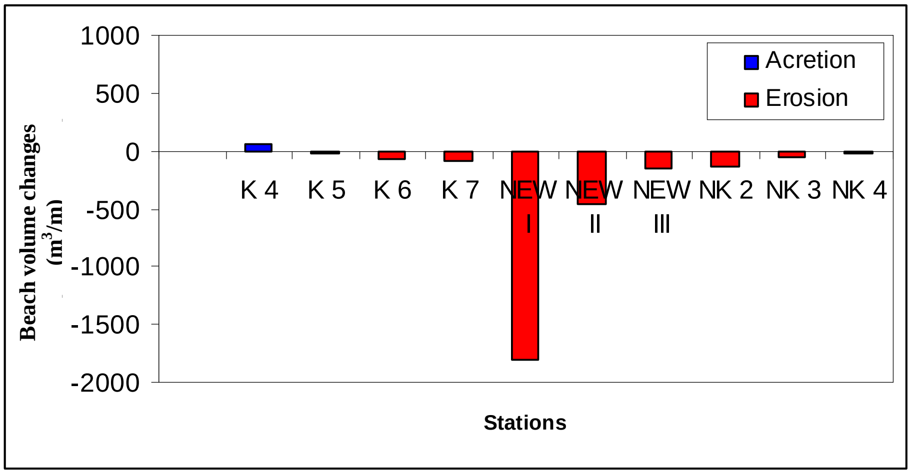


Fig. 4.20 Beach volume changes (m^3/m) from pre-monsoon to monsoon of 2004 at different stations of Kayamkulam inlet area

4.5.3 Sediment characteristics

The beach sediment characteristics for different seasons are presented in Table 4.9. Samples could not be collected during monsoon from the beach face, as the frontal beach was almost absent and waves were plunging at the berm crest. It is seen that the sediments are mostly fine sand with a mean size ranging from 0.08 to 0.20 mm during the different seasons. Berm crest and beach face have more or less same values. The sediments are finer in the post-monsoon season when compared to the pre- and monsoon

Table 4.9 Mean beach sediment characteristics for the Kayamkulam inlet sector

Station and Date of sampling		**Beach Loc.**	**Mean Size**	
			Phi	**mm**
Pre-monsoon	17.03.04	Berm crest	2.33	0.20
		Beach face	2.42	0.19
Monsoon	12.08.04	Berm crest	2.37	0.19
		No Beach face samples		
Post-monsoon	17.11.04	Berm crest	3.56	0.08
		Beach face	3.46	0.09

4.6 Trikkunnapuzha

Trikkunnapuzha located about 6 km north of Kayamkulam inlet (Fig. 4.21) is one of the study areas. As mentioned in the previous sections, erosion is high north of the Kayamkulam inlet and Trikkunnapuzha sector is further north of that eroding zone. During the monsoon season the erosion is high along this coastline and there is no frontal beach in this sector during monsoon. The environmental and hydrodynamic conditions are similar to the Kayamkulam inlet sector.

4.6.1 Wave characteristics

The hydrodynamic data were collected off Trikkunnapuzha at 8 m depth (see Fig. 4.14) during 27.02.04 to 20.03.04 in pre-monsoon season, 13.07.2004 to 13.08.2004 in monsoon season and 27.10.04 to 18.11.04 in post-monsoon season.

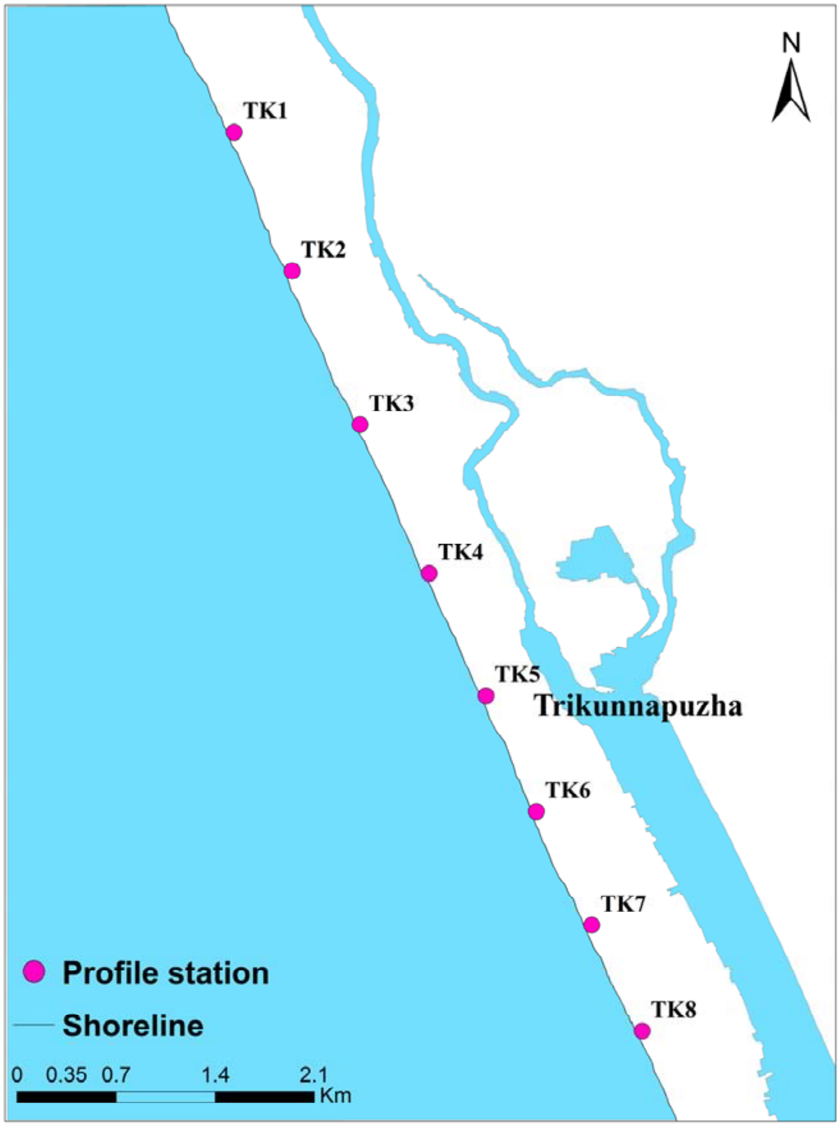


Fig. 4.21 Location map of Trikkunnapuzha study area.

4.6.1.1 Wave height (H_s)

The wave statistics and time series data for different seasons are given in the Table 4.10.and Fig. 4.22. The significant wave height (H_s) at Trikkunnapuzha during pre-monsoon ranges from 0.29 to 0.83 m with a mean of 0.51 m and standard deviation 0.1, indicating a very narrow band of wave heights. During monsoon period, wave heights shift to higher values, with a minimum of 0.86 m and maximum of 2.06 m with mean of 1.43 m, indicating rough sea. Standard deviation of significant wave height during monsoon season is 0.26 indicating a wider spreading of wave heights compared to pre-monsoon. During post-monsoon period, moderate wave activity comparable to pre-monsoon was recorded. The significant wave heights vary between 0.34 and 0.97 m, with a mean of 0.57 m and standard deviation of 0.15.

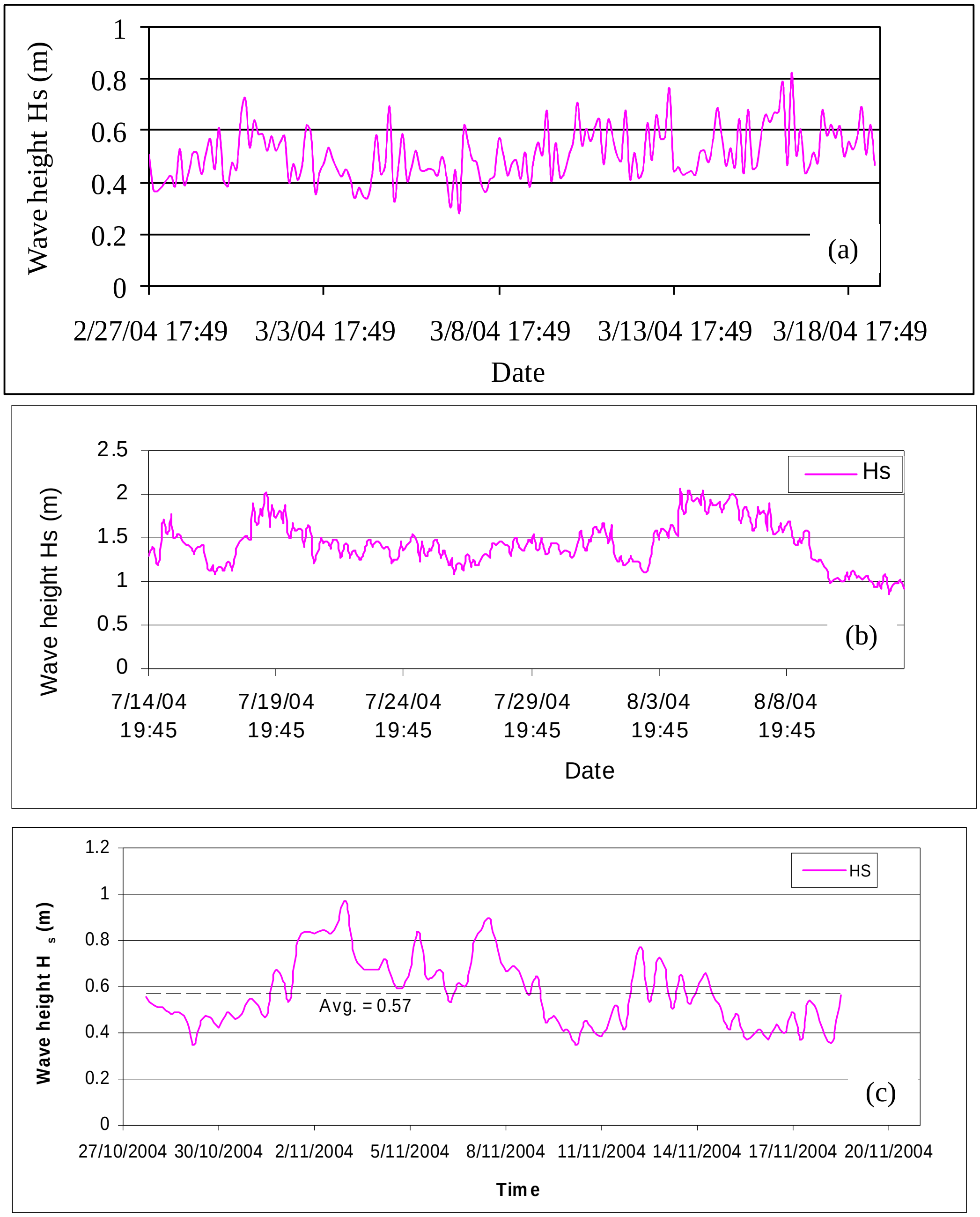


Fig. 4.22 Time series of H_s off Trikkunnapuzha during (a) pre-monsoon 2004, (b) monsoon 2004 and (c) post-monsoon 2004

Table 4.10 Nearshore wave statistics off Trikkunnapuzha during different seasons of 2004

Parameters	Period	Min.	Max.	Mean	Standard Div.
Wave Height (H_s), m	Pre-monsoon (27.02.04 – 20.03.04)	0.29	0.83	0.51	0.1
Wave Period, (T_z) s		4.20	13.0	8.6	1.6
Wave direction, ◦N		-	-	-	-
Wave Height (H_s), m	Monsoon (13.07.04 – 13.08.04)	0.86	2.06	1.43	0.26
Wave Period, (T_z) s		6.08	9.78	7.96	0.63
Wave direction, ◦N		-	-	-	-
Wave Height (H_s), m	Post-monsoon (27.10.04 – 18.11.04)	0.35	0.97	0.57	0.14
Wave Period, (T_z) s		6.02	11.4	8.89	1.07
Wave direction, °N		143.5	245	200	16.9

4.6.1.2 Wave period (T_z)

The wave period statistics and time series data are given in the Table 4.10 and Fig. 4.23. The zero crossing periods (T_z) for pre-monsoon range from 4 s to a maximum of 13 s with an average of 8.6 s and standard deviation of 1.6. The data show that, swell waves are prevailing during this period. During monsoon the T_z shifts to the range 6.1 to 9.8 s with mean of 8.0 s and standard deviation is 0.63 indicating a narrow spreading of wave period. During post–monsoon, T_z varies between 6 and 11.4 s, with mean of 8.8 s. The standard deviation is 1.07, which is higher than monsoon season. During this period long period swell waves are prevailing like pre-monsoon period.

4.6.1.3 Wave direction

The time series data on wave direction is available only for post-monsoon season (Fig. 4.24). The mean wave direction is in the range 143-245 °N, with a mean wave of 200ºN, i.e., SSW direction.

4.6.2 Beach profiles

Beach profile data were collected at 8 stations simultaneous with offshore deployment and retrieval of equipments during different seasons of 2004. As expected the profiles reach the nadir during the monsoon, when the wave intensity is at its maximum. The profile survey could not be carried out at some stations during monsoon because of intense erosion of beach up to sea wall. Selected profiles at different stations are given in Fig. 4.25.

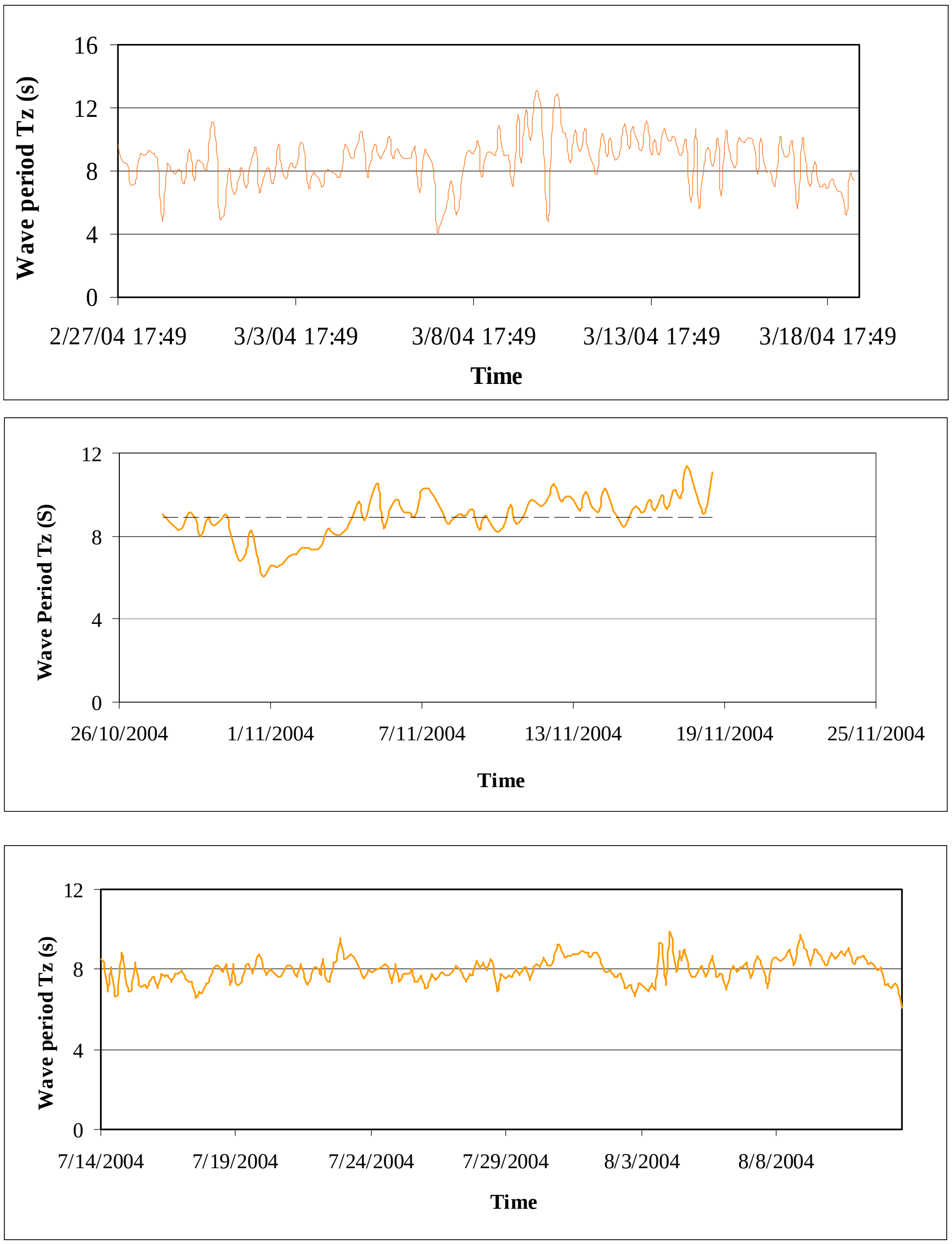


Fig. 4.23 Time series of T_z off Trikkunnapuzha during (a) pre-monsoon 2004, (b) monsoon 2004 and (c) post-monsoon 2004

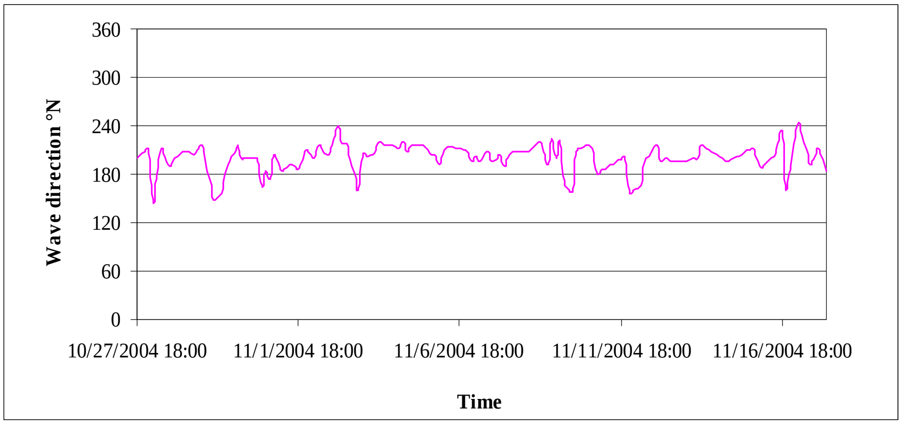


Fig. 4.24 Time series of mean wave direction off Trikkunnapuzha during post-monsoon 2004

4.6.2.1 Beach volume changes

The beach volume changes are presented in the Table. 4.11 and Fig. 4.26. Significant erosional tendency is observed during the pre-monsoon months of April and May and it picks up with the onset of monsoon in June. Beach build up tendency starts in the month of July, but is often interrupted during monsoon and continues till March. The stations TK2, TK3 and TK4 are different from the above pattern with net deposition even in monsoon; this may due to the mud bank phenomena seen during monsoon season. No frontal beach is available in certain stations (TK4, TK5 and TK8) due to monsoonal erosion. As expected, there is noteworthy fluctuation between the stations and not all stations behave identically, which can be explained by the effects of nearshore circulation, presence of sea walls and the influence of the breakwater construction at Kayamkulam inlet.

Table 4.11 Beach volume changes at Trikkunnapuzha sector during 2004

Sl. No	Stations	Volume Changes			
		During 03.03.04-18.07.04 (m^3/m)	Remark	During 03.03.04-28.10.04 (m^3/m)	Remark
1	TK 1	-5.18	Erosion	-0.71	Erosion
2	TK 2	5.37	Deposition	3.46	Deposition
3	TK 3	10.36	Deposition	12.95	Deposition
4	TK 4	22.79	Deposition	No data	No data
5	TK 5	-9.61	Erosion	No data	No data
6	TK 6	-45.66	Erosion	14.59	Deposition
7	TK 7	-3.13	Erosion	-2.14	Erosion
8	TK 8	-19.04	Erosion	No data	No data

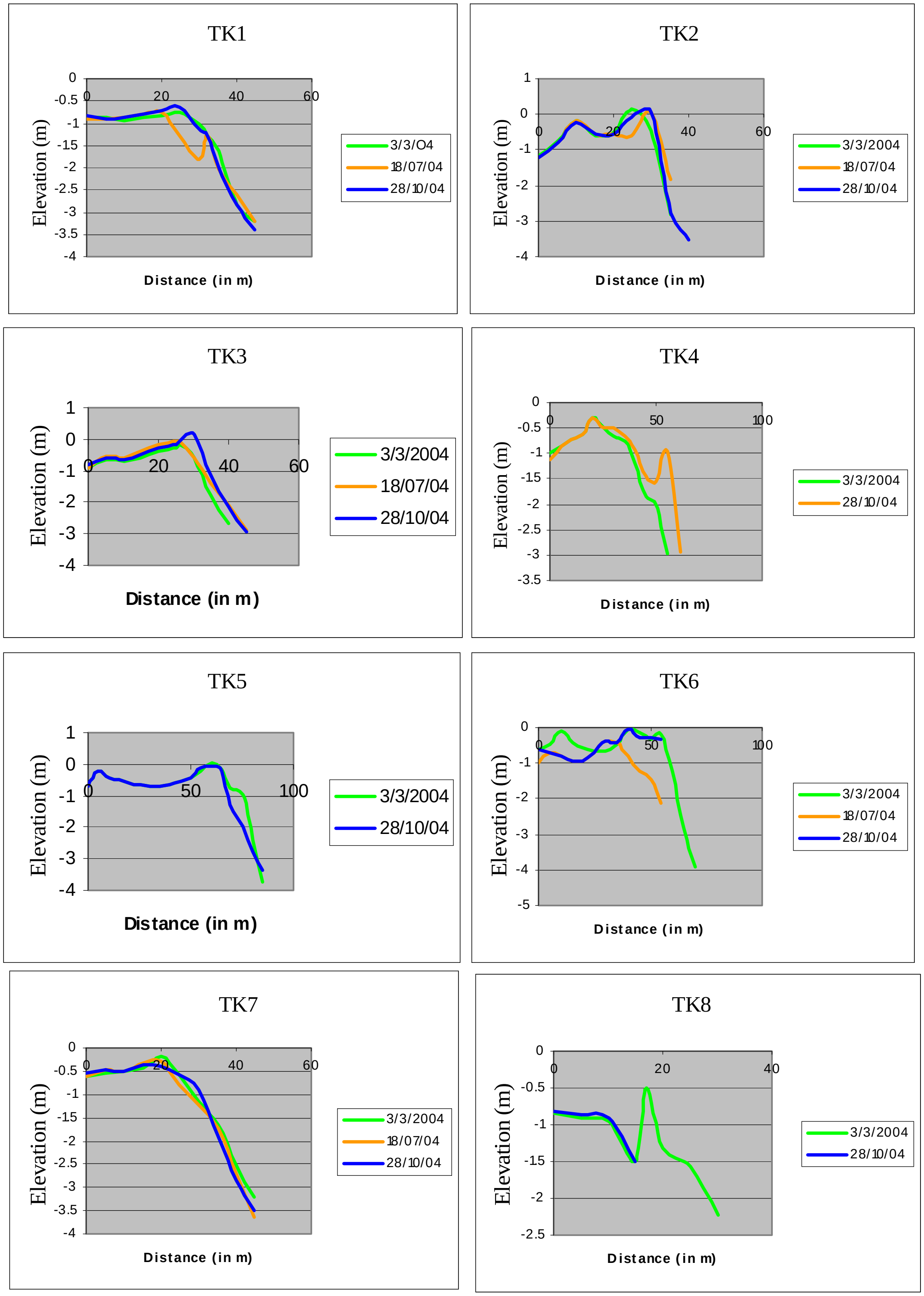


Fig. 4.25 Beach profiles at different stations of Trikkunnapuzha sector during different seasons of 2004

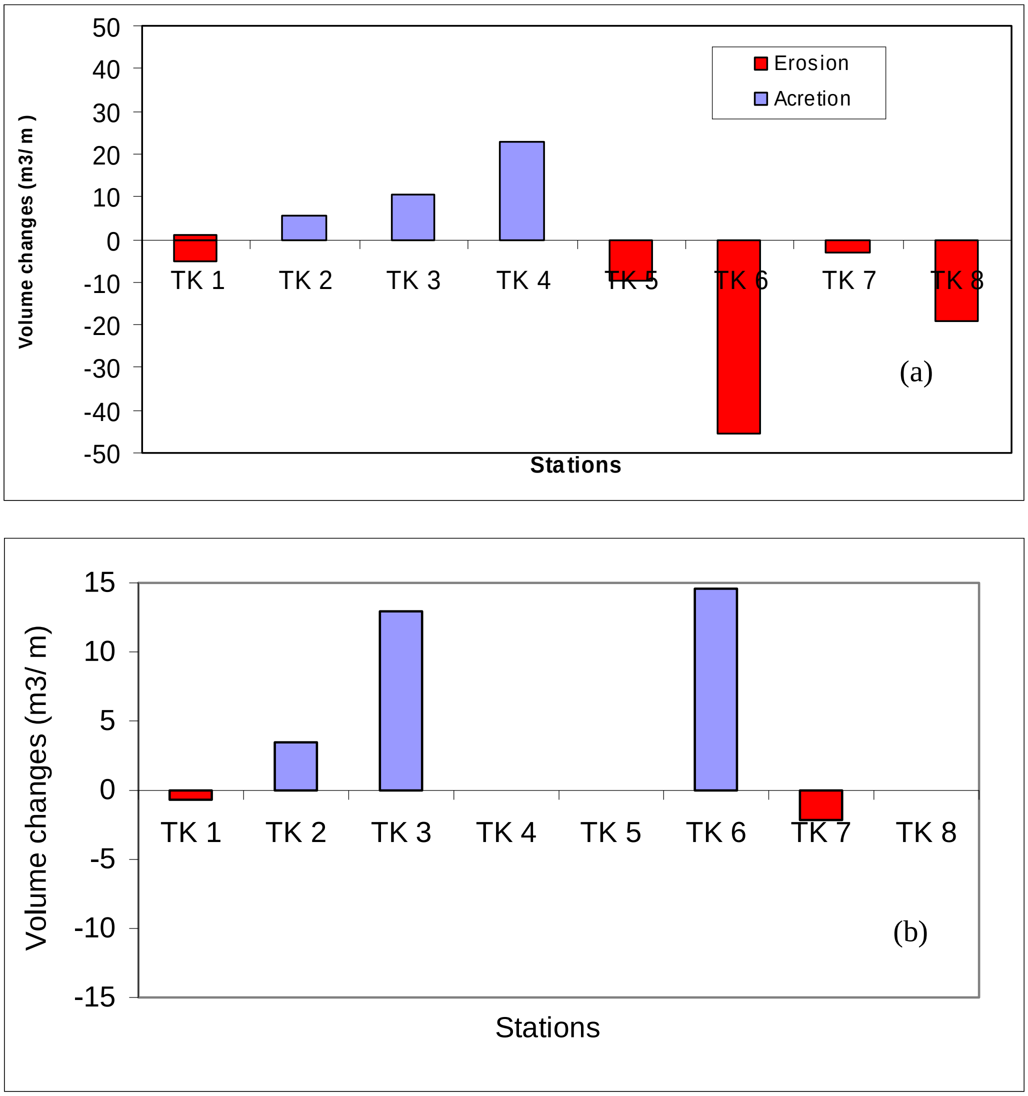


Fig. 4.26 Beach volume changes at Trikkunnapuzha (a) from pre-monsoon to monsoon and (b) from pre-monsoon to post-monsoon during 2004

4.6.3 Sediment characteristics

The sediment characteristics are presented in Table 4.12. Sediment samples could not be collected during monsoon from beach face, as the frontal beach was almost absent and waves were plunging at the berm crest. The sediments are mostly fine sand with a mean size ranging from 0.08 to 0.20 mm. The sediments are fine grained throughout the year. Samples from berm crest and beach face show more or less same values in both the seasons. The sediments are finer in the post-monsoon season when compared to the pre- and monsoon seasons.

Table 4.12 Mean grain size parameters of the beach sediments at Trikkunnapuzha during different seasons

Station and Date of sampling		Beach Loc.	Mean Size	
			(Phi)	(mm)
Pre-monsoon	Trikkunnapuzha 17.03.04	Berm crest	2.33	0.20
		Beach face	2.42	0.19
Monsoon	Trikkunnapuzha 12.08.04	Berm crest	2.37	0.19
		No Beach face samples		
Post-monsoon	Trikkunnapuzha 17.11.04	Berm crest	3.56	0.08
		Beach face	3.46	0.09

4.7 Mararikkulam

Mararikkulam, about 10 km north of Alleppey has a slightly different coastal setting when compared to Trikkunnapuzha. The inner shelf of this coast is more gentle and hence the wave intensity is expected to be lesser. This is one location with a very wide beach. The hydrodynamic, sedimentological and beach profile measurements (see Fig. 4.27 and Fig. 4.28) at this location were carried out during 27.02.04 – 20.03.04 in pre-monsoon, 13.07.04 – 13.08.04 in monsoon and 27.10.04 – 18.118.04 in post-monsoon season.

4.7.1 Wave characteristics

4.7.1.1 Wave height (H_s)

The time series of H_s and wave statistics off Mararikkulam are presented in Fig 4.29 and Table 4.13 respectively. The significant wave height (H_s) for the pre-monsoon season ranges from 0.26 m to 0.69 m with an mean value of 0.46 m, which indicates a very low wave height (Fig. 4.29). The standard deviation is estimated to be 0.094, indicating very narrow band of wave heights. During monsoon period, wave heights have shifted to higher values as expected, with minimum H_s of 0.85 m, which is much more than the maximum during the pre-monsoon period, and a maximum of 2.27 m. The mean value of H_s during monsoon deployment period is 1.36 m, with a standard deviation of 0.29 indicating a wider spreading of wave height than during the pre-monsoon time. During post-monsoon, the H_s vary between 0.3 m and 0.9 m, with a mean value of 0.53 m and standard deviation of 0.14.

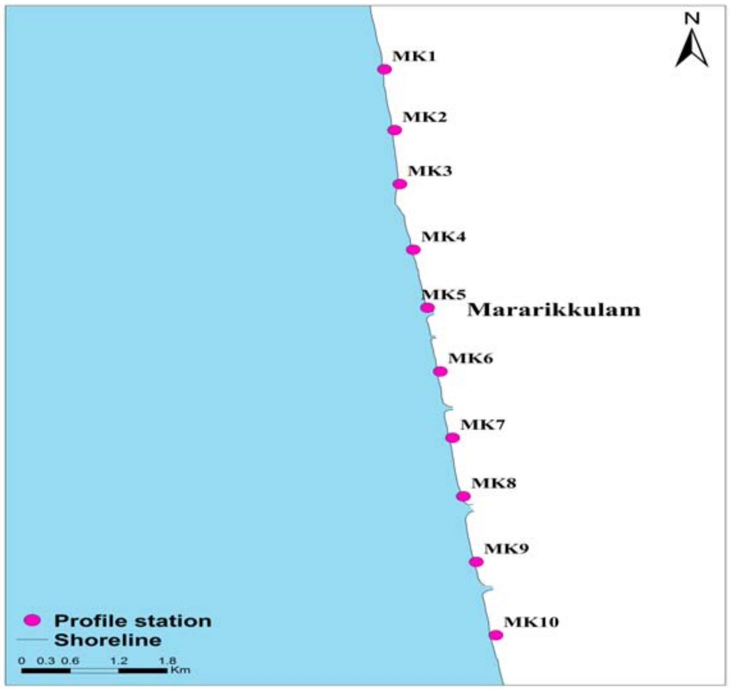


Fig. 4.27 Location map of Mararikkulam study area

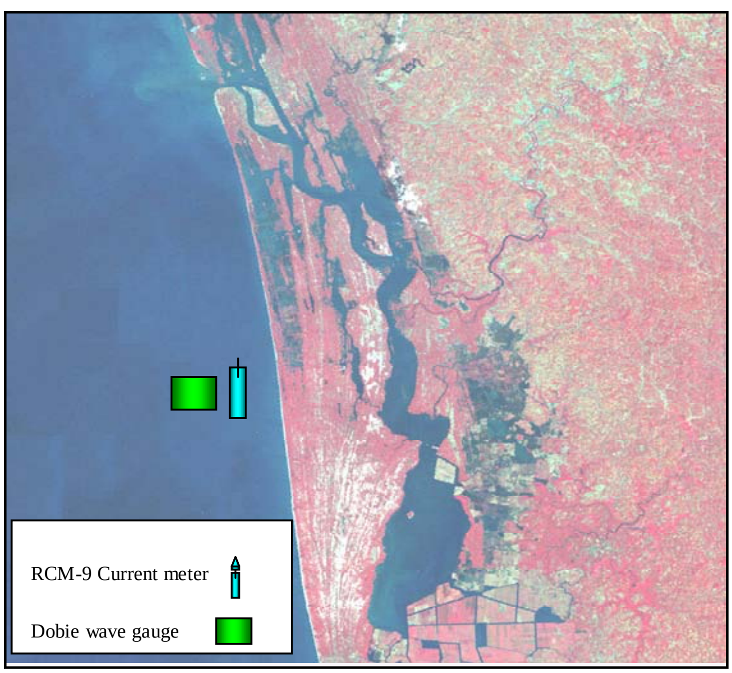


Fig. 4.28 Scheme of offshore deployments off Mararikkulam coast

Table 4.13 Nearshore wave statistics off Mararikkulam during different seasons of 2004

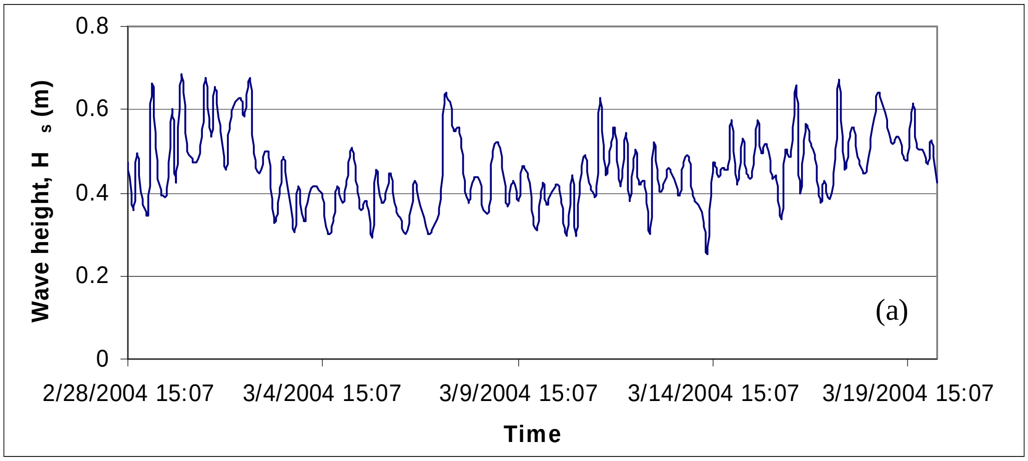
Parameters	Period	Min.	Max.	Mean	Standard Div.
Wave Height (H_s), m	Pre-monsoon (27.02.04 – 20.03.04)	0.26	0.69	0.46	0.09
Wave Period, (T_z) s		4.05	15.2	7.77	1.7
Wave direction,°N		174.7	299.8	233.3	15.5
Wave Height (H_s), m	Monsoon (13.07.04 – 13.08.04)	0.85	2.27	1.36	0.29
Wave Period, (T_z) s		6.6	9.6	8.4	0.51
Wave direction,°N		-	-	-	-
Wave Height (H_s), m	Post-monsoon (27.10.04 – 18.11.04)	0.30	0.90	0.53	0.14
Wave Period, (T_z) s		4.05	12.3	8.7	1.6
Wave direction,°N		-	-	-	-

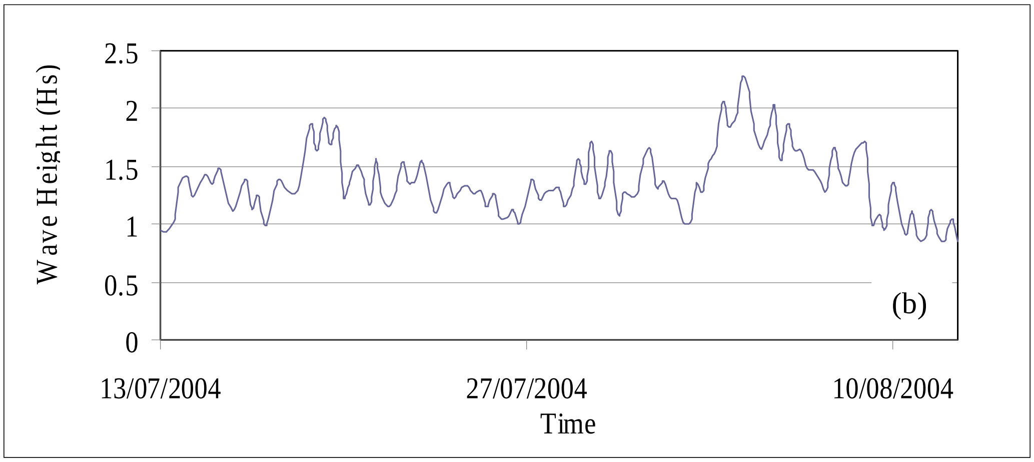
4.7.1.2 Wave period (T_z)

The zero crossing wave period, T_z (Fig. 4.30) for pre-monsoon ranges from 4 s to a maximum of 15 s with mean of 7.8 s and standard deviation of 1.7. During monsoon the T_z ranges from 6.6 s to 9.6 s. The mean value during monsoon season is estimated to be 8.4 s with a standard deviation of 0.51 indicating a narrow spreading of wave period. During post-monsoon, T_z varies between 4 s and 12 s, with mean value of 8.7 s with a standard deviation of 1.6. The recorded high wave period and larger standard deviation for pre-monsoon and post-monsoon indicates that long period waves are present during these seasons.

4.7.1.3 Wave direction

Wave directions are measured only for pre-monsoon period at Mararikulam. The time series of the peak wave directions during pre-monsoon are presented in Fig. 4.31. Pre-monsoon record of wave direction at Mararikulam shows that the mean wave direction is almost same as that of Trikkunnapuzha, but it is more oblique here because the shore normal is around 255^0N.





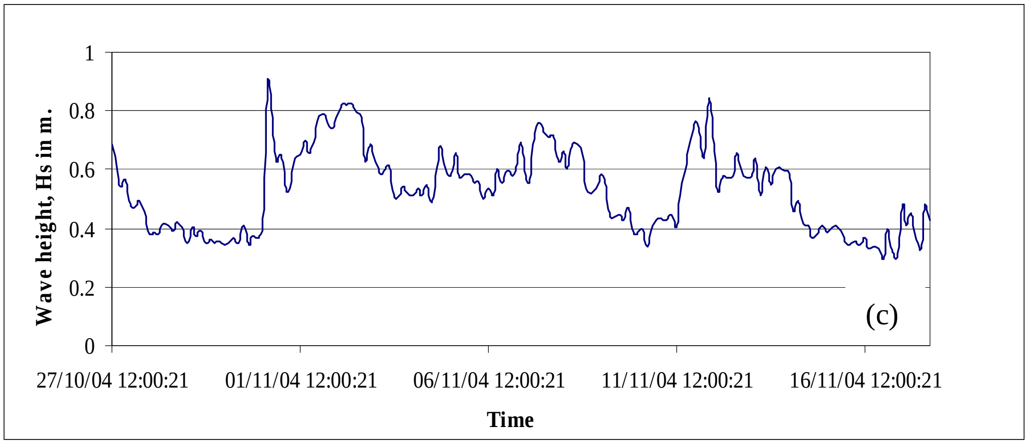


Fig. 4.29 Time series of H_s off Mararikkulam during(a) pre-monsoon (b) monsoon and (c) post-monsoon 2004

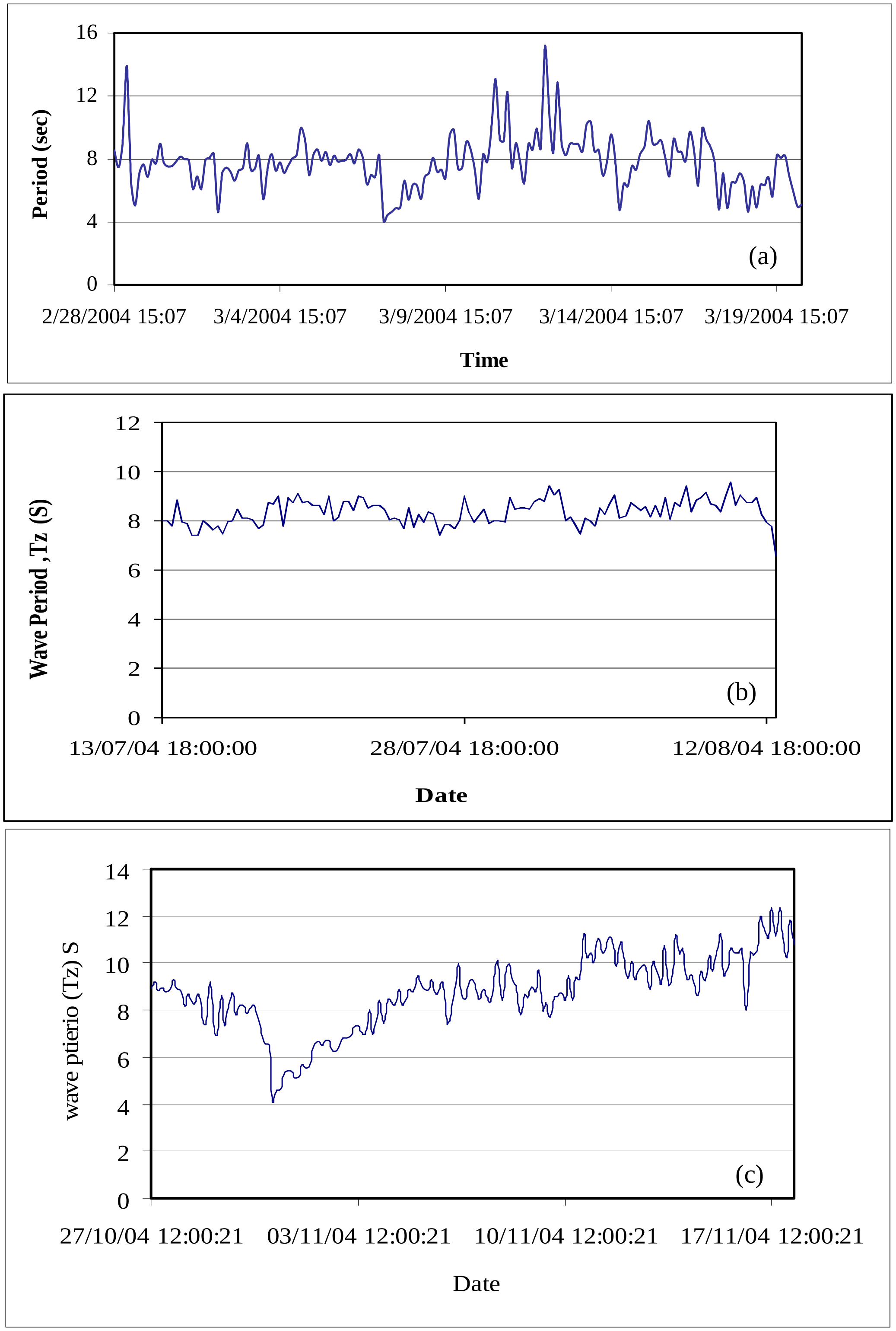


Fig. 4.30 Time series of T_z off Mararikkulam during (a) pre-monsoon, (b) monsoon and (c) post-monsoon 2004

Fig. 4.31 Time series of mean wave direction off Mararikkulam during pre-monsoon of 2004

4.7.2 Beach profiles

The seasonal beach profile data collected at 10 stations in this location during 2004 were analysed and the profiles are given in Fig. 4.32. As per the field observation the beach has maximum width during the fair weather months. As in the case of other locations, high erosion is observed during the monsoon months, when the wave intensity is at its maximum. Beach build up tendency starts in the month of July, but is often interrupted during monsoon and continues till March

Table 4.14 Seasonal beach volume changes at different stations of Mararikkulam sector during 2004

Sl. No.	Station	Volume Changes			
		During 02.03.04-16.07.04 (m^3/m)	**Remark**	**During 02.03.04-29.10.04 (m^3/m)**	**Remark**
1	MK 1	-32.49	Erosion	-10.53	Erosion
2	MK 2	46.48	Deposition	48.90	Deposition
3	MK 3	-181.88	Erosion	40.24	Deposition
4	MK 4	-0.76	Erosion	-53.75	Erosion
5	MK 5	-65.40	Erosion	-6.24	Erosion
6	MK 6	41.46	Deposition	-226.40	Erosion
7	MK 7	121.63	Deposition	175.13	Deposition
8	MK 8	30.48	Deposition	31.13	Deposition
9	MK 9	9.08	Deposition	51.88	Deposition
10	MK 10	-21.04	Erosion	-50.48	Erosion

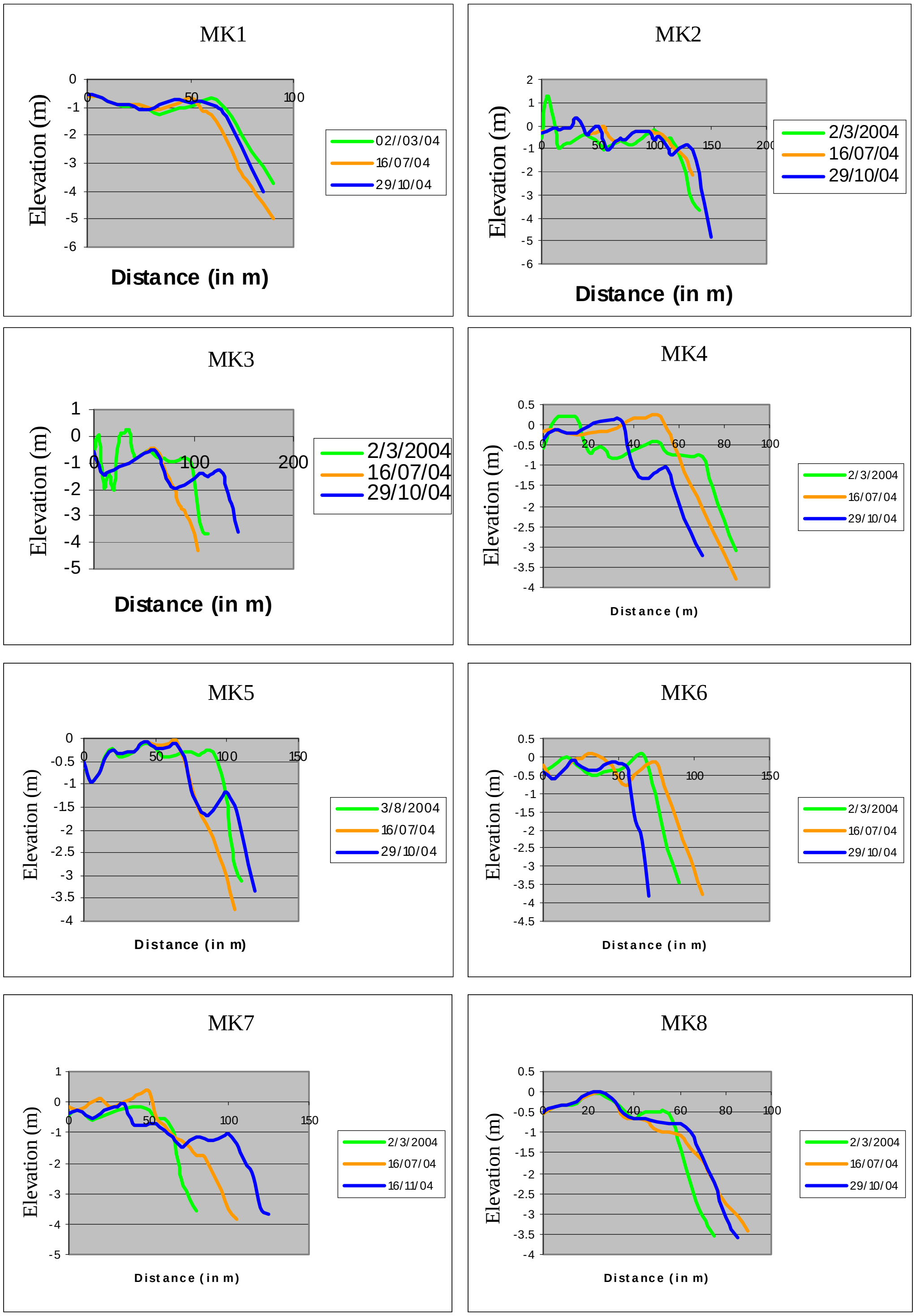


Fig. 4.32 Beach profiles in the Mararikkulam sector for different seasons of 2004

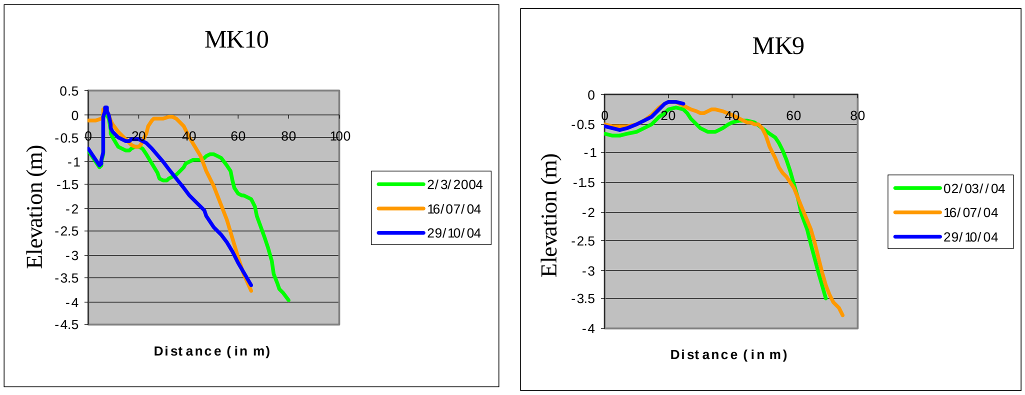


Fig. 4.32 Beach profiles in the Mararikkulam sector for different seasons of 2004 (Cont'd)

4.7.2.1 Beach volume changes

The beach volume changes between successive measurement dates are presented in Fig. 4.33 and Table 4.14 for the Mararikkulam sector. The results indicate the uniqueness of this coast with net deposition in stations MK2, MK7, MK8 and MK9 irrespective of seasons. This could be due to the occurrence of mud bank which protects the beach even during peak monsoon time. At other stations, the pattern is similar to what is normally observed with the beach yet to regain its pre-monsoon profile in October.

4.7.3 Sediment characteristics

The beach is composed of mostly fine sand with the mean size ranging between 0.13 and 0.30 mm (Table 4.15). Grain size for berm crest and beach face does not show much variation irrespective of the seasons. The grain size distribution shows a fining tendency from pre-monsoon to post-monsoon

Table 4.15 Grain size parameters of the beach sediments at Mararikkulam during different seasons

Station and Date of sampling		**Beach Loc.**	**Mean Size**	
			Phi	**mm**
Pre–monsoon	Mararikkulam 18.03.04	Berm crest	1.76	0.30
		Beach face	1.88	0.27
Monsoon	Mararikkulam 13.08.04	Berm crest	No data	
		Beach face		
Post–monsoon	Mararikkulam 18.11.04	Berm crest	2.85	0.14
		Beach face	2.98	0.13

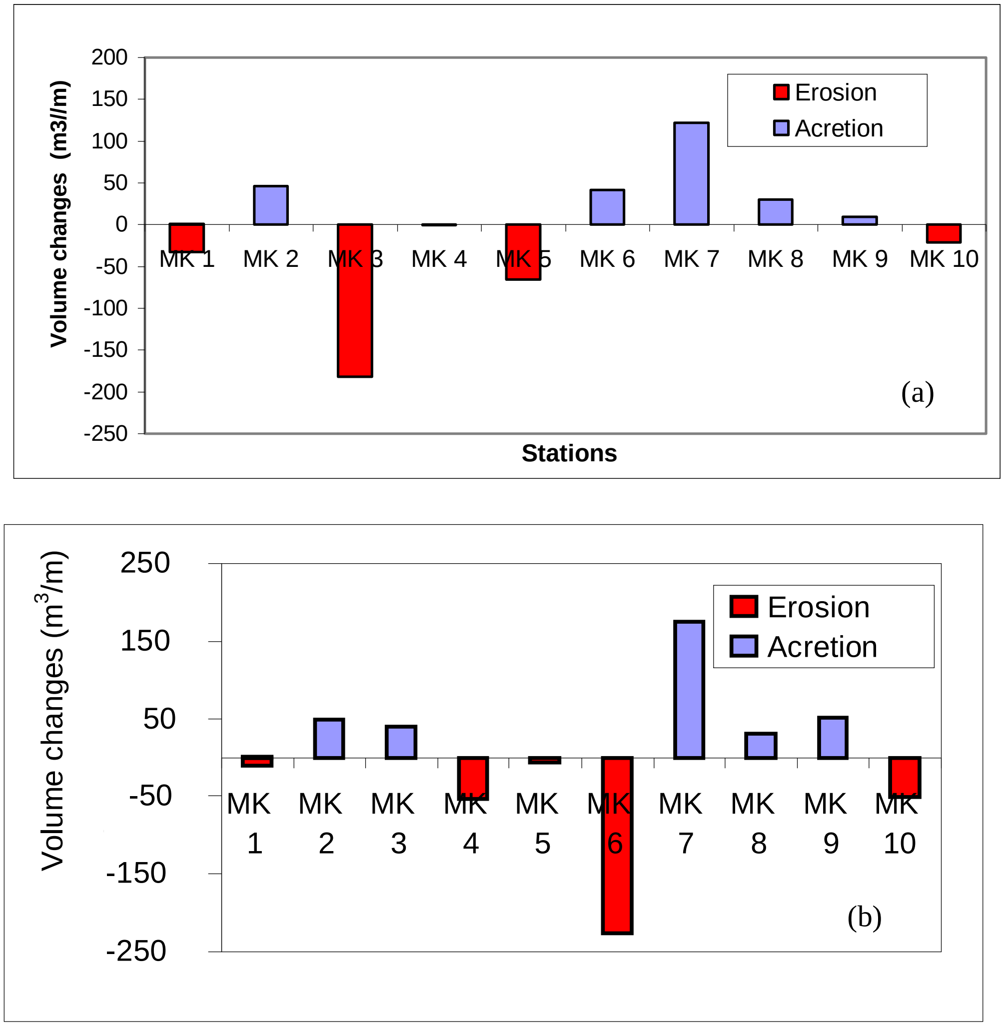


Fig. 4.33 Beach volume changes at Trikkunnapuzha (a) from pre-monsoon to monsoon and (b) from pre-monsoon to post-monsoon during 2004

4.8 Calicut

The Calicut coastline is generally straight and oriented in NNW direction. The beach is comparatively wide, with a foreshore of moderate slope. Towards south of the coast two rivers Kadalundi and Beypore debouch into the sea (Fig. 4.34).

4.8.1 Wave characteristics

Secondary data on nearshore waves, beach morphology, sediment characteristics, etc. available at CESS for the period 1981-1982 are used for the study. The nearshore waves were measured at a depth of 5 m during 1980-1985 as part of a wave project conducted by CESS. A pressure type wave and tide telemeter system was used for recording the waves (Fig. 4.32). The data used are taken from the Data Report published by CESS (Baba et.al., 1989). The wave parameters like, H_s, T_z and wave direction recorded during July-August 1981 were taken for analysis and modelling study.

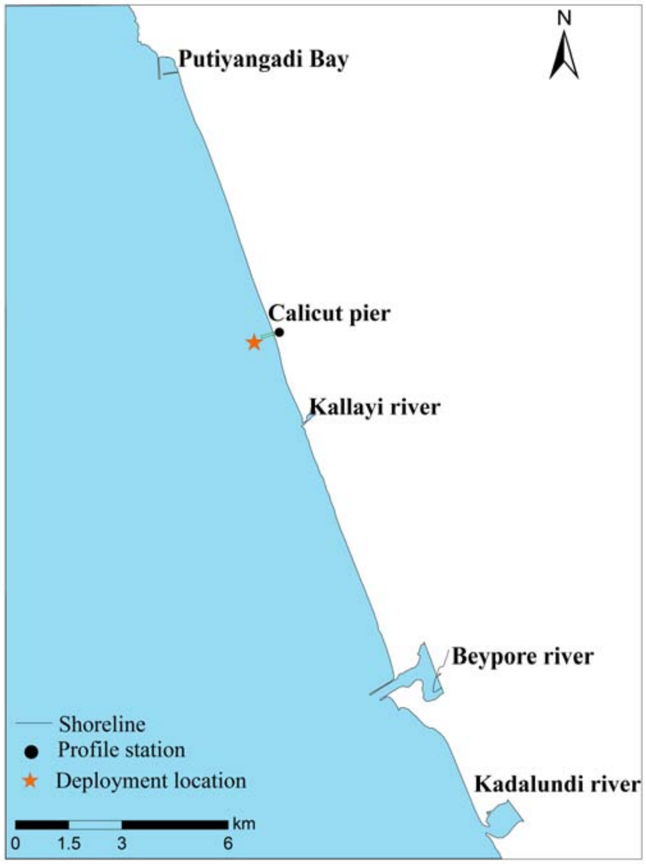


Fig. 4.34 Location map of Calicut study area

Table 4.16 Nearshore wave statistics off Calicut pier during monsoon 1981

Parameters	**Period**	**Min.**	**Max.**	**Mean**	**Standard Div.**
Wave Height (H_s), m	Monsoon (01.07.1981 to 01.08.1981)	0.13	1.53	0.93	0.32
Wave Period, (T_z) sec		8.0	20.5	9.93	1.83

The statistical parameters of waves relevant for the period July-Aug 1981 are given in the Table 4.16. The frequency distribution graphs of wave height (H_s), wave period (T_z) and wave direction during this period are presented in Figs. 4.33, 4.34 and 4.35.

4.8.1.1 Wave height (H_s)

The frequency distribution of significant wave heights (H_s) indicates spreading of wave heights during the study period. Frequency distributions of wave heights show (Fig. 4.35) that about 16.7% of the H_s are below 1.2 m, 38% in the range 1.2 – 1.8 m, 41% in the range 1.8 – 2.4 m and 11% above 2.4m. The standard deviation 0.32 indicates wide spreading of wave height.

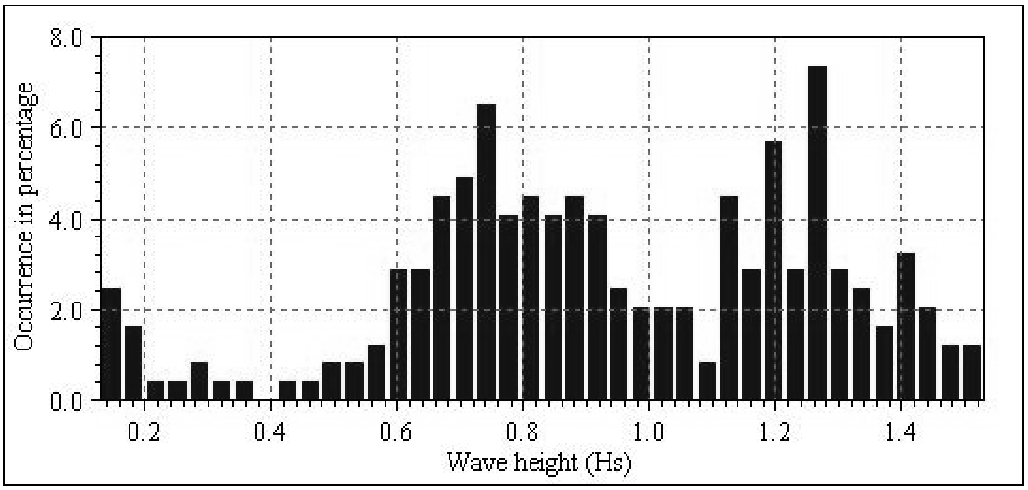


Fig. 4.35 Frequency distribution of H_s during July-August 1981 off Calicut

4.8.1.2 Wave period (T_z)

The frequency distribution of wave period (T_z) shows characteristics typical of monsoonal wave (Fig. 4.36). The T_z varies from 8 to 20 s, with T_z less than 8.7 s for 16.7 % of time, T_z in the range 8.7–10.5 s for 50% of time and T_z above 10.5 s for 33.3 % of time.

4.8.1.3 Wave direction

During this period the wave direction varies from 235 to 300 ºN. The mean wave direction is 260 ºN, which is (Fig. 4.37) typical of monsoon waves. The frequency distribution diagram shows that directions in the range 248–263 account for 50% of time while direction > 263 account for 33 % of time.

4.8.2 Beach profiles

The beach profiles for the monsoon of 1981 were taken for modelling study. In the early stages of monsoon significant erosional tendency is observed with the formation of bar in the nearshore (Fig. 4.38). Towards the latter part of monsoon the beach build-up is initiated with the shoreward migration of bars.

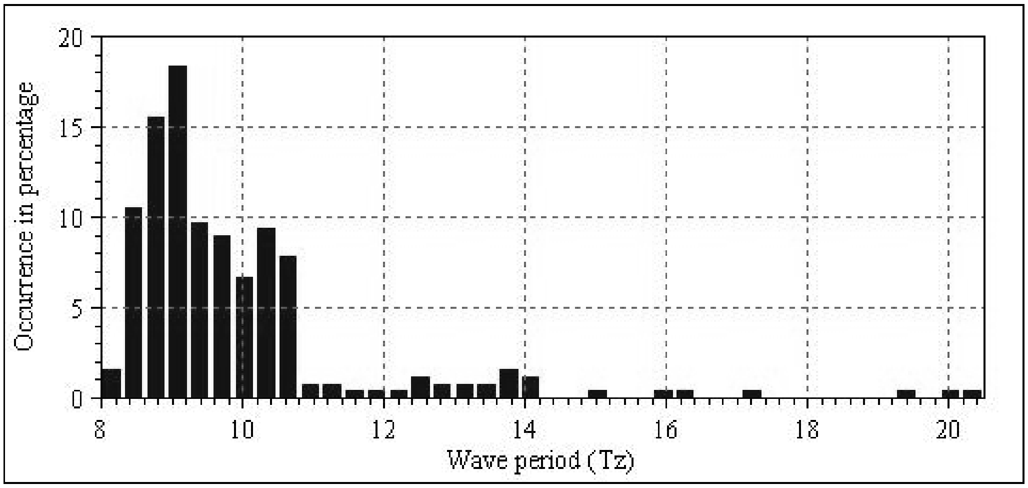


Fig. 4.36 Frequency distribution of T_z during July-August 1981 off Calicut

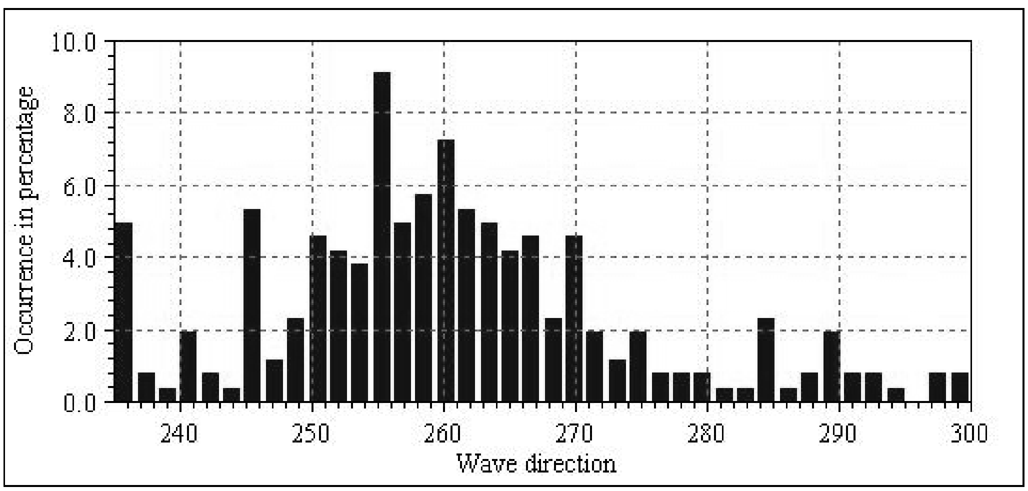


Fig. 4.37 Frequency distribution of wave direction during July-August 1981 off Calicut

4.8.2.1 Beach volume changes

The beach volume changes for the period May-July 1981 are presented in Table 4.17 and Fig. 4.39 In the month of May the erosion is initiated in a significant way with a quantum of 55.25 m^3/m. In the month of June the erosion reaches the peak value of

91.27m^3/m. The process is reversed in the month of July with an accretion of 24.95m^3/m.

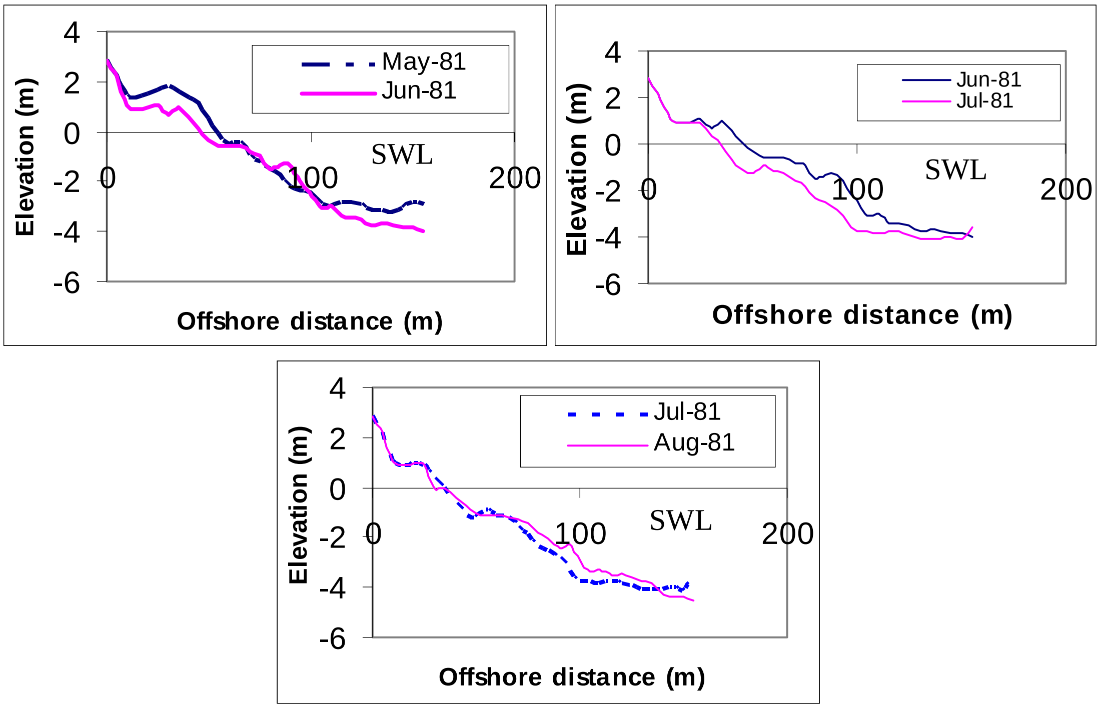


Fig. 4.38 Beach profiles measured at Calicut pier during monsoon period of 1981

Table 4.17 Beach volume changes at Calicut pier during monsoon period of 1981

Sl. No.	Stations	Period	Beach Volume change (m^3/m)	Remarks
1.	Pier	01/05/81 – 01/06/81	-55.25	Erosion
2.	Pier	01/06/81 – 01/07/81	-91.27	Erosion
3.	Pier	01/07/81 – 01/08/81	24.95	Accretion

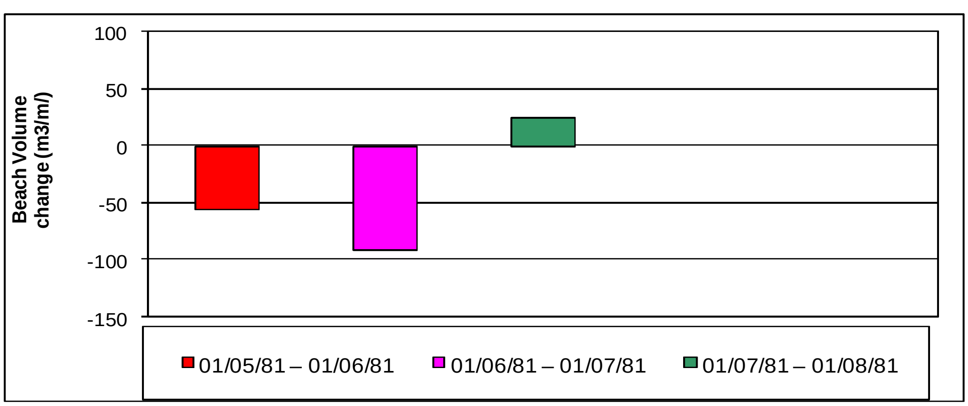


Fig. 4.39 Seasonal beach volume changes (m^3/m) during monsoon season of 1981

4.8.2.2 Sediment characteristics

The sediment size characteristics for berm crest and beach face at Calicut pier station are used for the study (Table 4.18). The mean grain size varies from 0.17 mm at foreshore to 0.28 mm at berm.

Table 4.18 Sediment characteristics of Calicut beach for June 1981

Station	Sampling point	Mean (mm)	Median (mm)
Pier	Berm	0.23	0.24
	Foreshore	0.17	0.16

4.9 Discussion

The southwest coast of India is characterised by differing wave energy levels. The highest wave intensity is reported along the Trivandrum coast which decreases gradually towards north till Tellicherry (Baba, 1988). Kurian (1987) has categorized the coast into different wave energy zones. According to that categorization, Trivandrum coast corresponds to high energy, Alleppey to medium energy and Calicut coast to low energy. The wave data used in the present study for different locations starting from Valiathura in the south to Calicut in the north corroborate this categorisation. Seasonal variations in the wave energy are observed all along the coast. This is in tune with the results of earlier studies (Hameed, 1988; Kurian, 1988; Thomas, 1988).

The study also brings out the variation in height-period-direction grouping during different seasons. During the peak of monsoon steeper waves of higher wave heights, shorter wave periods and westerly direction are observed. These are characteristic of sea waves generated in the Arabian Sea during the peak monsoon spell, the generating area being relatively closer to the coast on the western side. The periods are relatively shorter when compared to the long travelled swells, and directions are westerly. Wave characteristics during non-monsoon seasons (i.e. pre- and post-monsoon seasons) are characterized by low wave height (<1m), wide range in periods (7-15 s) and SSW-SW directions. These are indicative of long period swells reaching the coast from the south Indian Ocean. In addition to the long period waves as above, very short period low waves generated by local winds are also seen during the fair weather period. Intermittent breaks in intensity of waves during monsoon for periods extending for 10 to 15 days are observed, which is in tune with the normal characteristic of the monsoonal climate.

Study on beach morphodynamics brings out seasonal changes of beach erosion/accretion which are quite similar with those reported by earlier researchers for different locations of southwest coast of India (eg. Hameed, 1988; Thomas, 1988; Kurian, 1988). Generally beach shows maximum width during the fair weather season. Beach reaches its nadir during the peak of monsoon. The monsoonal waves being steep, offshore transport of sediments takes place resulting in intense erosion. The beach volume changes during this season points out the high erosion and the quantity of erosion vary from site to site, which may due to varying hydrodynamic, beach and inner shelf morphological characteristics and human intervention. During the fair season, long period waves prevail; onshore transport of sediments takes places under the spell of these waves resulting in beach re-building process. The impact of human interventions by way of construction of breakwater, shore protection structures is also evident in locations like Muthalapozhi, Kayamkulam inlet. Another interesting aspect was the influence of mud bank, which leads to build-up of beach at locations like Thrikkunnapuzha and Mararikkulam.

The beach sediment distribution pattern along the southwest coast is characteristic of the varying geomorphology and energy conditions. At Valiathura and Muthalapozhi which forms part of the high energy Trivandrum coast, the sediments are medium sands. The sediment size doesn’t show any noteworthy variation with seasons or changing wave conditions. Further north, the Kayamkulam inlet region, part of the medium energy coast, has fine sand which continues further north towards the Trikkunnapuzha sector. However, the Mararikkulam sector which has more or less same energy level as the Kayamkulam-Thrikunnapuzha sector has slightly coarser sand belonging to the fine and medium sand category. The relatively finer sand in the Kayamkulam-Thrikunnapuzha sector in relation to the wave energy may be due to the occurrence of heavy sands which are fine grained along this coast. The heavy sands being denser will tend to have smaller size when compared to the quartz (white sand).

At Calicut, the sediments belong to the fine sand category which obviously is in tune with the low to medium energy level of this coast.

4.10 Summary

Based on energy regime and environmental characteristics, three representative coastal stretches *viz.* Trivandrum (Valiathura, Muthalapozhi), Alleppey (Kayamkulam, Trikkunnapuzha, Mararikkulam) and Calicut of south-west coast of India were selected for the collection/collation of hydrodynamic, sedimentologic and beach morphological data required for model calibration/validation. An analysis of the extensive field data used for the study has been carried out. Considerable spatial variation in wave intensity in consonance with the available literature is shown. Seasonal variations are characterised by steep waves of higher wave heights, shorter wave periods and westerly direction during peak monsoon and long period swells from the SSW-SW directions during the pre- and post-monsoon seasons. Analysis of beach profile data brings out the seasonal and spatial variations in the erosion/accretion processes brought about by the natural as well as anthropogenic factors. Irrespective of location the beach shows maximum width during the fair weather season due to the onshore transport of sediments by the long period swells and reaches its nadir during the peak of monsoon due to the offshore transport of sediments by the steep monsoonal waves. Human-induced activities such as construction of shore structures, sand mining brings out anomalies in erosion/accretion pattern within the same location. The beach sediment characteristics show considerable spatial variation in tune with the hydrodynamic regime and beach morphological characteristics. The Trivandrum coast with the highest energy level is characterised by the coarsest sediments and the Calicut coast with the lowest energy regime by the finest sediments. At each location across-the- profile and seasonal variations in the sediment characteristics are also seen. It can be seen that the data set used for model calibration/validation pertain to a wide range of coastal environmental conditions representing different energy regime and geomorphological characteristics thereby ensuring the reliability and credibility of the calibration process.

05

Calibration and Validation of Models

Chapter 5

CALIBRATION AND VALIDATION OF MODELS

5.1 Introduction

Calibration of numerical model is the most important requirement of any numerical model study. Calibration achieves two main outcomes: (i) the level of confidence in the model predictions is established and (ii) the best possible accuracy is obtained after the calibration is carefully done and the model is properly optimised. However, the process of calibration requires a detailed assessment of the local data and field site characteristics to fully realise the potential of models. Each location may have its own features arising from the natural processes as well as anthropgenetic influences. Hence errors may develop in a model simulation if these special features are not properly accounted for. Consequently, numerical models are to be refined to match the site, and the decision making leading to any modifications occurs during the calibration phase. The process of calibration also involves adjustment of empirical coefficients in the models. The empirical coefficients may vary from site to site and therefore need calibration so that predictions best match the data. It is desirable to relate empirical parameters in the model directly to physical quantities or assign them a constant value to minimize the degree of freedom in the calibration process. The number of parameters available for adjustment in the calibration process is thereby reduced with little loss of accuracy in determining an optimal calibration.

Coastal engineers and policy makers are not concerned about the internal processes of the particular model they use but are concerned about the credibility and performance of the model. The performance of numerical models is used to establish the credibility of the model. It depends on mathematical stability of the model as well as the quality of the model. Statistical and graphical tests have been carried out to determine the performance of the model. The model performance was also assessed by comparing the model simulations with commercially available LITPACK (DHI, 2002) model.

5.2 Calibration of Shoreline Change Model

The Shoreline Change model has been developed as an engineering tool for predicting beach profile response to alongshore transport of sediments mainly by the action of longshore currents. The developed numerical model was applied to simulate shoreline change in different environmental conditions. As an objective criterion for judging agreement between the simulated and measured shoreline, the values of different model parameters were varied in the calibration process. It becomes apparent that values of some coefficient would have to be modified to achieve agreement between measured and simulated shoreline. The process of calibration carried out for different coastal sectors is discussed in the following sections.

5.2.1 Valiathura

5.2.1.1 Data used

The basic input data were collected from secondary data available in CESS and field measurements as discussed in Chapter 4. The input data includes wave parameters, shoreline position, dimensions of coastal structures, berm characteristics, etc. The berm height is a critical input parameter. The measured shoreline positions at Valiathura coastal stretch during 1981-1982 is given in the Fig. 5.1.

5.2.1.2 Model calibration

The model has been calibrated by varying input parameters like wave height, period and depth of closure (D_c). The calibration was carried out for the period from May 1981 to May 1982. Initially the simulation was carried out by putting the root mean square wave height (H_{rms}) as one of the input parameters. Simulated shoreline using monthly average H_{rms}, T_z and mean wave direction and depth of closure of 10 is given in the Fig. 5.1

As can be seen the simulated shoreline is not matching with measured shoreline. The simulation was continued with H_s instead of H_{rms}. Fig. 5.2 presents the results of simulation with monthly average H_s, T_z and mean wave direction with D_c of 10m. Some improvement in the output in comparison with measure shoreline is seen. The simulation was continued further with daily average H_s.

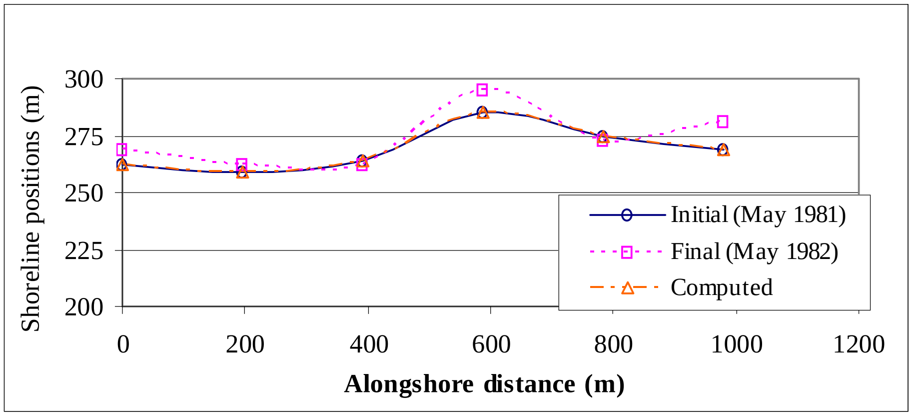


Fig.5.1 Simulated shoreline change using monthly average H_{rms}, T_z and mean wave direction along Valiathura coast during 1981-1982

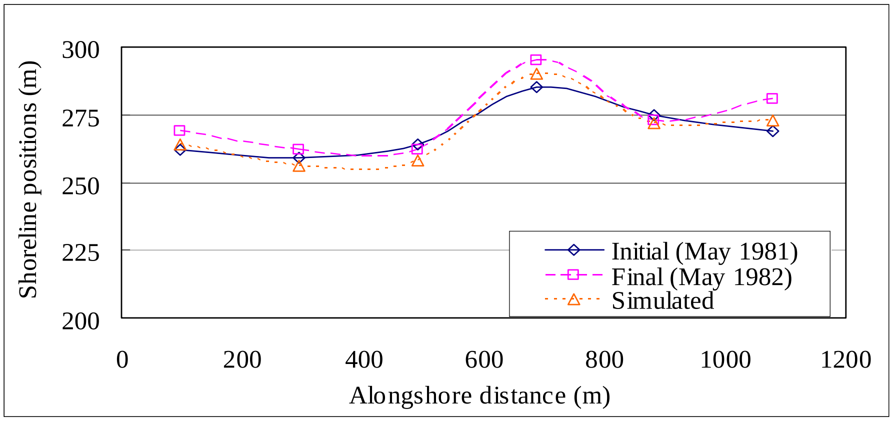


Fig. 5.2 Simulated shoreline during 1981-1982 along Valiathura coast using monthly average H_s, T_z and mean wave direction

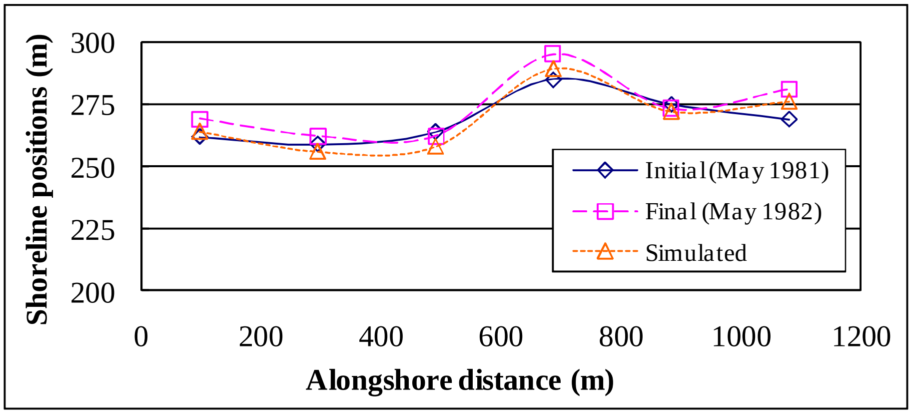


Fig. 5.3 Simulated shoreline using daily average H_s, T_z and mean wave direction along Valiathura coast during 1981-1982

Fig. 5.3 shows simulated shoreline change using daily average wave parameters and depth of closure (D_c) of 10 m along Valiathura coast. No marked improvement in the performance is seen.

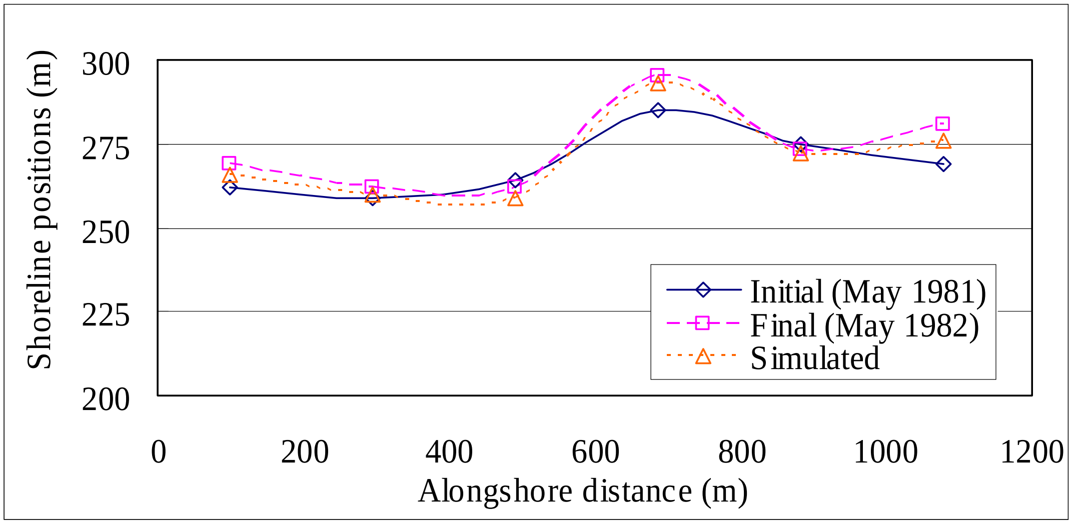


Fig. 5.4 Simulated shoreline changes with input of daily average of H_s, T_z and D_c, 15 m along Valiathura coast during 1981-1982

The simulation so far was attempted using average values of wave parameters with constant value of D_c, 10m. Further simulations were carried out to investigate the varying effect of D_c. It is found that when D_c =15 m, the simulation gives good results. Hence, it is concluded that the model gives best results with daily average wave parameters (H_s, T_z and mean wave direction) and depth of closure (D_c) of 15m.

5.2.2 Muthalapozhi

As mentioned in Chapter 4, Muthalapozhi is selected as another location in the Trivandrum coast for calibration of the shoreline change model. The construction of two breakwaters on either side of the inlet which was nearly completed in 2003-04 during the period of field measurements has induced erosion in the northern side and accretion in the southern side (Fig. 5.5).

5.2.2.1 Data used

The data used are secondary data available in CESS. The shoreline change over a period of one year in the southern and northern sectors of Muthalapozhi inlet during 2003 - 2004 are given in Figs. 5.6 and 5.7.

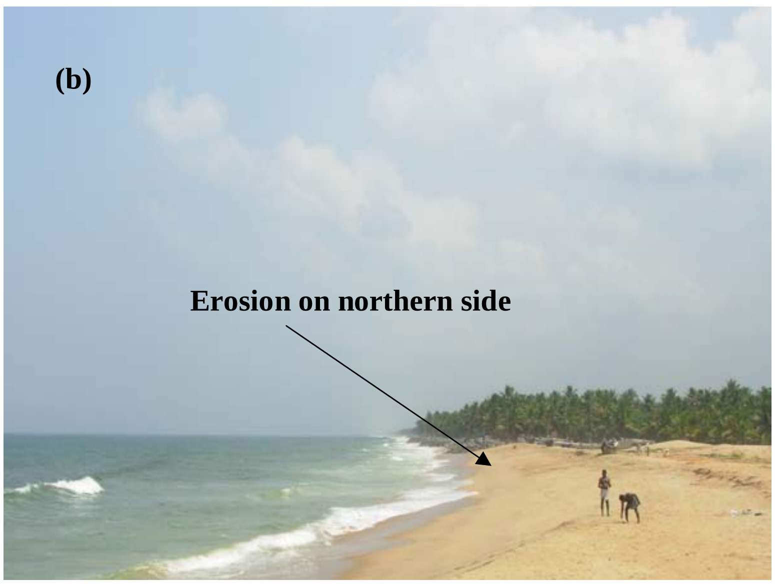


Fig. 5.5 Impact of breakwaters at Muthalapozhi inlet: (a) accreted beach in the sector south of breakwater and (b) eroded beach north of the inlet

5.2.2.2 Model calibration

The calibration exercises were carried out for the beaches in the southern and northern sectors of Muthalapozhi inlet. The model has been calibrated by varying input parameters like wave parameters, depth of closure as in the case of Valiathura.

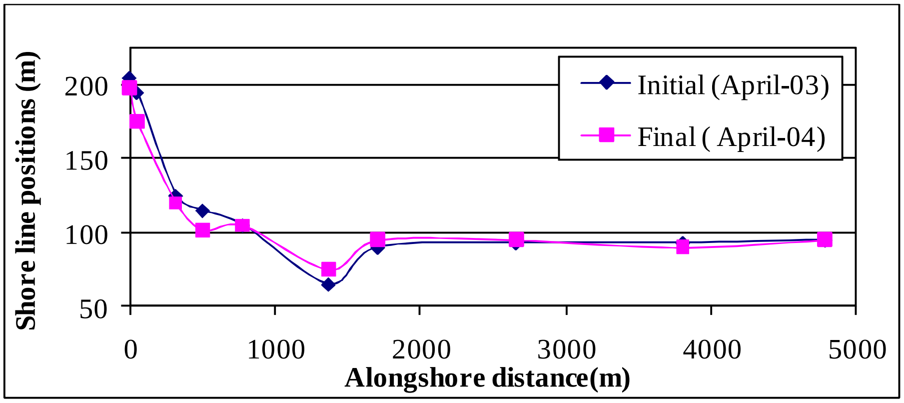


Fig. 5.6 Shoreline changes in the sector south of Muthalapozhi inlet during April 2003 to April 2004

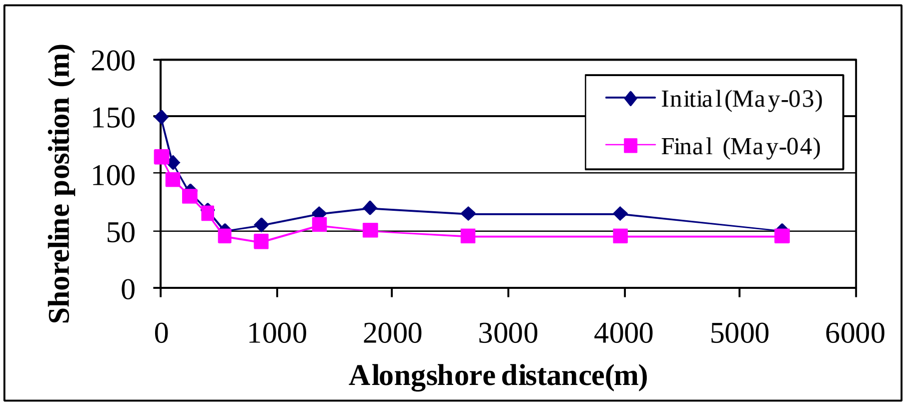


Fig. 5.7 Shoreline changes in the sector north of Muthalapozhi inlet during April 2003 to April 2004

The simulated shorelines south and north of the inlet are given in Fig. 5.8 and Fig. 5.9 respectively. The model gives best results for depth of closure ($D_{c)}$ of 14m.

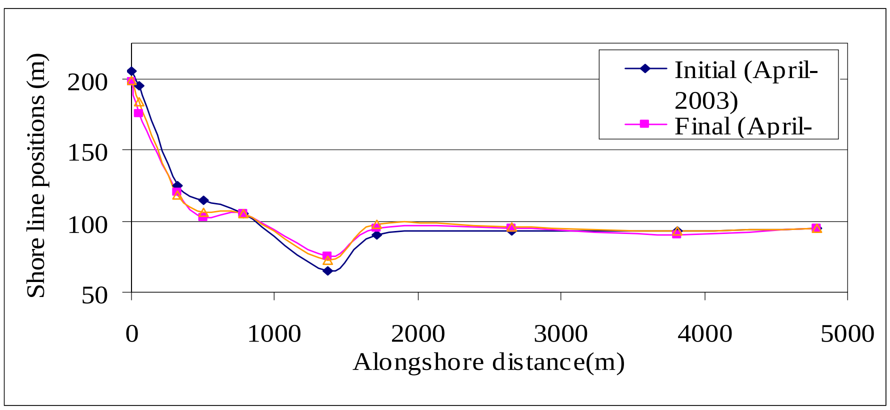


Fig. 5.8 Simulated shoreline with input of daily average wave parameters and depth of closure of 14 m in comparison with the initial and final profiles in the sector south of Muthalapozhi inlet.

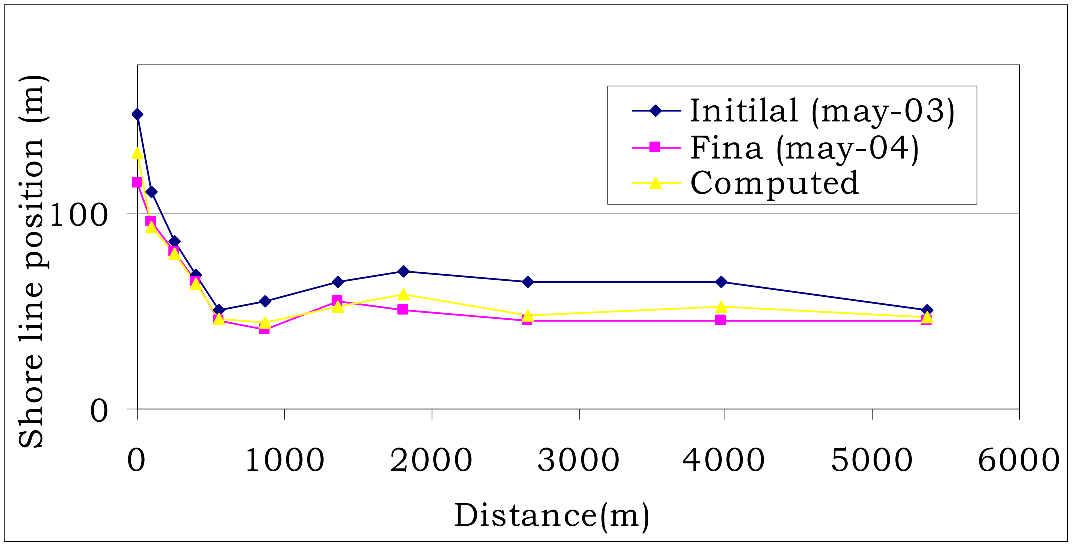


Fig. 5.9 Simulated shoreline with input of daily average wave parameters and depth of closure of 14 m in comparison with the initial and final profiles in the sector north of Muthalapozhi inlet.

5.2.3 Kayamkulam

The study area as discussed in Chapter 4 is flanking both sides of the Kayamkulam inlet. The construction of two breakwaters at the inlet has induced erosion in the northern side of the Kayamkulam inlet and accretion in the southern side. The model study was conducted for the period 2003-04 when the construction of breakwater was nearing completion (Fig. 5.10).

5.2.3.1 Data input

The basic input data are partly secondary data available in CESS and rest measured data under the study. The input data includes wave parameters, shoreline position, dimensions of structure, berm characteristics etc. The detailed analyses of data are given in the Chapter 4.

5.2.3.2 Calibration of model

The model has been calibrated by varying input parameters like wave parameters and calibration parameters such as depth of closure. The calibration was carried out for the period 2002-04. The results of the calibration trials obtained by using H_{rms} and D_c of 10 m is presented in the Fig. 5.11

Fig. 5.10 Impact of breakwaters at Kayamkulam inlet: (a) accreted beach in the sector south of breakwater and (b) eroded beach north of the inlet

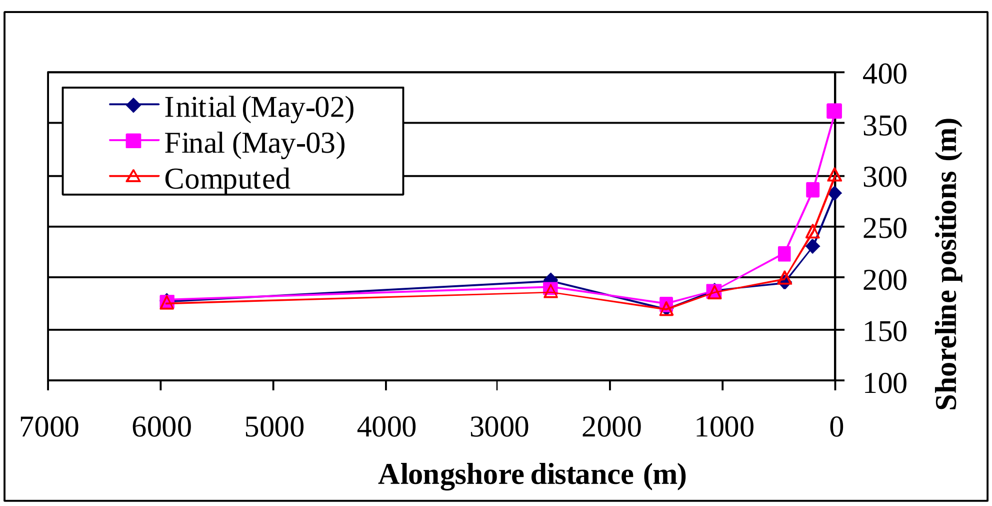


Fig. 5.11 Simulated shoreline changes with input of H_{rms} in the sector south of Kayamkulam inlet for May 2002 - May 2003

It can be seen that the simulated shoreline does not match with measured shoreline. As in the earlier cases, simulation using H_s is found to give better results. However, for still better results the model was later calibrated by varying the depth of closure (D_c) starting with a value of 5m. It is found that the model shows best results when depth of closure is 9 m with daily average wave parameters (Fig. 5.12)

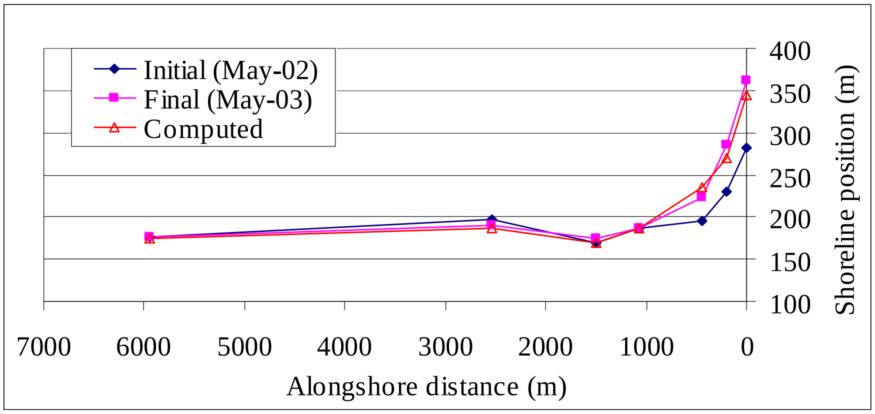


Fig. 5.12 Simulated shoreline change south of Kayamkulam inlet for the period May 2002 to May 2003 with input of daily average H_s, T_z and depth of closure of 9m

5.3 Validation of shoreline change model

The model requires validation after appropriate calibration process. The present model is validated for the sector south of Kayamkulam inlet, one of the critically accreting sectors of southwest coast of India. The same values as used in the earlier calibration processes for Kayamkulam south were used in the validation. The model simulation more or less matches with measured shoreline positions (Fig. 5.13).

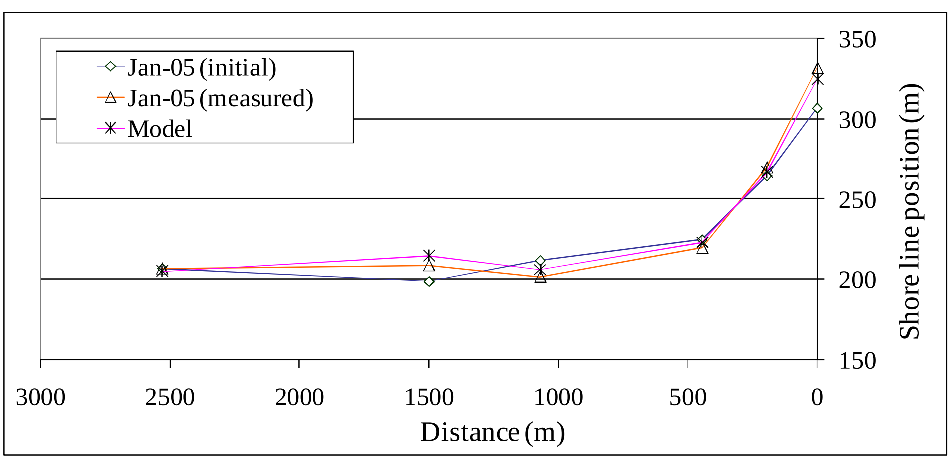


Fig. 5.13 Simulated shoreline change south of Kayamkulam inlet with input of daily average of H_s, T_z and depth of closure of 9 m.

Thus, the Shoreline Change model has been calibrated/validated for different locations of the southwest coast of India. The model can predict shoreline with and without shore connected structure. It simulates well the impact of breakwater on the adjoining coast.

5.4 Calibration of Profile Change Model

The Profile Change model has been developed as an engineering tool for predicting beach profile response to episodic events like storms, monsoon etc. The processes of calibration require a detailed assessment of the local data and field site characteristics to fully realise the potential of models. The calibration was carried out for different locations with varying environmental conditions. The results are discussed in the following sections.

5.4.1 Valiathura

5.4.1.1 Data used

The calibration was carried out for Valiathura for the peak monsoon period of mid-June 2005. High erosion was observed during this period as seen in the beach profiles presented in the Fig. 5.14. The berm is completely eroded and deposited as a bar in the offshore. The input data for simulations include wave parameters, cross-shore beach positions, berm characteristics, etc.

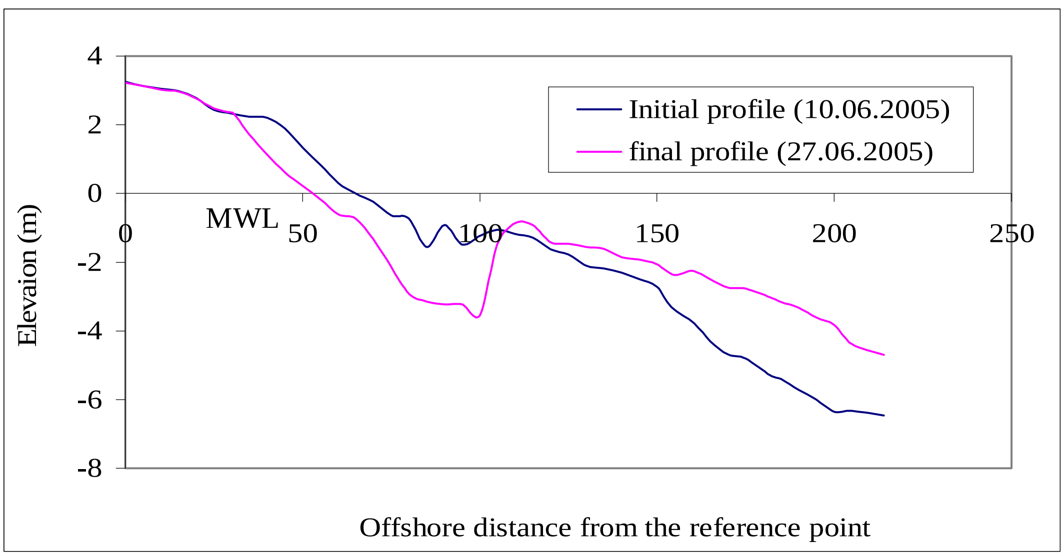


Fig. 5.14 Measured beach profiles at Valiathura pier during June 2005

5.4.1.2 Model calibration

To minimise the difference between measured and computed values, the model has been run for many cases. The main calibration parameter is transport coefficient, K in the transport rate equation (Larson and Kraus 1989). The other calibration coefficients are slope dependent coefficient epsil (€) and empirical coefficient Gama (Ѓ).

Transport coefficient K is an empirical constant, which governs the time response of the beach profile. The transport rate coefficient is fixed by carrying out the model simulations for different values and comparing with field conditions. The empirical transport coefficient (K) has greater influence in equilibrium beach profile change. A number of simulations were carried out for different values of K by keeping other calibration coefficients as constant. When the value of K increases erosion increases and bar volume increases (Fig. 5.15). The effect of K mainly depends on time. The simulations show that the model gives best results for K =$2x10^{-6}$ m^4/N for Valiathura.

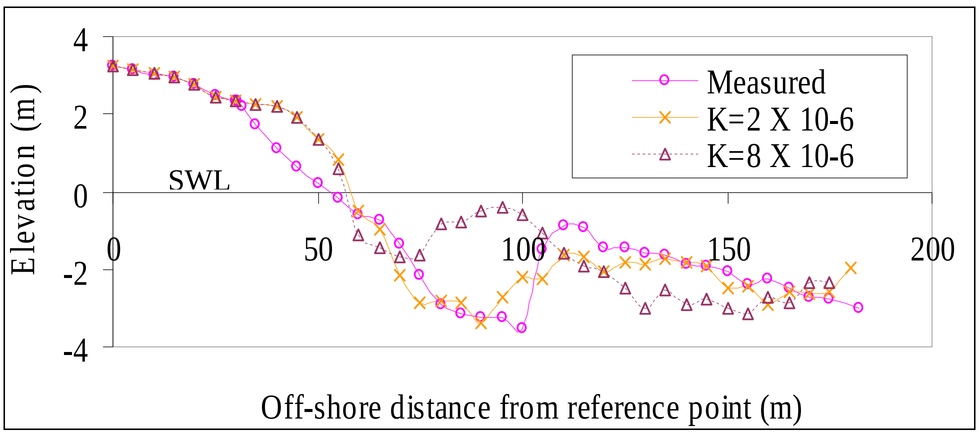


Fig. 5.15 The simulated profiles for different values of transport coefficient (K)

The other calibration parameter is a slope dependent term Epsil (€), which mainly influences equilibrium bar volume (Larson and Kraus, 1989). The profile response to € is highly dependent on time and it has less effect on the initial stages of profile change. For the calibration, simulations were carried out varying €, which was finally set to 0.00183 m^2/sec for study area.

The profile change is influenced by another factor Gama (Ѓ), which is the ratio between wave height and water depth. When the value of Gama (Ѓ) decreases, the tendency for bar formation decreases. Calibration exercise was carried out by using different values of Ѓ, keeping the other parameters constant at the calibrated values

(Fig. 5.16). By calibration process the value of Ѓ is found to be 0.4 for this location. The simulated profile using the coefficient as arrived at above is shown in Fig. 5.17. It can be seen that there is good correspondence between the measured and simulated profiles.

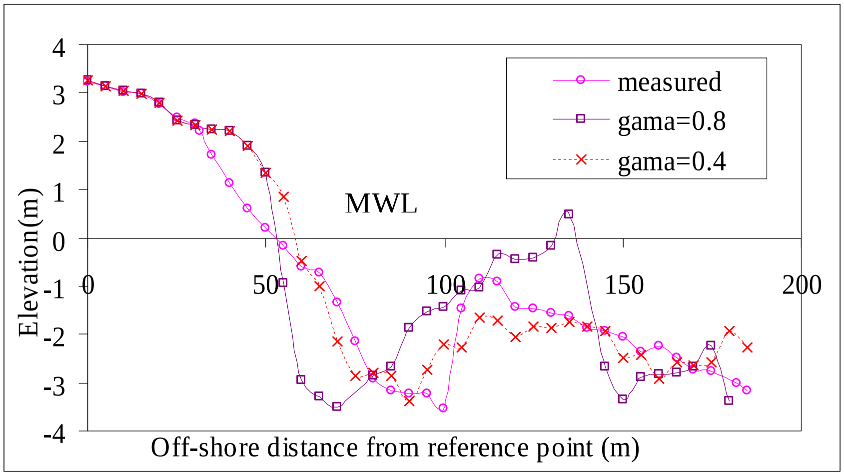


Fig. 5.16 The simulated profiles for different values of Ѓ

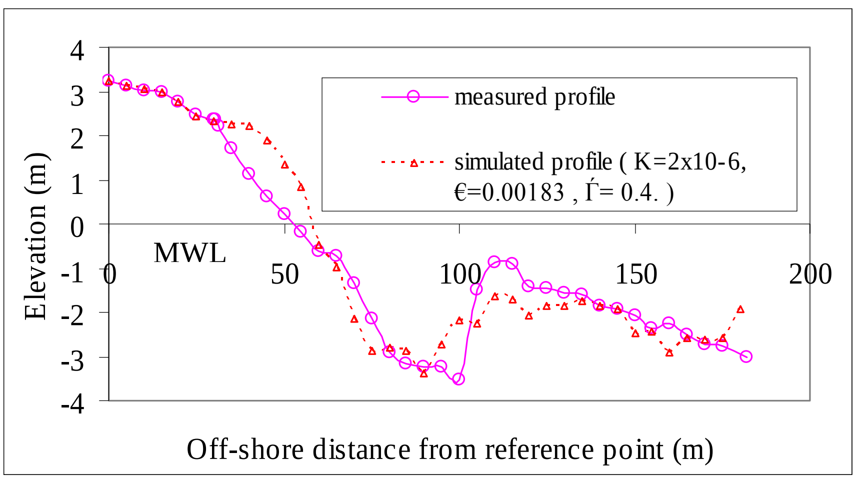


Fig. 5.17 Comparison of the calibrated model output with the measured profile at Valiathura, June 2005

5.4.2 Kayamkulam

The profile change model was calibrated for Kayamkulam inlet area, where hydrodynamic and beach characteristics are entirely different from the Valiathura region. The data used are already discussed in the Chapter 4.

5.4.2.1 Data used

Hydrodynamic, beach morphologic and sedimentological data collected at Kayamkulam from 15/7/2004 to 13/8/2004 in the monsoon season was used for model calibration. The measured beach profiles used for the study are presented in Fig. 5.18.

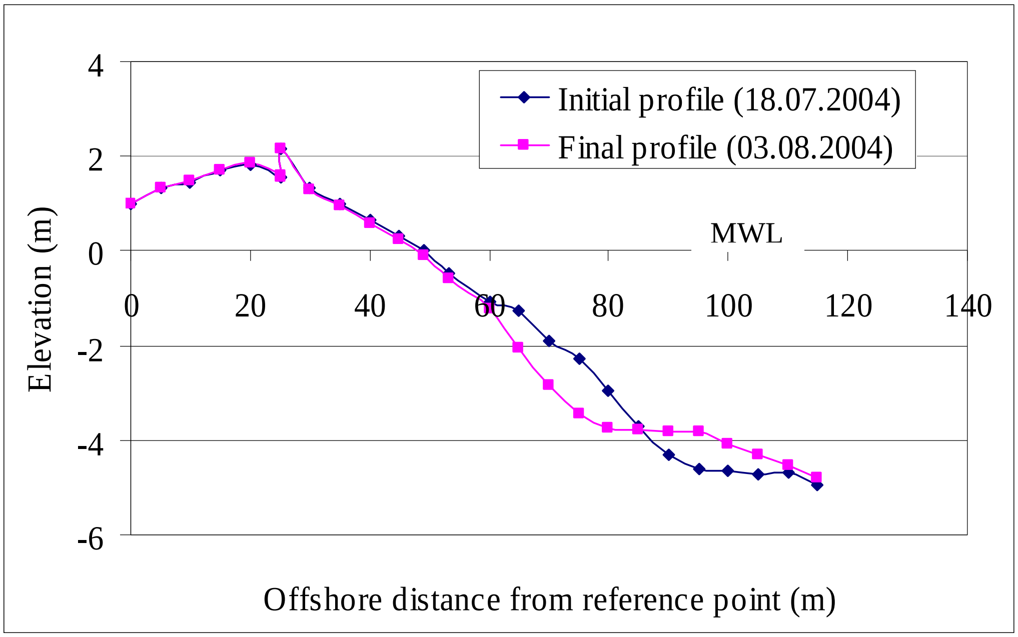


Fig. 5.18 Measured beach profile at station south of Kayamkulam inlet

The initial profile for 18.07.2004 shows a case where the beach face has more or less uniform slope. However, the profile for 03.08.2004 shows a scenario with considerable erosion of the of the lower half beach face coupled with a bar formation in the offshore.

5.4.2.2 Model calibration

The process of calibration was carried out by adjusting various calibration parameters as done earlier. The simulations were carried out till the simulated results more or less matched with the measured values. The study shows the computation values give good results when GAMA is the range 0.6 to 0.8, K=7.0 X10 $^{-6}$ and epsil=0.0156 - 0.0099. The calibrated profile more or less matches with the measured as can be seen in Fig. 5.19.

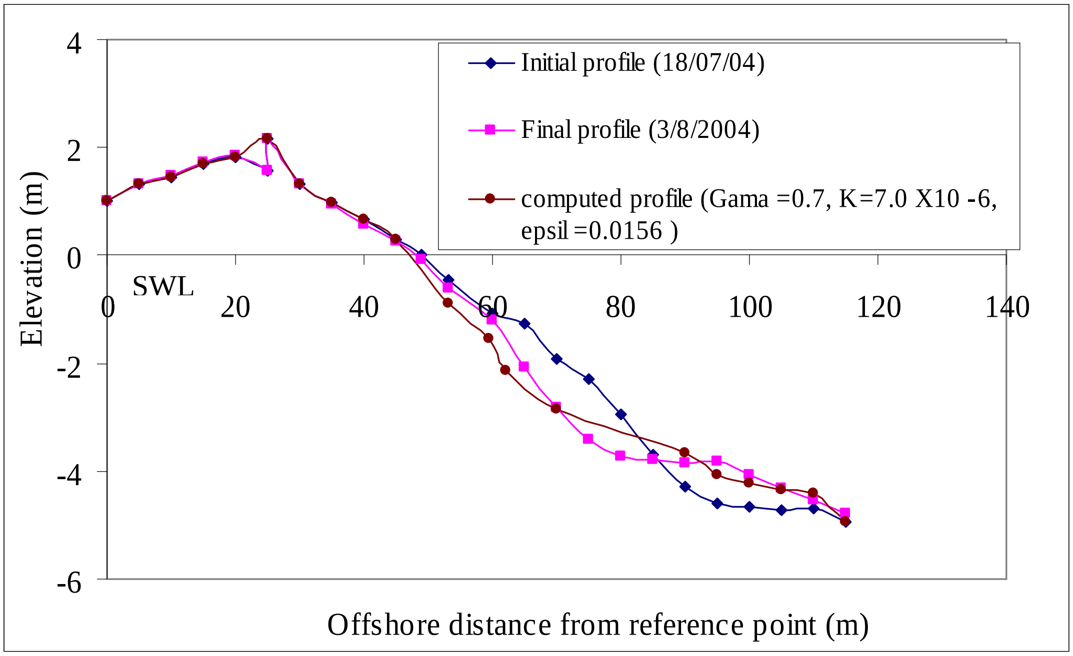


Fig. 5.19 Comparison of the calibrated model output with the measured profile at station south of Kayamkulam inlet for Aug. 2004

5.4.3 Calicut

5.4.3.1 Data used

The nearshore wave data used for the model calibration were measured at a depth of 5 m as part of a Wave Project implemented by CESS during 1980-1985. The beach profile and textural characteristics were also used for the model calibration. The data used pertain to the period July 1981 to August 1981. The details of study area and data used are discussed in the Chapter 4.

5.4.3.2 Model calibration

The initial and final beach profiles used for model calibration are given in Fig. 5.20. The model was calibrated by varying the calibration coefficients as discussed in the previous sections. The model gives plausible results (Fig. 5.20) for K=0.5x10^{-6} m^4/N, epsil=0.0280 m^2/sec and Gama=0.8. The calibration parameters are not identical with those at Valiathura and Kayamkulam because of the different hydrodynamic and sedimentological characteristics.

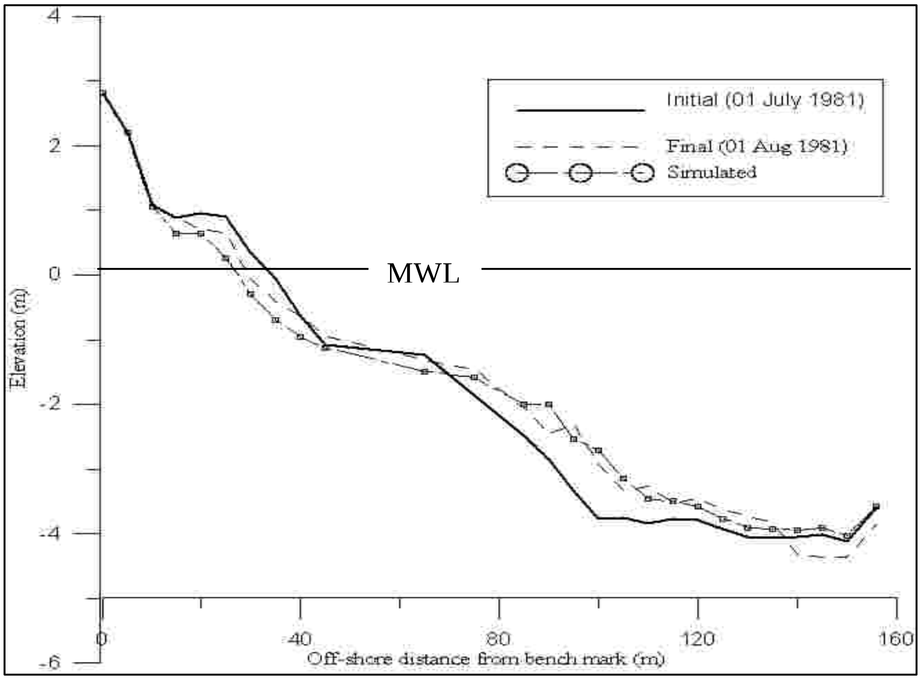


Fig. 5.20 Comparison of the calibrated model out put with the measured profile at Calicut for K=0.5x10^{-6} m^{4}/N, epsil=0.0280 m^{2}/sec and Gama=0.8

5.5 Validation of Profile Change Model

The validation of profile change model has been carried out for Mararikkulam by using the data, for this location, which is discussed in Chapter 4. As can be seen from Chapter 4, this location has the wave and beach morphological characteristics similar to those of Kayamkulam. Computations using the values of calibration parameters arrived for Kayamkulam gives good results as can be seen in Fig. 5.21.

5.6 Evaluation of bar/berm criterion

Depending on the wave conditions, profile shape and sediment properties, the cross-shore sand transport rate will be generally either offshore or onshore over the entire profile. Offshore transport results in erosion at the landward end of the beach profile and formation of a bar near the break point, whereas onshore transport leads to accretion of sand on the foreshore and berm build-up, and the gradual disappearance of the bar near the break point. These two types of profile responses forming two distinctly different beach shapes are commonly observed in both laboratory and field studies, and are known as bar and berm profiles. As the formation of bar and berm profiles are related to the direction of cross shore sediment transport, the criterion used for delineating bar and berm profile could be used to determine cross-shore transport direction. The berm profile corresponds to the on-shore transport of sediments and a bar profile correspond to the offshore transport. The bar/berm profile configurations are also referred to as erosional/accretional, winter/summer or storm/normal or dissipative/reflective profiles.

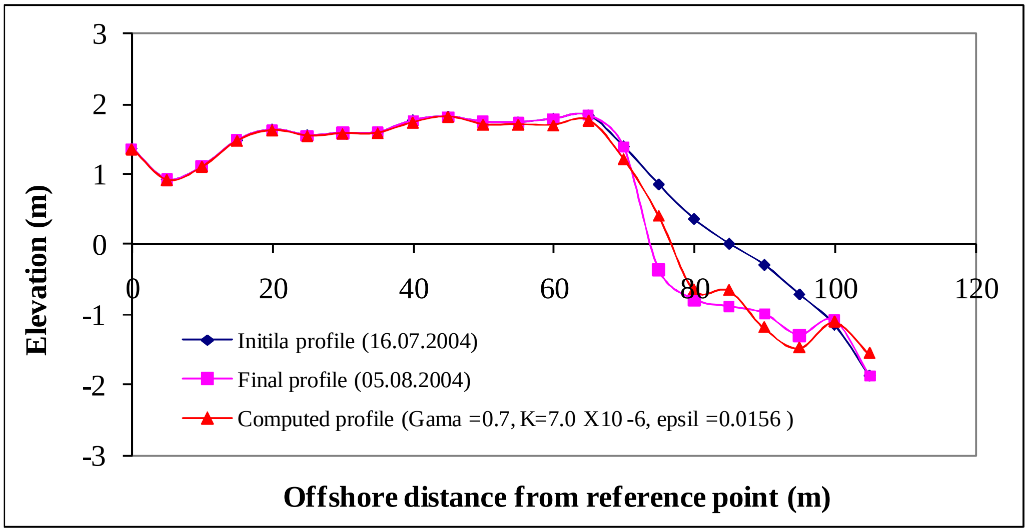
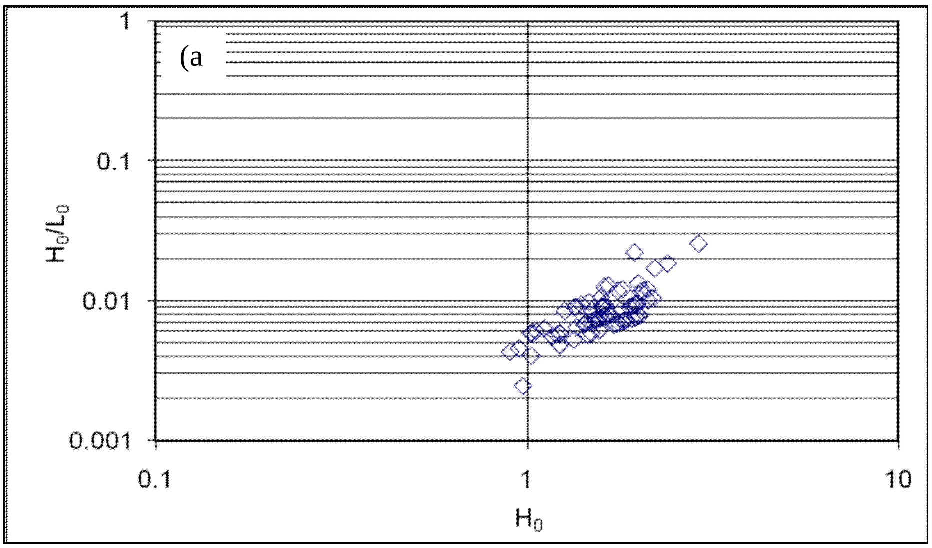


Fig. 5.21 Results of validation of Beach Profile Change model for Mararikkulam

A number of bar/berm criteria have been developed for predicting the general response of a beach profile (a bar or berm profile) to incident waves (e.g. Dean, 1973; Sunamura and Horikawa 1974; Hattori and Kawakawa, 1981 and Larson and Kraus, 1989). The deep-water wave steepness 'H_o/L_o' (the ratio between the wave height and the wave length in deep water, a dimensionless parameter) appears in all criteria. Other parameters appearing in these criteria are the sediment characteristics, such as average grain size or fall velocity and the beach slope. The suspension of bed material depends on the energy flux of waves. The energy flux of incident waves acts as a forcing factor for beach morphology changes, which determine the fluid mixing as well as velocity field under waves (Uda and Omata, 1990). The energy flux can be represented in terms of deep water wave height and wave steepness (Fig.5.22). An evaluation of the bar /berm criterion proposed by Larson and Kraus (1989), which is the most widely used one, is undertaken here using the comprehensive field data for different locations. As stated in Chapter 3, the criterion is used to delineate the accretion and erosion nature of beach in response of wave. The criteria include deep-water wave steepness 'H_o/L_o' (the ratio between the wave height and the wave length in deep water), the sediment characteristics, such as average grain size or sediment fall velocity and beach slope. The parameters appearing in these criteria have distinct physical meaning. The deep-water wave steepness (H_o/L_o) is a measure of the wave asymmetry, which influences the direction of the flow field in the water column. The dimensionless fall speed 'H_o/wT' is a measure of the time that a sediment grain remains suspended in the water column.



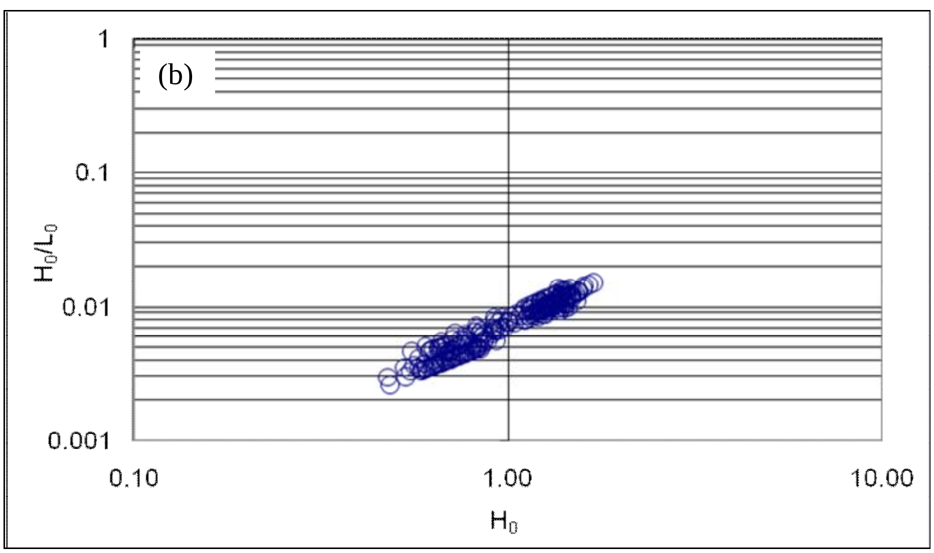


Fig. 5.22 Relationship between wave height and wave steepness at (a) Valiathura for the period 10.06.2005 to 26.06.2005 and (b) Calicut, for the period 01.07.1981 to 01.08.1981

As stated in Eqn. 3.45, the criteria used by them for fixing the direction of transport is,

$(H_o / L_o) \quad < M (H_o/wT)^3,$ bar profile or offshore transport

$\quad\quad\quad\quad\quad > M (H_o/wT)^3,$ berm profile or on-shore transport

The deep water wave parameters were calculated from the measured wave data for the period 10.06.2005 to 26.06.2005. The dimensionless fall speed (H_o/wT) and wave steepness were calculated for each data set. Fig. 5.23 shows a plot of the wave steepness against dimensionless fall speed for Valiathura. It can be seen that for most part of the time, wave steepness is less than fall speed parameter. In other words, the profile is characteristic of a bar profile characterised by offshore transport of the sediments. It is well reflected in the field signature like bar formations, high erosion, etc.

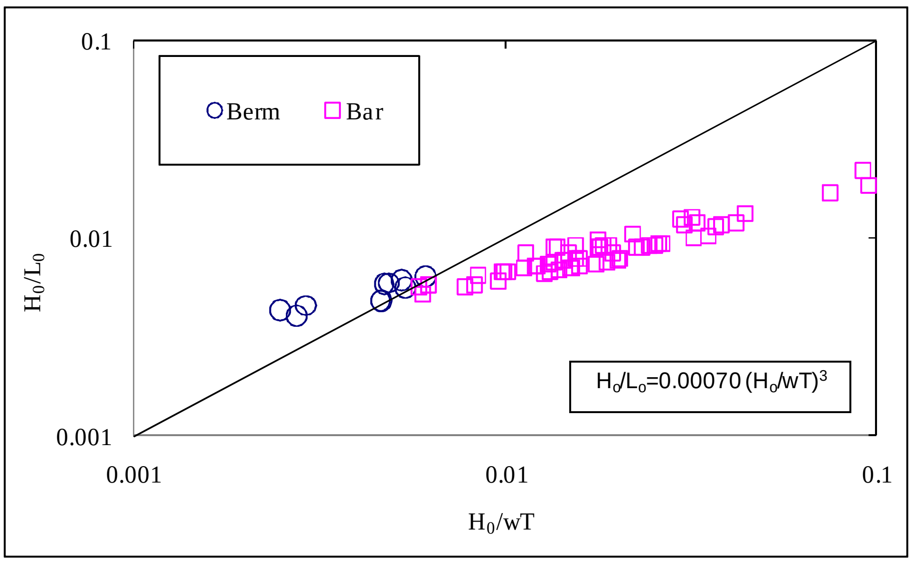


Fig. 5.23 Categorisation into bar and berm profiles based on wave steepness and dimensionless fall speed at Valiathura for the period 10.06.2005 to 26.06.2005 following Larson and Kraus criterion

Even though the coastal environmental conditions at Calicut differ from Valiathura, the results are quite similar to that of Valiathura as can be seen in Fig. 5.24. The bar/berm criterion for Calicut shows the condition is favourable for bar formation, which is corroborated with field conditions. Hence it can be summed up that this criterion can be applied to the southwest coast of India, irrespective of location.

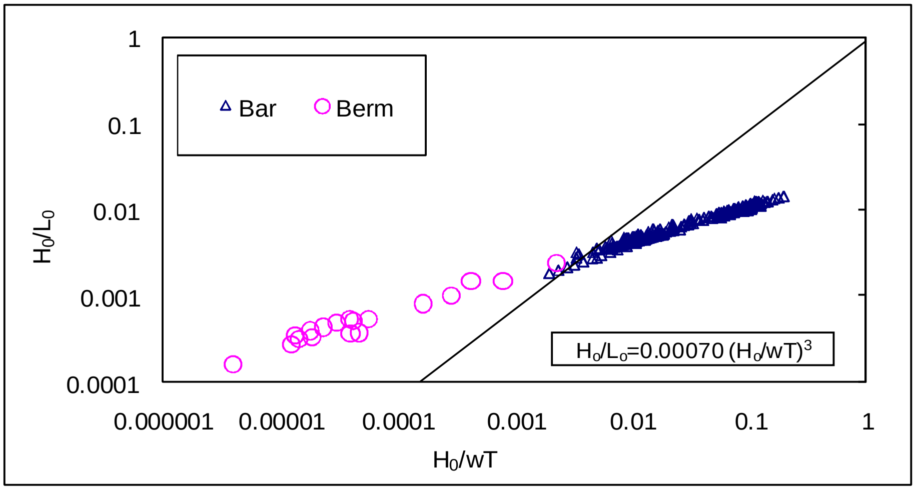


Fig. 5.24 Categorisation into bar and berm profiles based on wave steepness and dimensionless fall speed at Calicut for the period 01.07.1981 to 01.08.1981 following Larson and Kraus criterion

5.7 Directional criterion

Another criterion to establish the erosion/accretion nature of the beach in response to wave is proposed based on the present study. It is called direction constant, which is defined as the ratio between wave steepness and fall speed parameter. If the value of direction constant is <1 the seaward movement of sediment takes place, which leads to erosion and if the value of direction constant is >1 onshore movement of sediments takes place, leading to accretion. Fig. 5.25 presents the distribution of direction constant showing the number of events against each value at both the locations. It can be seen that the values of direction constant fall below '1' in most of the cases at both the locations and very few cases fall above '1', indicating the eroding nature of the beach, which is obvious from the field observation. Hence the directional criterion could be successfully used to find the profile response to waves.

5.8 Model performance

Coastal engineers and policy makers are not concerned about the internal processes of the particular model they use but are concerned about the credibility and performance of the model. Model performance gives the answer to the question "how well the model simulates reality?" (Sutherland et al., 2004). The performance depends on mathematical stability of the model as well as the quality of the model. There are different methods to evaluate the performance of models (Murphy and Epstein, 1989; Gandin and Murphy, 1992; Wilks, 1995; Livezey et al., 1996; Potts et al., 1996). Two methods viz. statistical and graphical have been used to determine the quality of the models.

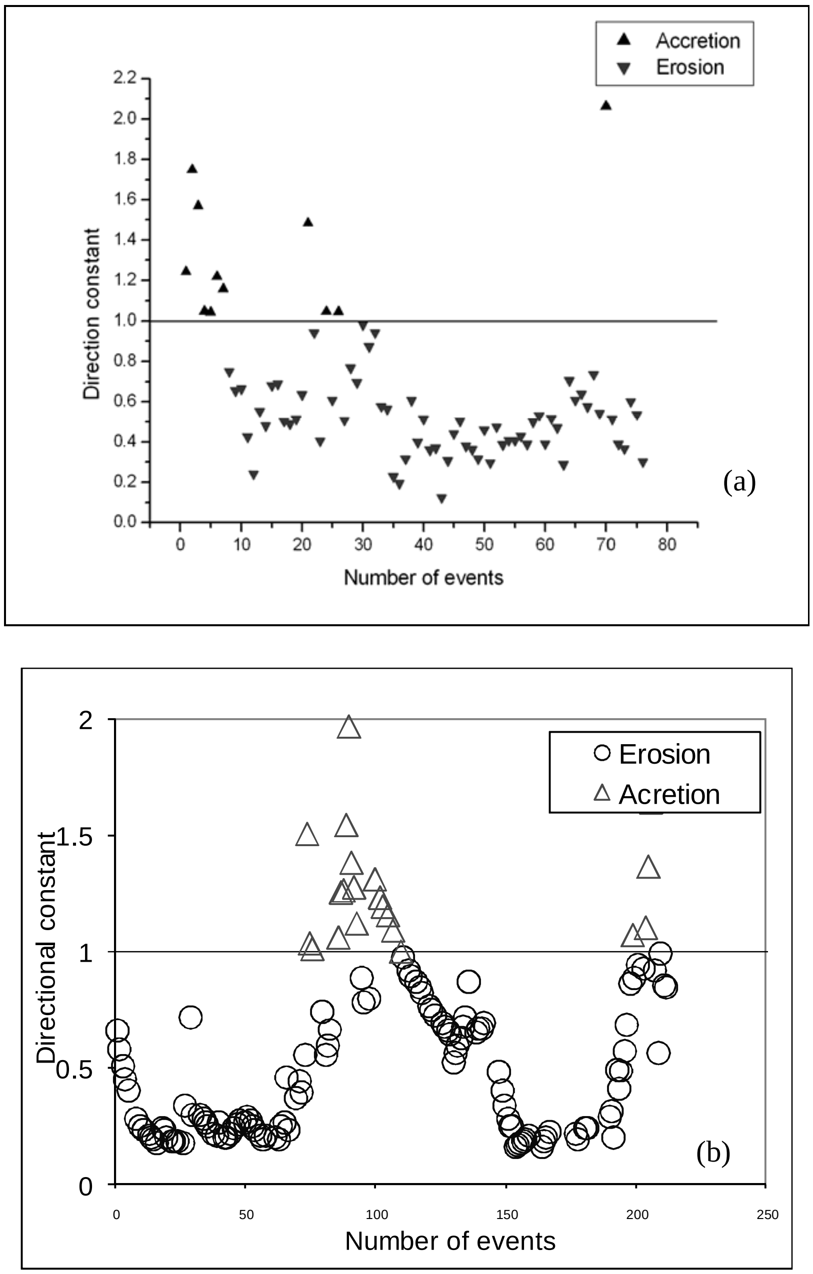


Fig. 5.25 Directional criterion for erosion/accretion for (a) Valiathura for the period10.06.2005 to 26.06.2005 and (b) Calicut for the period 01.07.1981 to 01.08.1981

5.8.1 Statistical method

The performance of the model can be evaluated by calculating the statistical parameters like, bias, correlation coefficient and RMS, from the model simulations. ‘Bias’ measures the difference in central tendencies of the predictions and observations. The correlation coefficient measures the linear relationship between the two variables and Root Mean Square (RMS) error gives a measure of the differences between the predicted and observed values. The model parameters have been calculated from the model results.

5.8.1.1 Performance assessment of Shoreline Change model

The performance assessment was carried out for shoreline change prediction for the sector Valiathura and south of Kayamkulam inlet for the period from May 1981 to May 1982 and from May 2002 to May 2003 (Fig. 5.4 & 5.12). The statistical parameters have been calculated from the model results and are given in the Table. 5.1. It can be seen from the values that the performance of the model is good.

Table 5.1 Statistical parameters of the Shoreline Change model at Valiathura and Kayamkulam

Locations	**Statistical parameters**		
	Bias	**RMS**	**Correlation coefficient**
Valiathura	-2.67	2.94	0.99
Kayamkulam	0.20	0.60	0.99

5.8.1.2 Performance assessment of Profile Change model

The performance of the profile change model was assessed for the simulations made for Valiathura and Calicut, two locations with strikingly different coastal environmental conditions. For Valiathura, the statistical parameters of model simulations were calculated for the first spell of south west monsoon of 2005, whereas the performance of model predictions for Calicut was assessed for the later period of monsoon of July 1981. The statistical parameters calculated from model results are presented in Table 5.2. It can be seen that the model gives very good results in both the locations.

Table 5.2 Statistical parameters of the profile change model at Valiathura and Calicut

Locations	Statistical parameters		
	Bias	**RMS**	**Correlation coefficient**
Valiathura	0.05	0.60	0.96
Kayamkulam	0.02	0.28	0.99

5.8.2 Graphical method

The model predictions of shoreline change at Kayamkulam and profile change at Valiathura were used for the graphical study. The predicted shoreline positions at Kayamkulam were plotted against measured shoreline positions for the period May 2002 to 2003 (Fig. 5.26). It is seen that the observed values are close to those predicted with a correlation coefficient of 0.9

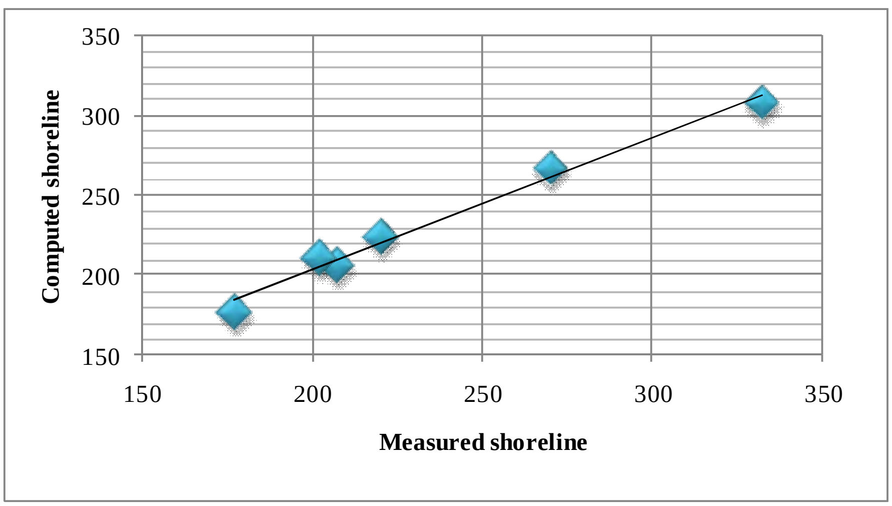


Fig. 5.26 Plot of computed shoreline positions against observed at Kayamkulam

The same exercise was carried out for the profile change model predictions for Calicut for the south-west monsoon of July 1981. The predicted elevations at different locations of the profile are plotted against the observed in Fig. 5.27. It is seen that the observed values are close to those predicted with a correlation coefficient of 0.99.

Hence, it can be conclusively told that the models in its present form can be used to make predictions of beach morphological changes and shoreline changes.

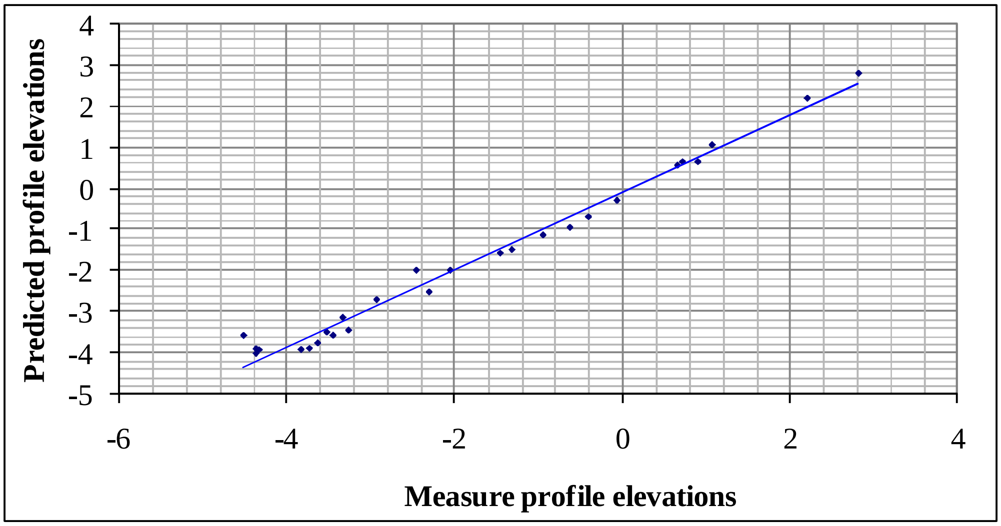


Fig. 5.27 Plot of computed beach elevation against observed at Calicut

5.9 Comparison of Performance with Commercial Model

The numerical models developed have been compared with the commercially available and widely used software for coastal evolution viz., LITPACK. LITPACK (DHI, 2001), a professional engineering software package for the modelling of non-cohesive sediment transport along quasi-uniform beaches, is part of the new generation of DHI software comprising of fully Windows integrated Graphical User Interface. All LITPACK modules apply a fully deterministic approach and allow consideration of many and sometimes dominating factors which are not considered in semi-empirical formulations. The two different modules of LITPACK viz., LITLINE and LITPROF were used for the comparative study. The LITLINE simulates the coastal response to gradients in the longshore sediment transport resulting from natural features and a wide variety of coastal structures. The LITPROF simulates the beach profile in response to episodic events.

5.9.1 Comparisons of Profile Change model with LITPROF model

The LITPROF model was set-up for Valiathura and simulation were carried out for the month of June 2005 for which simulations are available using the profile change model. The scale parameter (SP) and wave breaking parameters Gama1 (G_1) and

Gama2 (G_2) are the calibration parameters used. The model gives best results for values of 0.9 for scale parameter and 0.88 and 0.8 respectively for wave breaking parameters gama1 and gama2 (Shamji et al., 2010). Then the simulation results were compared with the simulated results of “Profile change model” for the same period (Fig. 5.28). It can be seen that the Profile Change Model output is quite comparable to the LITPROF model output.

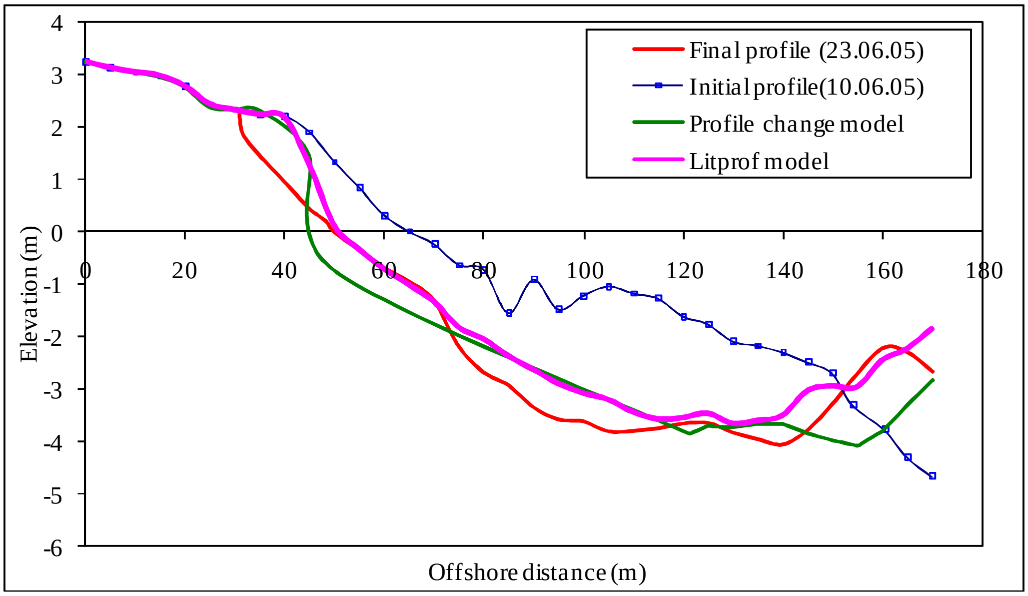


Fig. 5.28 Comparisons of outputs of Profile Change model with LIPROF of LIPACK model for Valiathura

5.9.2 Comparisons of Shoreline Change model with LITLINE model

The simulation studies were carried out for the sector south of the Kayamkualm inlet, where accretion is prevailing all over the season except monsoon. The LITLINE model was setup giving appropriate input data to replicate the actual geomorphologic and littoral environmental conditions.

The main input data are shoreline position, which has been derived from satellite imagery of 2006 (Fig. 5.29) and cross-shore profiles which are extracted from the bathymetric data from C-MAP (Fig. 5.30). The wave climate is defined by giving mean values of wave parameters (Table 5.3) (wave heights, wave periods and wave directions) for different seasons to simulate the wave conditions. The wave data used for the present investigation has been derived from the wave data recorded during different seasons off Kayamkulam inlet during 2004, as discussed in Chapter 4.

Fig. 5.29 Satellite imagery of Kayamkulam inlet and the sector south of it for 2006

Table 5.3 Input wave data for LITLINE model

Season	Duration (% of year)	H_{rms} (m)	Mean Wave Direction (MWD-deg N)	T_z (s)
Pre-monsoon	33.50	0.81	226	8.60
Monsoon	33.00	2.24	246	8.00
Post-monsoon	33.50	0.86	204	9.00

The other important input parameter is sediment characteristics. The sediment characteristics are provided from the field data as discussed in the Chapter 4. The LITLINE sediment transport table generation program, LINTABL is used for the computation of longshore sediment transport. For this computation, sediment characteristics along each of the defined cross-shore profile are given as a line series data file. The input sedimentological data is given in Table 5.4.

Table 5.4 Sediment characteristics defined across the cross-shore profile

Parameter	**Value**
Specific gravity	2.65
Mean grain diameter (mm)	0.15 to 0.3
Fall velocity	0.015 to 0.024
Bed roughness	0.006
Geometrical spreading	1.5

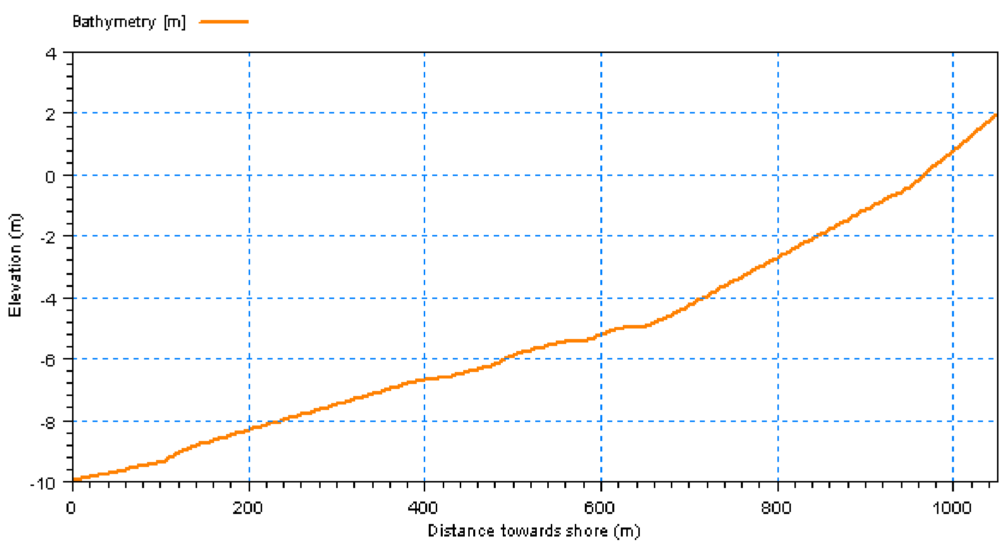


Fig. 5.30 Typical cross-shore profile for the study area.

The model simulations were carried out by varying the model coefficients. The model gives plausible results, which has good correspondence with field signature. The model results indicate the net longshore sediment drift is towards north. For the coast under study, the waves approach from south of the shore normal during pre- and post-monsoon while it is north of the shore normal during monsoon. This in turn could induce predominant northerly longshore currents during pre- and post-monsoon and southerly longshore current during monsoon. The predominace of northerly drift in this coast is corroborated from field observations in the study area (Kurian et al., 2007). The model simulation study was carried out using Shoreline Change Model with same hydrodynamic data used for LITLINE. The simulated results of shoreline change model and LITLINE are presented in Fig. 5.31 for comparison. It is seen that the shoreline predicted by shoreline change model and LITLINE model compare very well. The intense accretion in the adjoining to the breakwater is simulated very well by both the models. This shows the reliability of the shoreline change model developed in this study.

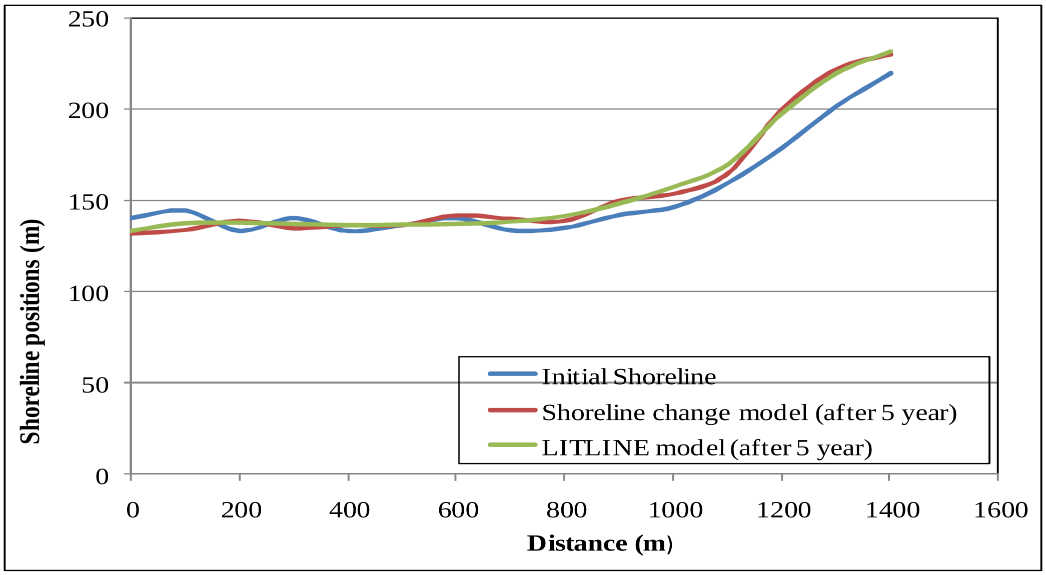


Fig. 5.31 Comparison of simulated shoreline using the Shoreline Change model and LITLINE of LIPACK model for the beach south of Kayamkulam inlet

5.10 Summary

Calibration and validation of the shoreline change and profile change models were carried out using the comprehensive field data collected/collated from different locations of SW coast of India representing different wave energy regimes and geomorphological characteristics. The calibration process involved varying the values of calibration parameters of both the models. The main calibration coefficient, D_c of shoreline change model varies from 15 to 9 m from south to north (i.e., from Trivandrum to Kayamkulam). The calibration coefficient of profile change model also varies spatially. The calibration coefficients K varies from 2×10^{-6} to 0.5×10^{-6}, Ѓ from 0.4 to 0.8 and € from 0.00183 to 0.0280. A criterion proposed by Larson and Krauss (1989) to delineate bar and berm profile has been evaluated with field data corresponding to different environmental conditions. A directional criterion for the prediction of erosive or accretive nature of the beach based on wave characteristics was proposed and validated with field data. The performance assessment of numerical models against observations is an essential part of establishing their credibility and hence the performances of both the models were assessed by calculating different statistical parameters. Also, the performance of the models was compared with a commercially available software pack *viz.* LIPACK. The elaborate calibration/ validation exercises together with the performance assessments and comparative evaluation with the commercial model have established the credibility of the models. The models can be used with confidence for other coastal locations subject to its validation.

06

Applications of the Models for Different Coastal Locations

Chapter 6

APPLICATIONS OF THE MODELS FOR DIFFERENT COASTAL LOCATIONS

6.1 Introduction

The Shoreline Change and Profile Change Models developed as a result of the present study has been calibrated and validated for different coastal locations with a wide range of hydrodynamic, sedimentological and morphological characteristics. Numerical modelling studies on beach morphological changes and its applications are lacking in the Indian scenario. Predictions of beach morphological changes have immense practical applications related to coastal zone management. Such models could help to avert the hazards due to erosion by formulating management plans for mitigation. The capability of Shoreline Change Model to predict shoreline evolution over a period of years becomes very handy in selecting suitable shore protection measure for a coast. The impact of any coastal engineering structure on the adjoining coast can be predicted by the Shoreline Change Model. In this Chapter, the Shoreline Change and Profile Change models have been used to predict the beach morphological changes at selected locations of south-west coast of India.

6.2 Long-term Shoreline Change Predictions

The long-term shoreline change predictions were carried out for two locations viz. Adimalathura and Kayamkulam inlet.

6.2.1 Adimalathura

6.2.1.1 Study area and model domain

Adimalathura, the sector south of Vizhinjam in Trivandrum coast (Fig.6.1) is an almost straight long stretch of beach. The northern end of the Adimalathura shoreline is fronted by a rocky headland and southern end of the shoreline is intercepted by a seasonal inlet. This is an accreting beach. The reason for intense accretion is yet to be realised. The modelling study and field signatures indicate northerly littoral transport. The shoreline predictions were carried out for the region based on shoreline derived for the coast from the 2006 satellite imagery (Fig. 6.2)

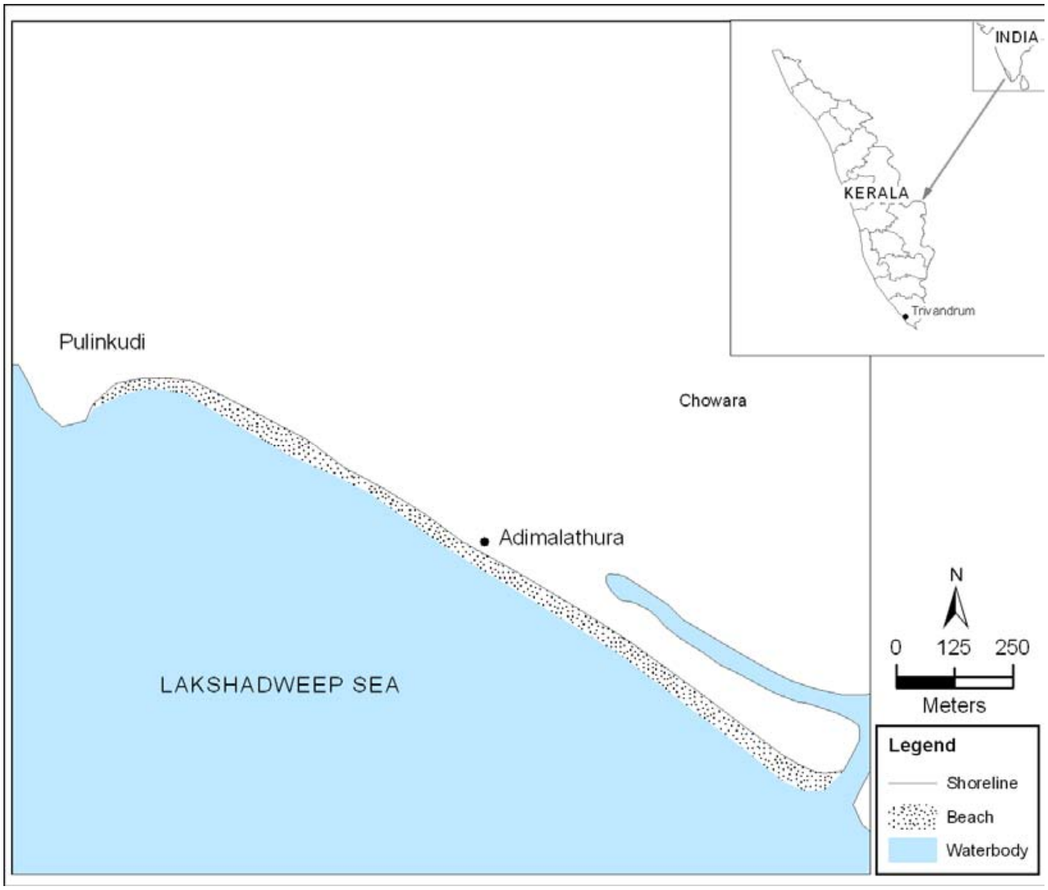


Fig. 6.1 Location map of Adimalathura beach

6.2.1.2 Input data

Since the environmental and morphological conditions of Adimalathura coast are similar to Valiathura, the data available at CESS for Valiathura were used for the numerical model study. The wave data recorded off Valiathura during May 1981 - May 1982 were taken for the study. The characteristics of the data are already discussed in the Chapter 4.

6.2.1.3 Model predictions and discussion

Shoreline change over a period of 5 years was predicted using the Shoreline Change Model. The predicted shoreline at the end of the 5 years' period in comparison with the initial shoreline of 2006 is shown in Fig. 6.2. The initial and predicted shoreline shows accretion for most part of the sector with maximum towards the northern end of the shoreline. The simulated results show a maximum shoreline advance of about 25m in the north. Shoreline retreat is also observed in a few pockets of the beach.

The intense accretion may be due to the blockage of northerly littoral transport by the rocky headland. One of the sediment sources could be the seasonal inlet in the south. An in-depth study is required to unravel the mechanisms of intense accretion including anthropogenic factors.

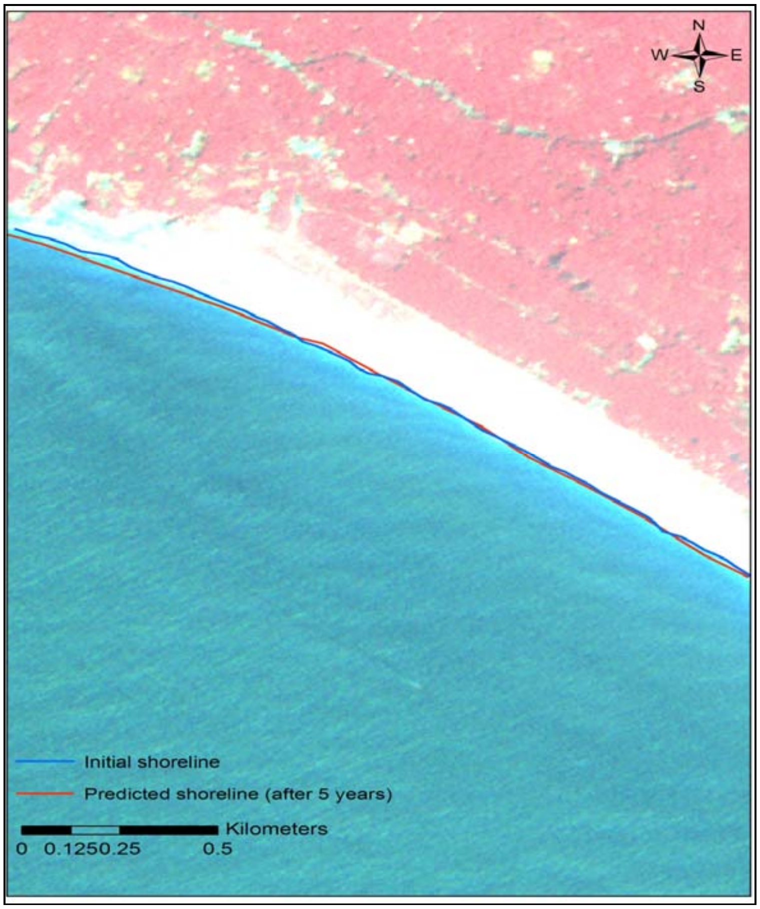


Fig. 6.2 Shoreline for the year 2006 and predicted shoreline at the end of the 5th year at Adimalathura

6.2.2 Kayamkulam

6.2.2.1 Study area and model domain

The details of morphological and hydrodynamic conditions of Kayamkulam inlet area (Fig. 6.3) are discussed in the Chapter 4. As can be seen from Fig. 6.4 the sectors on both the sides of the inlet were undergoing erosion till the construction of breakwaters commenced in 2000. The construction of breakwaters on both the sides of the inlet transformed the whole scenario. With the breakwater in position, the southern side has been transformed to an intensively accreting coast while the northern side has become one of the critically eroding sectors of south west coast of India (Kurian et al., 2007).

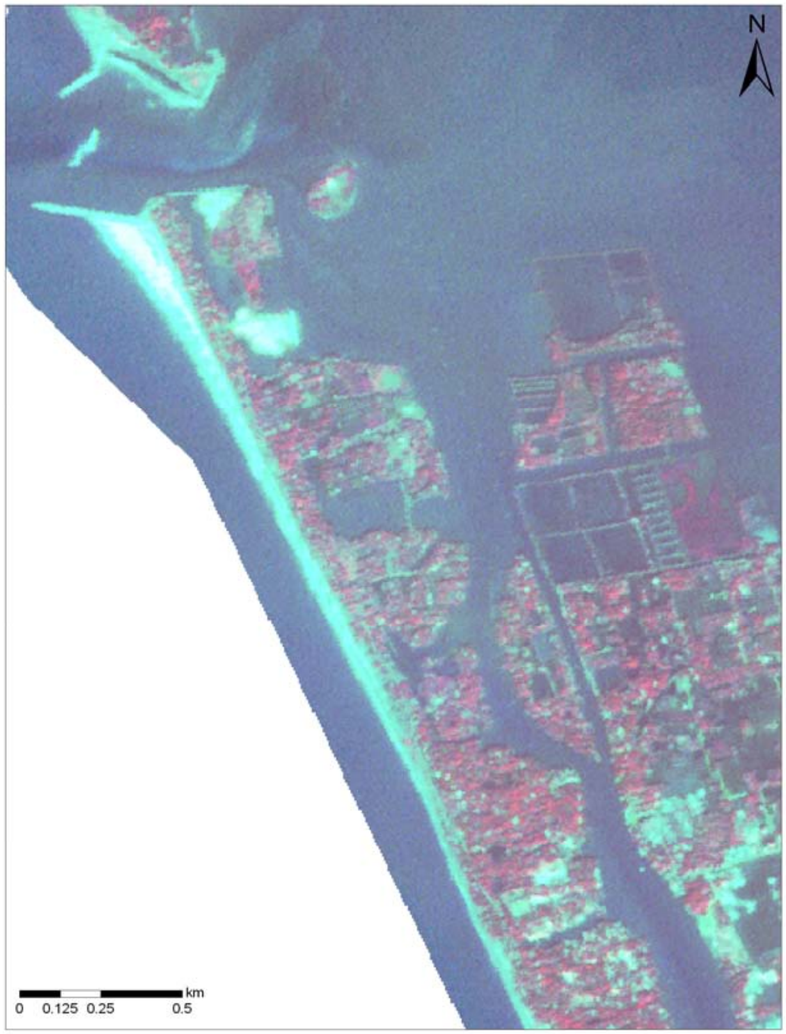


Fig. 6.3 Kayamkulam inlet and the extensively accreted beach sector south of it

For shoreline change prediction the sector of length 3 km south of the Kayamkulam inlet has been selected.

6.2.2.2 Input data

The input data included wave parameters, shoreline position, dimensions of structure, berm characteristics, etc. The shoreline is digitized from the IRS P4 satellite imagery of 2006.

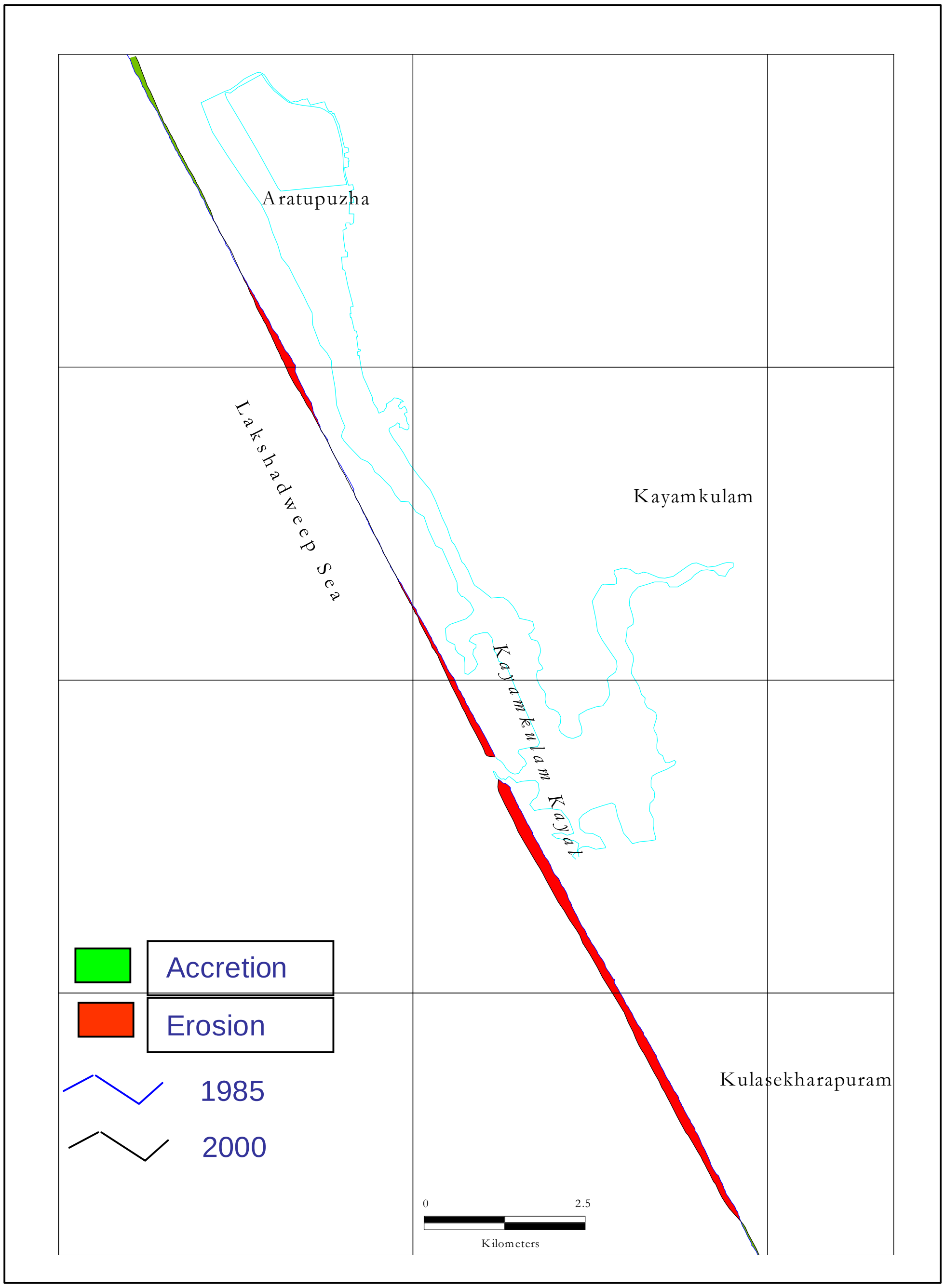


Fig.6.4 Long term shoreline changes in sectors on both the sides of the Kayamkulam inlet before construction of breakwater (after Kurian et al., 2007)

6.2.2.3 Model predictions and discussion

Numerical predictions were carried out for the period of five years. The simulated results are presented in Fig. 6.5. It shows a further advancement of the shoreline by about 25m in the north close to the breakwater which tapers down towards south. However, a couple of locations of accretion followed by erosion are also seen further from a distance of about 1.2km from the breakwater. The intense accretion in the northern portion adjoining the breakwater is due to the groin effect of the breakwater to the predominant northerly longshore current (Chandramohan et al., 1991; Sajeev, 1993; Sanil Kumar et al., 2003). However, unlike the Adimalathura coast, the sector of coast under accretion is very limited. This could be due the fact that this is a sediment deficient coast with very little of external input of sediments (Kurian et al., 2002).

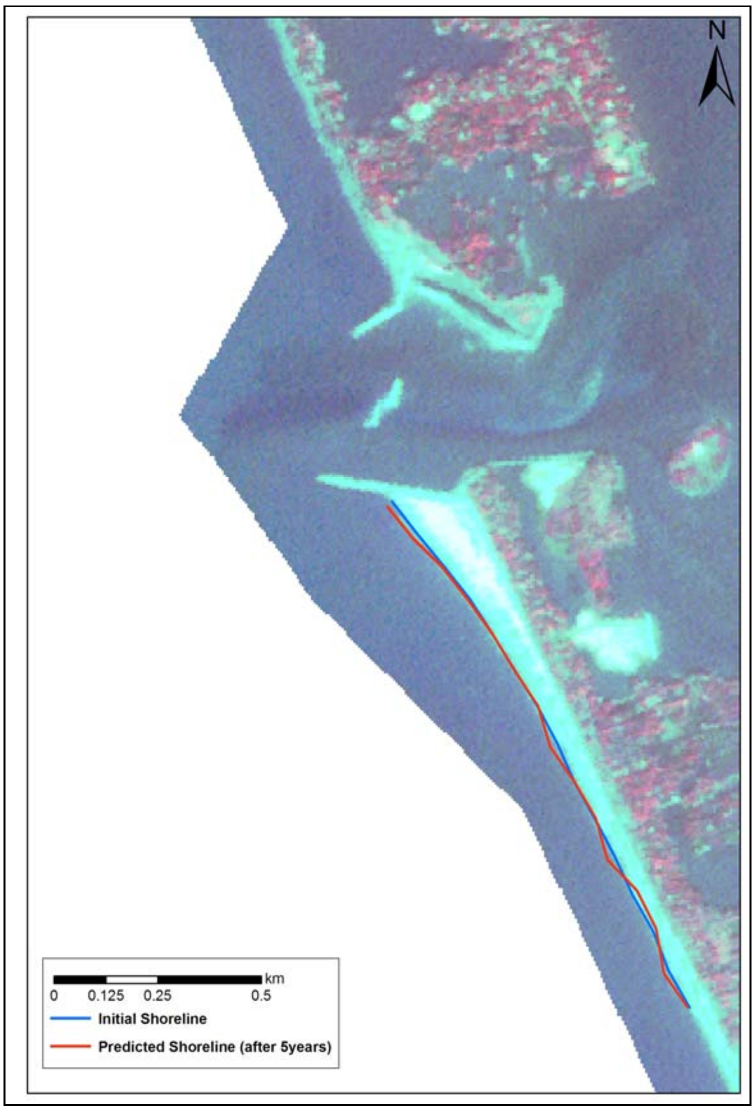


Fig. 6.5 The 2006 shoreline and the predicted shoreline at the end of 5th year south of Kayamkulam inlet

6.3 Short-term Profile Change Prediction

Study of profile change is often based on the onshore/offshore littoral transport. Since the physical processes associated with onshore/offshore transport is highly complex the prediction of profile change is a difficult task. For the southwest coast of India offshore transport of sediments become more predominant during south-west monsoon and hence severe erosion prevails all over the coast during the season. The offshore transport of sediments is well reflected in profile change. The profile change may occur due to the episodic events like south-west monsoon or other storm conditions. The locations chosen for short-term profile change prediction are Valiathura and Calicut which have more or less stable beaches (on a long term basis).

6.3.1 Valiathura

6.3.1.1 Study area

The geomorphological and littoral environmental characteristics of the Valiathura beach are already given in the Chapter 4. Valiathura coast experiences severe erosion during south-west monsoon and the beach is rebuilt during the ensuing fair weather period. The beach is more or less dynamically stable.

6.3.1.2 Input data

The numerical prediction study was carried out for Valiathura for the monsoon season of 2007. A two-weeks period in June 2007 was selected for simulation. The input data included wave parameters, cross-shore beach positions, berm characteristics, sediment characteristics, etc. The beach profile measured on 4^{th} June 2007 (Fig. 6.6) has been used as the initial profile for the study. The profile shows typical characteristics of a berm profile since it pertains to the period before onset of the first monsoon spell.

6.3.1.3 Model predictions and discussion

The predicted profile at the end of a 14 day period from 4^{th} June 2007 is given in Fig. 6.8. The simulated profile shows high erosion at the beach face and subsequent deposit of sediment at offshore region (Fig 6.7). The simulated profile shows the characteristics of a bar profile, due to the impact of monsoon waves. The simulated profile shows a retreat of shoreline by 30 m and erosion of 1694 m^3/m of sand from the beach face coupled with a deposition of 1009 m^3/m in the offshore leading to the formation of bar. The simulated bar height of 2 m is in good correspondence with the field observations made by Thomas et al. (1988).

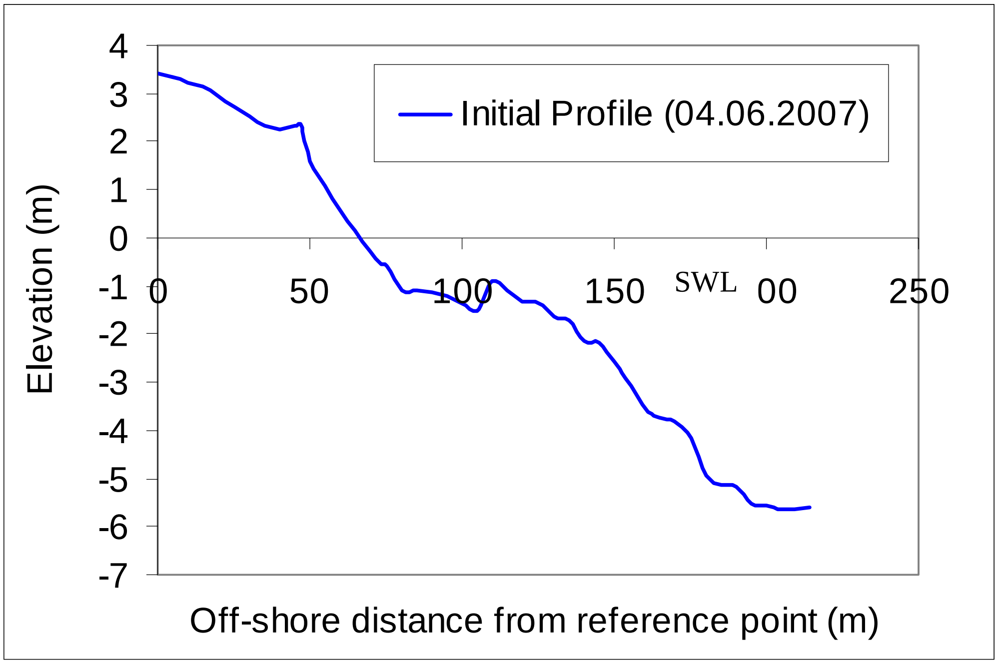


Fig. 6.6 Beach profile at Valiathura pier on 4th June 2007

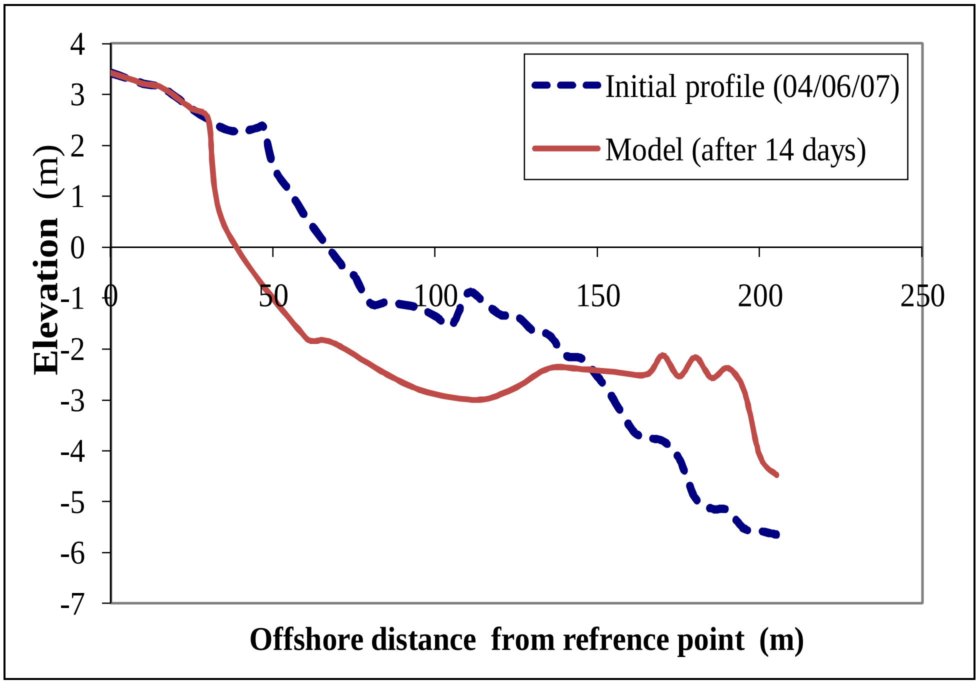


Fig. 6.7 Predicted beach profile at Valiathura pier at the end of a 14 day period starting from 04.06.2007

6.3.2 Calicut

6.3.2.1 Study area and input data

The geomorphological and littoral environmental characteristics of the Calicut beach are already given in the Chapter 4. The beach (Fig.6.8) is comparatively wide, with a foreshore of moderate slope. Beach profile data for the study were collected from secondary data available at CESS. The profile data collected on 1st July 1982 is used as the initial profile (Fig.6.9).

Fig. 6.8 A recent view of Calicut beach; the pier which was used as a platform for beach profile measurements and other littoral environmental observations in Eighties are in a dilapidated stage now

6.3.2.2 Model predictions and discussion

The model simulation has been carried out for duration of two weeks from 1st July 1982. The predicted profile together with the initial profile is given in Fig. 6.10. The simulated profile shows erosion in the beach face (393 m^3/m) and accretion in the offshore region (593 m^3/m) showing the bar formation as a result of high intensity monsoon wave. About 10 m retreat was observed in the shoreline. The bar formed in the offshore at a distance of about 80 m from the berm has a height of about 2.1 m, which is well corroborated with the observations made by Kurian et al. (1988) for this location.

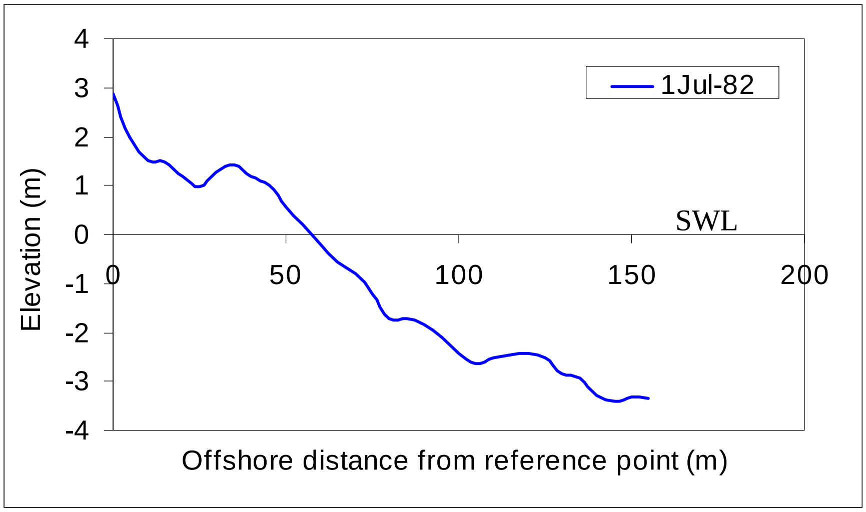


Fig. 6.9 Beach profile on 1st July 1982 at Calicut

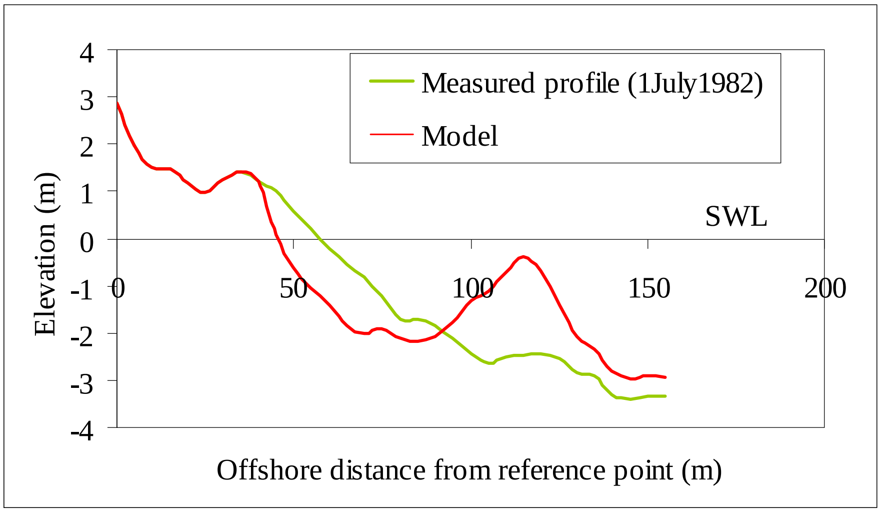


Fig. 6.10 Model predictions of beach profile change at Calicut using Profile Change model

6.4 Summary and Conclusion

Prediction of shoreline change (long-term) and profile change (short-term) has many practical applications related to the coastal zone management and coastal engineering projects. The demonstration of the calibrated/validated Shoreline Change and Profile

Change Models for prediction of shoreline change and profile change for sites of varying coastal geomorphological and hydrodynamical characteristics has been undertaken. The long-term shoreline change predictions have been carried out for Adimalathura, south of Trivandrum, and the southern sector of Kayamkulam inlet. The intense accretion observed at Adimalathura and Kayamkulam were reproduced by the model. The predicted shoreline shows good correspondence with field signature. Short-term predictions in beach profile change have been carried out for Valiathura and Calicut with differing littoral environmental characteristics. The change of berm profile to bar profile during monsoon season is well reproduced by the Profile Change model. The results demonstrate the applicability of the Models for various applications.

07

Summary and Conclusions

Chapter 7
SUMMARY AND CONCLUSIONS

7.1 Summary

Development of capabilities for beach morphological change prediction has immense application in coastal zone management. Though there have been some isolated efforts in the Indian scenario towards development of predictive capability they were not successful. Even the commercially available models and free software that are available have not been calibrated/validated for our coast. Since coastal erosion is a major problem faced by some sectors of the vast coastline of the country, there was a need to develop shoreline change and beach morphological change models applicable for our coast. The development of such models assumed greater importance in view of the increasing developmental activities along the vast coastal zone of the country. Hence an investigation on numerical modeling of beach morphological changes was felt very important and was undertaken with the following objectives:

- Study the wave characteristics and beach processes of a few selected sites of SW coast of India
- Develop a profile change model to predict short-term profile changes due to episodic events
- Develop a shoreline change model to predict the long-term changes in the shoreline
- Application of these models for different coastal locations

As part of the investigation a comprehensive review of the available literature on the beach morphological changes and its modelling was carried out. There are several studies on beach morphological changes due to cross-shore as well as longshore sediment transport. Although most of the major factors affecting beach morphology have been considered in the models, there are still many factors that are poorly understood which restrict the extension of models to simulate the beach processes. Cross-shore sediment transport models are mainly used to model beach profile changes. Models are reviewed based on their theoretical basis (mainly sediment transport) and the extent to which they were verified. Still many models are undergoing constant development and improvement and each model may be the best for a specific case (purpose) or under specific conditions. It is found that most studies on shoreline change models are based on the one-dimensional sediment balance equation which is referred to as one-line model equation. These models have the advantage of being very fast and they can predict long-term shoreline changes very well after suitable calibration. However, they cannot accurately predict the impact of morphological changes in the vicinity of coastal structures that occurs due to short-term events. Although a number of beach morphological change models have been developed in the global scenario, no such specific model has been developed, calibrated and verified in the field conditions of the country.

Based on the two modes of sediment transport viz. alongshore and onshore/offshore transport, two numerical models have been developed to simulate long-term and short-term beach morphological changes. To predict long-term change of shoreline, a numerical model developed based on the approach of Kraus and Harikai (1983) is proposed. This model can be used to predict shoreline change in the vicinity of coastal structures. Beach profile change model has been developed based on the concepts of Larson and Krauss (1989). This model can be used to predict short-term profile change due to episodic events. Both the models were developed in FORTRAN to facilitate faster computations to solve the equations.

The selection of coastal locations for calibration/validation of the model was carried out based on criteria such as bathymetry, wave energy, sediment characteristics, erosion / accretion scenario, etc. Various locations in three representative coastal sectors of Trivandrum, Alleppey and Calicut of southwest coast of India were selected for the field data collection. A comprehensive field measurement programme taking care of the primary data requirements for the investigation was meticulously planned and implemented. The secondary data were collated from the available data of CESS. Wave, beach profile and sedimentological data for different seasons like monsoon, pre-monsoon and post-monsoon were collected/collated. The field data were processed to understand the hydrodynamic, beach morphodynamics and sedimentary processes of the locations.

Analysis of data shows considerable spatial variation in wave intensity in consonance with the available literature. Seasonal variations are characterised by steep waves of higher wave heights, shorter wave periods and westerly direction during peak monsoon and long period swells from the SSW-SW directions during the pre- and post-monsoon seasons. Analysis of beach profile data brings out the seasonal and spatial variations in the erosion/accretion processes brought about by the natural as well as anthropogenic factors. Irrespective of location the beach shows maximum width during the fair weather season due to the onshore transport of sediments by the long period swells and reaches its nadir during the peak of monsoon due to the offshore transport of sediments by the steep monsoonal waves. Human-induced activities such as construction of shore structures, sand mining brings out anomalies in erosion/accretion pattern within the same location. The beach sediment characteristics show considerable spatial variation in tune with the hydrodynamic regime and beach morphological characteristics. The Trivandrum coast with the highest energy level is characterised by the coarsest sediments and the Calicut coast with the lowest energy regime by the finest sediments. At each location across-the-profile and seasonal variations in the sediment characteristics are also seen. It can be seen that the data set used for model calibration/validation pertain to a wide range of coastal environmental conditions representing different energy regime and geomorphological characteristics thereby ensuring the reliability and credibility of the calibration process.

The model calibrations were carried out using the comprehensive field data representing different energy regimes and geomorphological characteristics. Calibration achieves two main outcomes; firstly, the level of confidence in the model predictions is established and secondly, the best possible accuracy is obtained after the calibration is carefully done and the model is properly optimised. The calibration process involved varying the values of calibration parameters of both the models. The main calibration coefficient, D_c of shoreline change model varies from 15 to 9m from south to north (i.e., from Trivandrum to Kayamkulam). The calibration coefficient of profile change model also varies spatially. The calibration coefficients K varies from $2x10^{-6}$ to $0.5x10^{-6}$, Ѓ from 0.4 to 0.8 and € from 0.00183 to 0.0280. A criterion proposed by Larson and Krauss (1989) to delineate bar and berm profile has been evaluated with field data corresponding to different environmental conditions. A directional criterion for the prediction of erosive or accretive nature of the beach based on wave characteristics was proposed and validated with field data. The performance assessment of numerical models against observations is an essential part of establishing their credibility and hence the performances of both the models were evaluated by calculating the different statistical parameters. Also, the performance of the models was compared with a commercially available software pack viz., LIPACK. The elaborate calibration/ validation exercises together with the performance assessments and comparative evaluation with the commercial model have established the credibility of the models. The models can be used with confidence for other coastal locations subject to its validation.

Prediction of shoreline change (long-term) and profile change (short-term) has many practical applications related to the coastal zone management and coastal engineering projects. As part of this investigation, the demonstration of the calibrated/validated Shoreline Change and Profile Change Models has been undertaken for two sites each of varying coastal geomorphological and hydrodynamical characteristics. The long-term shoreline change predictions have been carried out for Adimalathura south of Trivandrum and the southern sector of Kayamkulam inlet. The intense accretion bserved at both these locations were reproduced by the model. The predicted shoreline show good correspondence with field signature. Short-term predictions in beach profile change have been carried out for Valiathura and Calicut with differing littoral environmental characteristics. The change of berm profile to bar profile during monsoon season is well reproduced by the profile change model. The model study brought to light the need for numerical simulations while planning coastal structures. The results demonstrate the applicability of the models for planning mitigation measures against coastal erosion.

Thus the investigation has realized all the objectives set while taking up the research problem. Numerical models for beach morphological change suitable for the south-west coast of India have been developed. The models have been calibrated and validated using comprehensive field data pertaining to a wide range of geomorphological and environmental conditions including the peak monsoon wave data. The performance of the model has been assessed using standard procedures in addition to comparing its performance with a commercial model and found to be good. Hence the model can be applied with confidence to other parts of the country subject to its validation. Evaluation of bar/berm criterion in the field conditions has been done for the first time in the country and is one of the few such studies done internationally. An important outcome of the investigation is the proposal of a Directional Criterion for prediction of erosional/accretional nature of a beach in response to waves.

7.2 Recommendations for Future Work

Through the present investigation major inroads have been made towards the development of numerical models for short-term and long-term beach morphological changes for the south-west coast of India. However, the work could be improved upon in the coming years by taking care of some of the limitations of the present study. Future research in the field of beach morphological change modeling could be channelized in the following directions:

- The models at present do not have a provision for input of sediment textural characteristics. The models have to be suitably modified to incorporate this.
- The Depth of Closure is an important input parameter of the Shoreline Change model. Estimation of Depth of Closure is a topic on which further research can be done. The parameters such as grain size, sediment transport rate and topography can be incorporated in the above equation as is done by Hallermeier (1978).
- The profile change model mainly simulates erosions due to episodic events like monsoon, cyclones etc. The capability of the model to simulate accretion also has to be explored.
- At present the domain of the Shoreline Change Model is only the surf zone. The model can be suitably modified to extend the domain to the nearshore. Such an extension will facilitate the applicability of the Shoreline Change Model for study of nearshore sediment dynamics, identification of sediment cells, etc.

- Many of the beaches are fronted by sea walls. In such conditions longshore transport rates estimated using the present models are overestimates. In future studies this aspect may have to be taken care of.
- Longshore transport models are very sensitive to wave breaker angles. An accurate directional wave measuring device should be made an essential component of the instrumentation system for measurement of hydrodynamic data.
- The shoreline change model can be further improved by providing provisions for incorporating the shoreline change due to shore-detached structures like offshore breakwater, submerged breakwater, etc. This will enhance the applicability of the model for a wider range of problems.
- Wind is an important forcing factor as far as nearshore processes are concerned. The models should be appropriately modified to include this important parameter.

It is earnestly hoped that further research along the above lines will go a long way in enhancing the credibility, reliability and applicability of the models for various applications in coastal zone management.

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