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1 Introduction

Alappuzha in kerala has a history of maritime activity. The town itself was planned with a network of navigable channels. It is blessed with the famous back waters, to which tourists comefrom all over the world. The City had a loading and unloading jetty on its coast which was operated until 1989. During the later end of 1990's kerala has caught the imagination of the world as a ideal tourist spot. Since then the number of tourists coming to Kerala has increased by leaps and bounds. Aleppy is one of the spot which has attacted a very good share of Tourists coming to Kerala. Almost all the tourists who come to Kerala do not return back without visiting the back wataers of Alappuzha. Keeping in view the anticatipated high growth rate of the number of tourists coming to the state which implies growth in the number of people visiting alappuzha via cochin, the government of kerala has appointed a consultant to evaluate the techno econimic feasibilty of developing a passenger ferry terminal and a container berth in Alappuzha.Towards this pupose two different layouts were developed by the consultants.

Both these layouts differ from each other only in the alignment and length of the break water. The layouts are shown in figures 1.1 and 1.2.It was decided to conduct numerical model studies to assess the sustainability and to arrive at the optimum length and alignment of the proposed breakwaters. Accordingly, following would be the scope of work, which is described in detail in this report.

- Analysis of the site conditions and the bathymetric data available
- Numerical modeling of sedimentation processes including estimating the littoral drift, studying the sedimentation pattern, impact of the breakwater on the sedimentation pattern.
- Numerical modeling of wave disturbance studies to assess the impact of diffraction waves on the harbour tranquility.
- Providing conclusions and recommendations on the alignment of the breakwater based on the above studies
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The objective of the above studies is to

- Simulate waves approaching the proposed cargo harbor with breakwaters and assess tranquility for the cargo harbor layout.
- Shoreline transformation simulations to determine the likelihood of erosion and accretion, leading to recommendations to minimize shoreline changes
- Estimation of maintenance dredging quantities.

1.1 Description of the Numerical models used

1.1.1 Tranquility studies

Tranquility model (wave disturbance model) is used to estimate the amount of agitation caused by the wave once it enters the basin. Bouss 2D was used for this purpose in this study. Bouss 2d can be accessed through SMS modeling suite.

BOUSS-2D is a comprehensive numerical model for simulating the propagation and transformation of waves in coastal regions and harbors based on a time-domain solution of Boussinesq-type equations.

The governing equations are uniformly valid from deep to shallow water and can simulate most of the phenomena of interest in the nearshore zone and harbor basins including:

- Shoaling/refraction over variable topography
- Reflection/diffraction near structures
- Energy dissipation due to wave breaking and bottom friction
- Cross-spectral energy transfer due to nonlinear wave-wave interactions
- Breaking-induced longshore and rip currents
- Wave-current interaction
- Wave interaction with porous structures

The governing equations in BOUSS-2D are solved in the time domain with a finite-difference method. Input waves may be periodic (regular) or non-periodic (irregular) and both unidirectional and multi-directional sea states may be simulated. Waves propagating out of the computation domain are either absorbed in damping layers or allowed to leave the domain freely.

1.1.2 Morphological Model Studies

Morphological model is used to estimate the effect of the breakwater on the surrounding area, changes in the accretion and erosion patterns due to the presence of breakwater and amount of sedimentation in the approach channel i.e. maintenance dredging in the approach channel. The morphological model was setup using CMS Flow and CMS Wave Models. Both these wave and flow models are in interactively to estimate morphological changes or littoral drift due to waves. These two models can be accessed through SMS modeling suite.

CMS-Flow is a finite-volume numerical engine which includes the capabilities to compute both hydrodynamics (water levels and current flow values under any combination of tide, wind, surge, waves and river flow) sediment transport as bed load, suspended load, and total load, and morphology change. The interface in SMS allows the user to set up and edit computational grids, specify model parameters, define interaction of this model with the wave counterpart (CMS-Wave), launch the model and visualize the results.

CMS-Wave is a 2-D wave spectral transformation (phase-averaged) model. The term "phase-averaged" means that it neglects changes in the wave phase in calculating wave and other near shore processes. This class of wave models represent changes that occur only in the wave energy density.

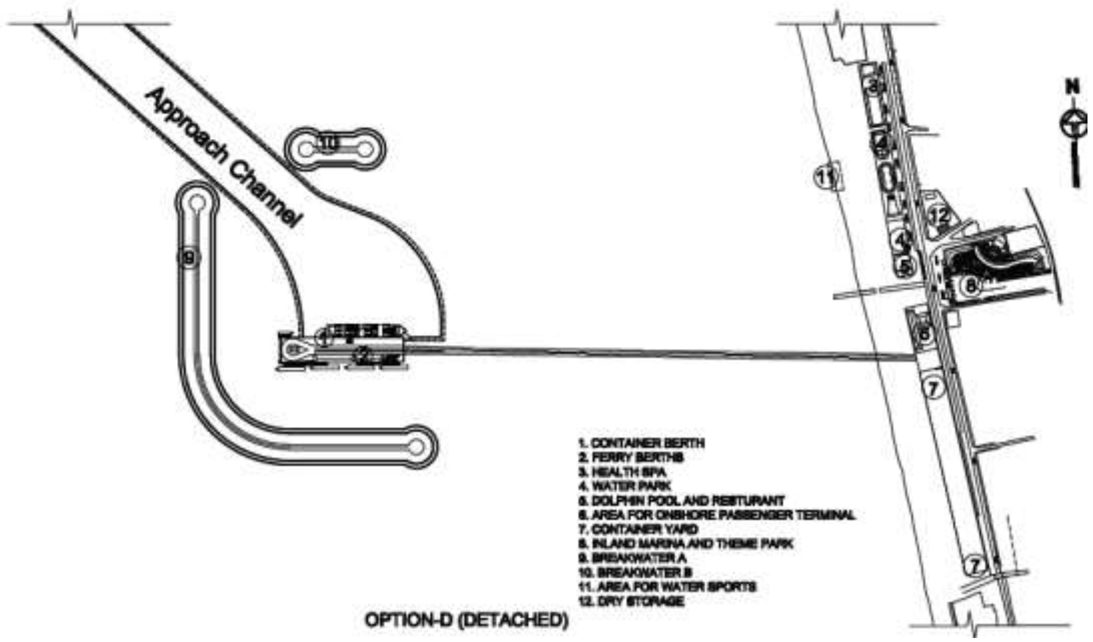

Figure 1.1 . Layout with detached Breakwater

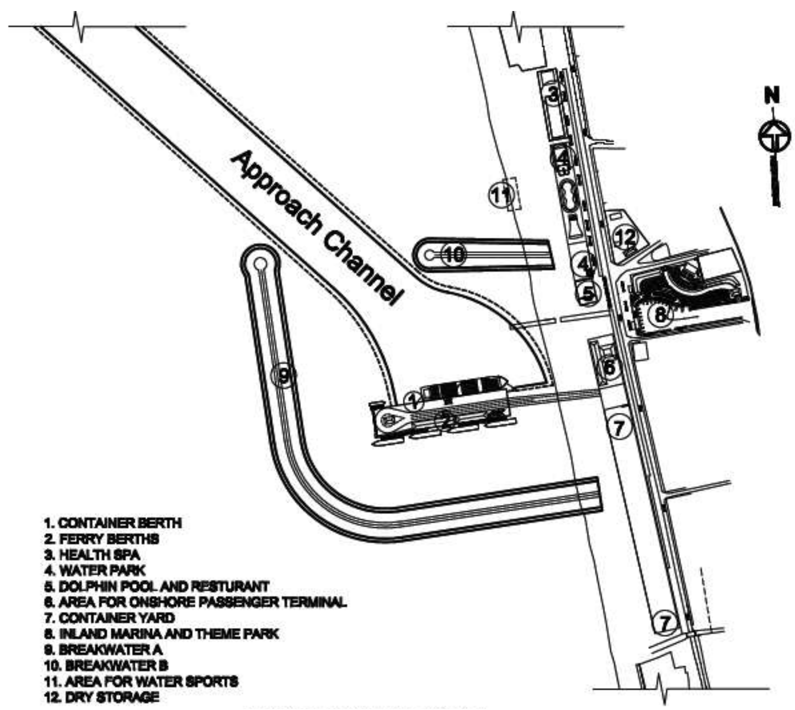

Figure 1.2. Layout with attached Breakwater

2 Site Conditions

2.1 Humidity

The relative humidity in Alappuzha is high compared to other parts if India and it varies in between 75% in non monsoon periods and 90% in monsoon periods.

2.2 Temperature

Temperature at Alappuzha varies from about 22^{0}C to 310C. There are not much distinct seasonal variations in the temperature, which more or less uniform throughout the year. Highest temperatures tend to occur in the months of March to May.

2.3 Oceanographic conditions

2.3.1 Tides

Alappuzha experiences semi diurnal tides. The tidal level with respect to chart datum as per naval Hydrographical chart No.2035 is as follows.

Table 1.3. Tide Data

Mean Sea Level, MSL	+0.60m
Mean Highest High Water Level, MHHWL	+0.90m
Mean Highest Low Water Level, MHLWL	+0.60m
Mean Lowest High Water Level, MLHWL	+0.80m
Mean Lowest Low Water Level, MLLWL	+0.30m

2.3.2 Waves

As per review by T.S. Shahul Hameed on wave data recorded from 3rd July 1980 to 31st December 1984(Chapter 4, OCEAN WAVES AND BEACH PROCESS,1988) in which wave data recorded for two periods in a year rough season (May-Oct) and fair season (Nov-Apr), the following values were arrived (Table 1.4). The waves will be more or less parallel to the shore during rough season and the effect of refraction will be smaller as the periods are lower. The large period waves were observed coming from directions around 255oN during both the seasons and high waves with small period were more or less parallel to the shore.

Table 2.1. Observed Wave Characteristics

Parameters	Rough season	Fair season
Significant wave height, Hs	3.00m	1.40m
Height of the highest wave, H_{max}	3.40m	2.00m
Time period of predominant waves	8-10s	9-11s
Percentage of times H_s exceeds specified wave heights		
30%	1.30m	0.65m
50%	0.95m	0.52m
75%	0.62m	0.42m
Percentage of times H_{max} exceeds specified wave heights		
30%	1.90m	0.90m
50%	1.35m	0.72m
75%	0.85m	0.58m
Wave Direction considering majority of waves	230-265^{o}N	235-240^{o}N
Wave direction range and percentage of times that rage is experienced		
	245-260^{o}N, 50%	230-245^{o}N, 44%
	235-265^{o}N, 83%	235-255^{o}N, 61%
Predominant wave direction and time period	250-265^{o}N, 8-10s	230-240^{o}N, 10-11s

Table 2.2. Wave Height Distribution

	Percentage Exeedence			
Wave Height (m)	**Rough Season**		**Fair Season**	
	H_s	**H_{max}**	**H_s**	**H_{max}**
0.25	95	98	96	98
0.50	82	89	59	86
1.00	47	68	5	20
1.50	22	42	0	8
2.00	8	20	0	0
2.50	4	14	0	0

The maximum wave breaking height each year didn’t vary much and it was around 2.9 to 3.3m. The braking wave height increases from April to June to reach maximum and start decreasing from August and remains low. During the period of intense wave activities wave starts spilling even beyond 1 km from shore, and they surge on beach face during September/October. Plunging type of breaking is observed rest of the period.

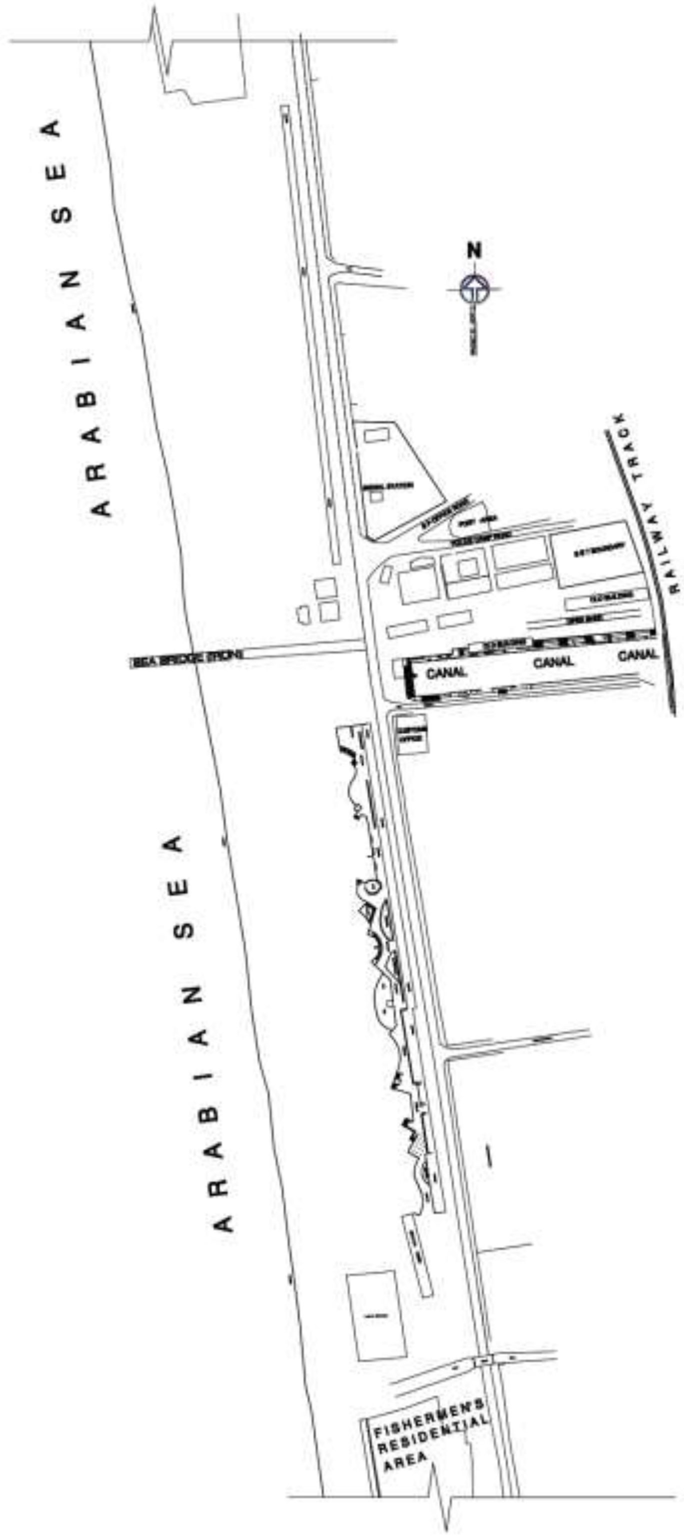

Figure 2.1: Existing Location

3 Morphology study

The waves approaching at an angle to the shore generates long shore currents which causes transport of non cohesive sediments along the fore shore which is called littoral drift. The morphological changes in the alleppy region are by virtue of this phenomenon. So to study the morphological changes in the region due to construction of break water a non cohesive transport mopdel was setup using CMS Flow and CMS Wave softwares. Mean Tidal variation in alleppy is in the order of 0.6 meters which implies that the tidal currents generated are very low to significantly contribute to the morphological changes in the area of interest. Keeping this in view, tidal variation is neglected and the numerical modelling was conducted at the mean sea level of 0.6 mters.As part of the study one model was setup to study the existing conditions and two more models were setup to study the effect of the construction of attached and detached break waters respectively.

3.1 Bathymetry

Bathymetry survey done for Alleppy Port area in 1999 and the admiralty chart no. 2035 were used for the generating the bathymetry for this model. The size of the model domain is 11.2 kms X 12.2 kms. CMS Wave and CMS flow models are run on a Cartesian grid. The bathymetry was generated using a Grid size of 100mX 100m.The final bathymetries used for this study are shown in figures 3.1 to 3.3. It should be noted here that positive depth is depth below CD and negative depth is depth above CD. The bathymetry without any development proposed for existing condition is shown in figure 3.1. Bathymetry with detached break water is shown in figure 3.2. Bathymetry with attached breakwater is shown in figure 3.3.

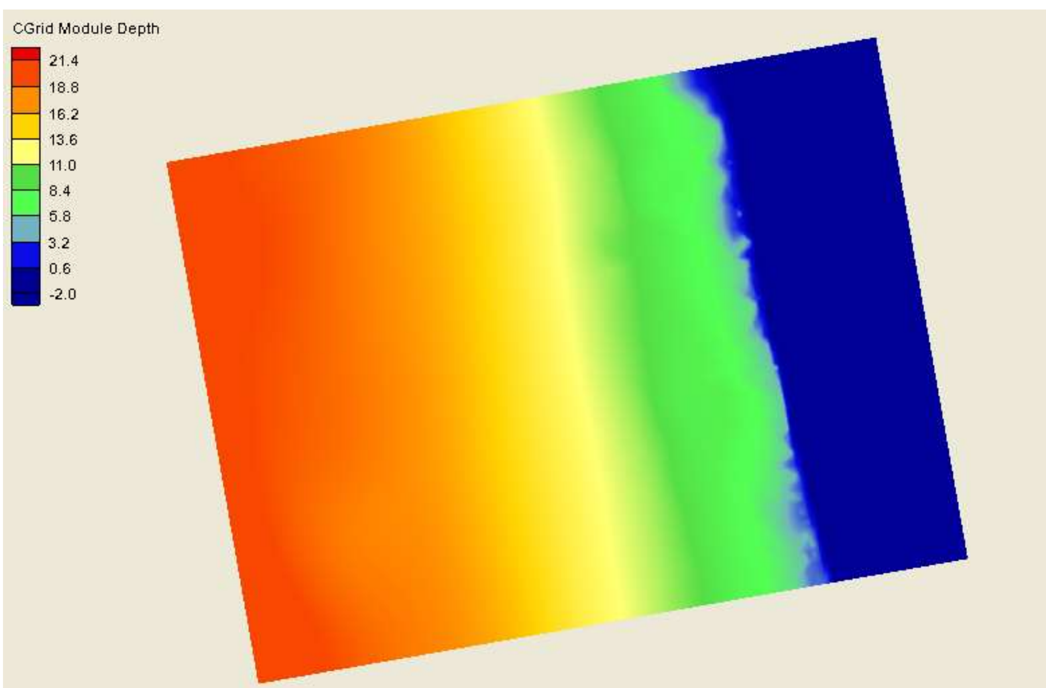

Figure 3.1. The bathymetry of the domain area without any development proposed

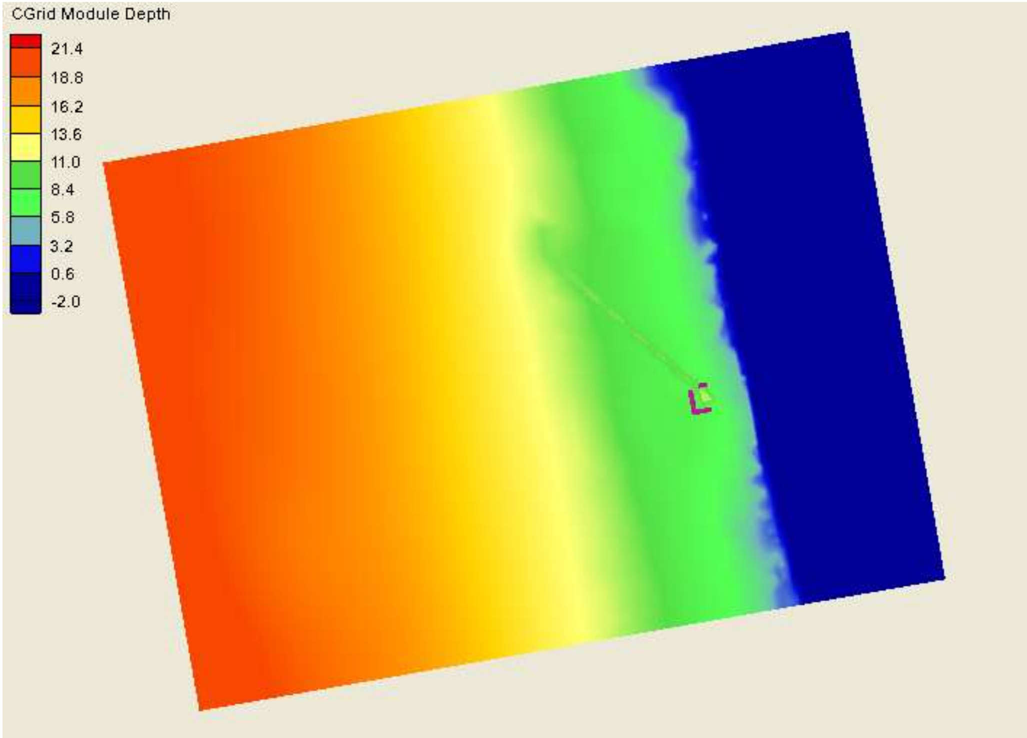

Figure 3.2. The bathymetry of the domain area with detached break water and approach channel

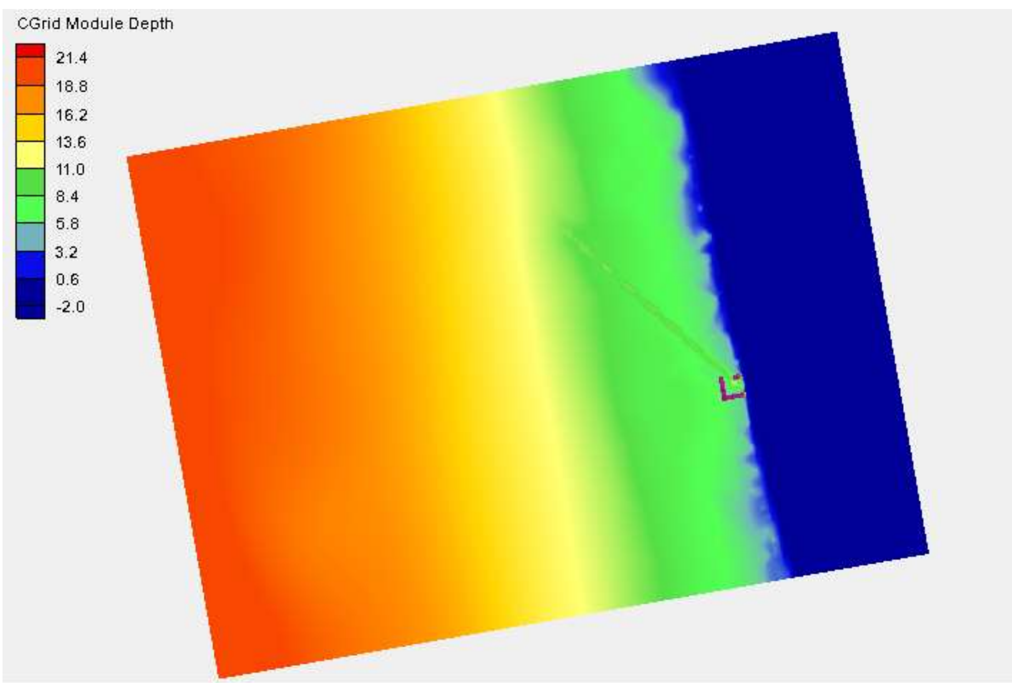

Figure 3.3. The bathymetry of the domain area with attached break water and approach channel

3.2 Boundary conditions

In this model the boundary conditions are given to the model by using cell strings. The model domain with the cell strings at boundaries is shown in the figure 3.4. The purple line denotes the incident wave boundary condition. Wave conditions as described below are applied all long this boundary.The orange colour lines shown on the lateral boundaries denote the no flow boundary condition applied all along the length of the boundary.

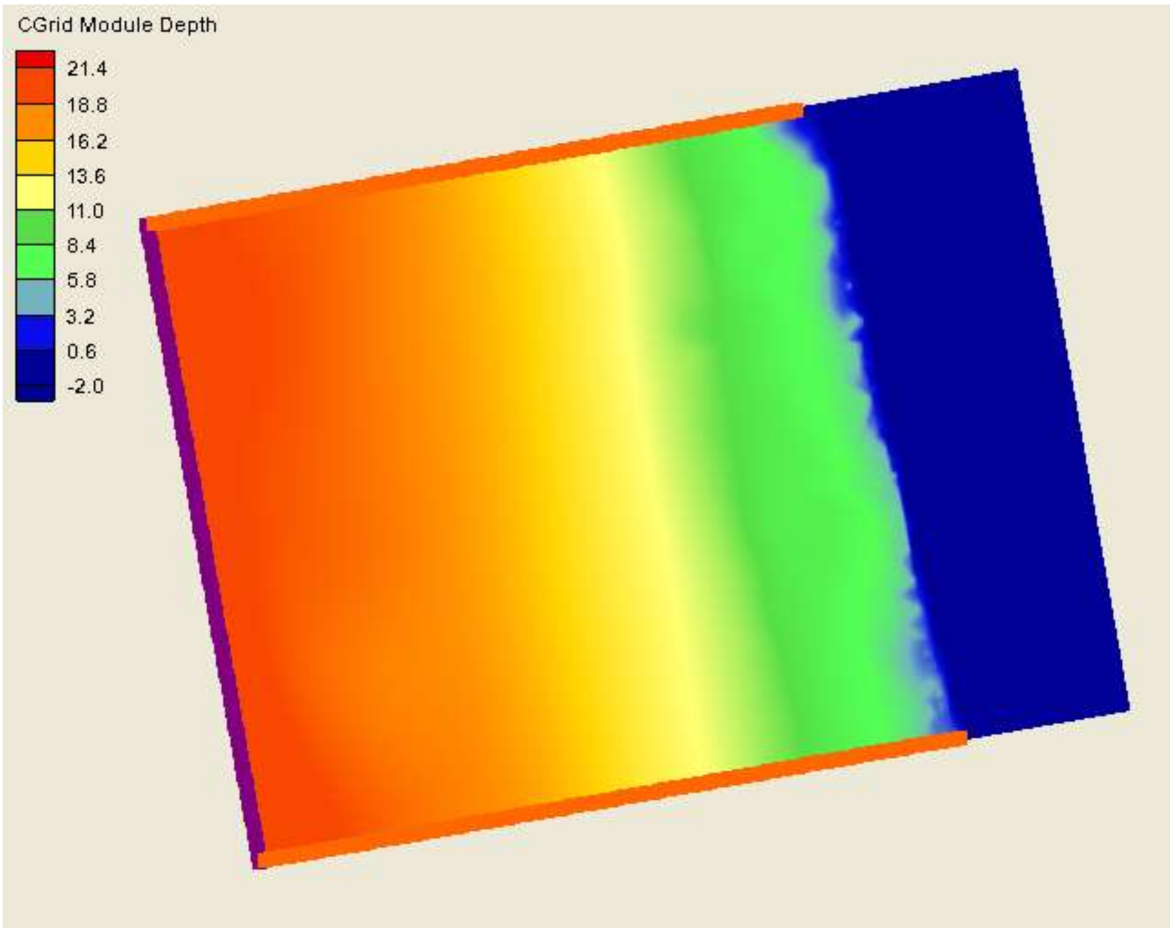

Figure 3.4. The domain with the boundary conditions applied

The input wave data used for the setting up the morphological model was taken from review by T.S. Shahul Hameed on wave data recorded from 3rd July 1980 to 31st December 1984(Chapter 4, OCEAN WAVES AND BEACH PROCESS,1988) in which wave data recorded for two periods in a year rough season (May-Oct) and fair season (Nov-Apr). The maximum significant wave height (H_s) during the rough season and fair season are 3.0 and 1.4 m respectively.

The wave observations as reported in the review mentioned in the paragraph above were taken at a distance a 500 meters into the sea from the shoreline. The effect of refraction on the direction of the waves is very low during the rough season and in general the refraction effect is also lower on the waves because the the lower time periods of the waves approaching the coast . keeping this low refraction effect in view, the wave observations as reported in the review were applied at the offshore boundary which is situated at a distance of 12 kms from the shore.

Scatter diagrams showing the relation between time period, significant wave height and direction of wave are given in figures 3.5 and 3.6 for both fair and rough seasons. This wave data is obtained by taking observations for every three hours. The annual wave climate for the model is obtained by using this scatter information. The three hour interval wave data is transformed to 12 hour interval data and used in the model as input waves. This is done to reduce the time of running. The final wave heights applied at the boundary are shown in figure 3.7

3.2.1 Fair Weather waves

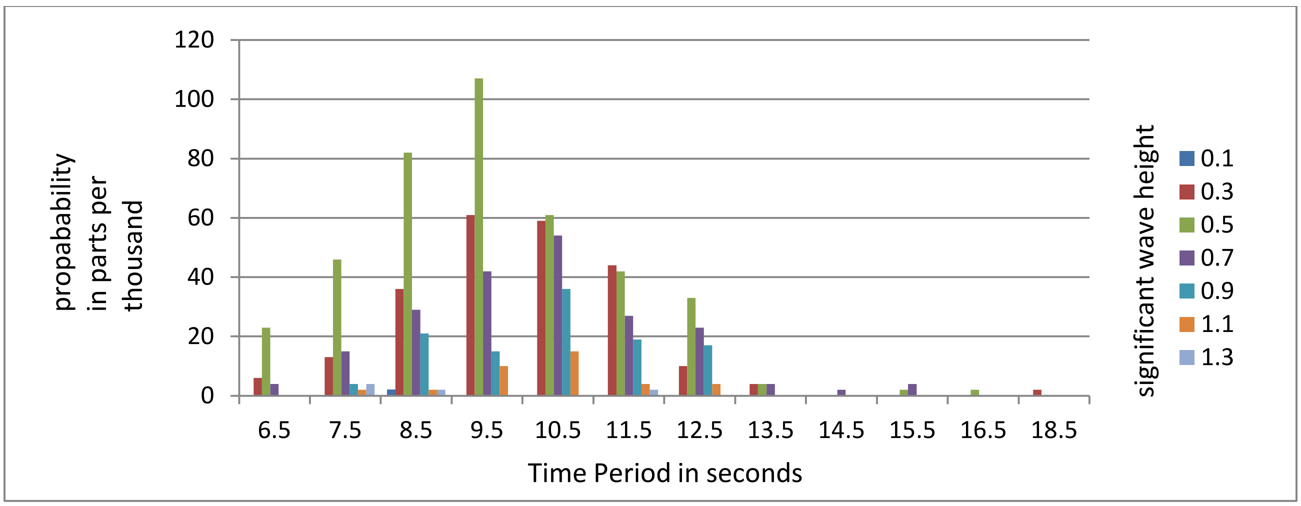

Figure 3.5.1 Scatter diagram for Fair weather Time period and significant wave height

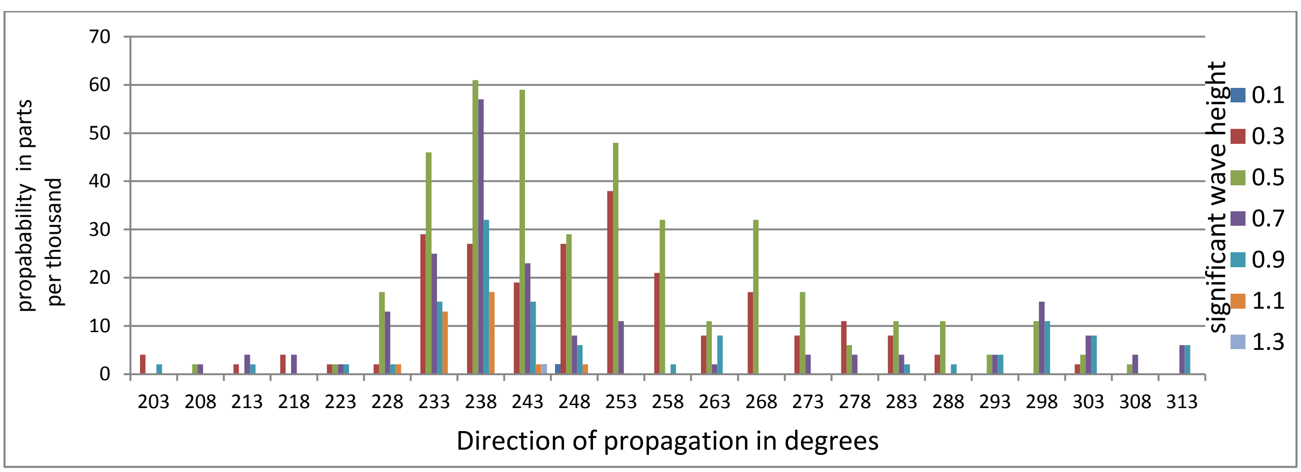

Figure 3.5.2 Scatter diagram for Fair weather Direction and significant wave height

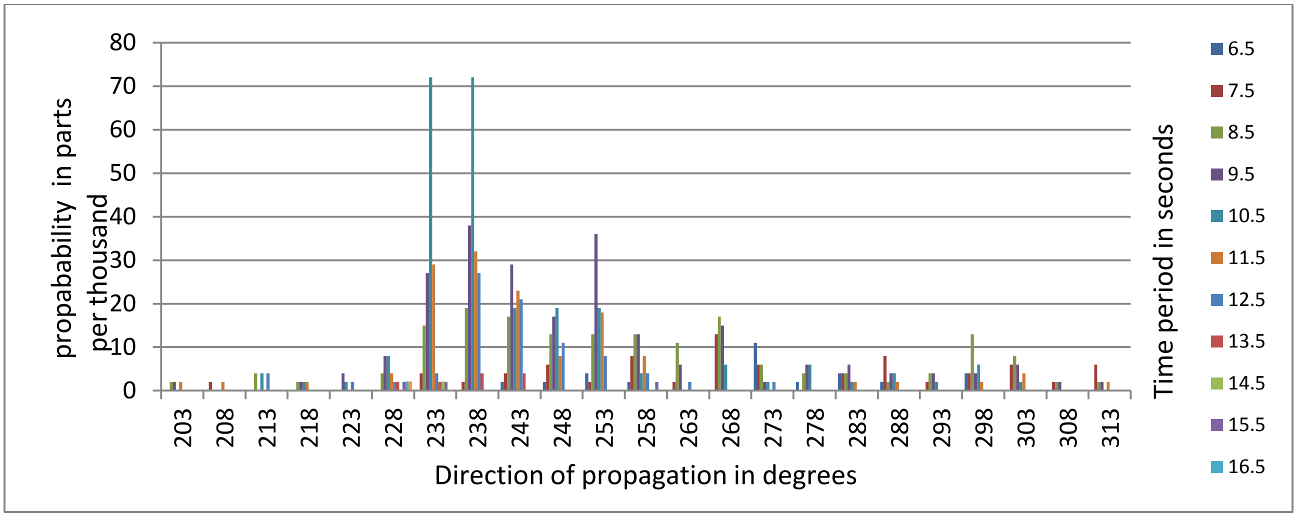

Figure 3.5.3 Scatter diagram for Fair weather Direction and significant wave height

3.2.2 Rough Weather waves

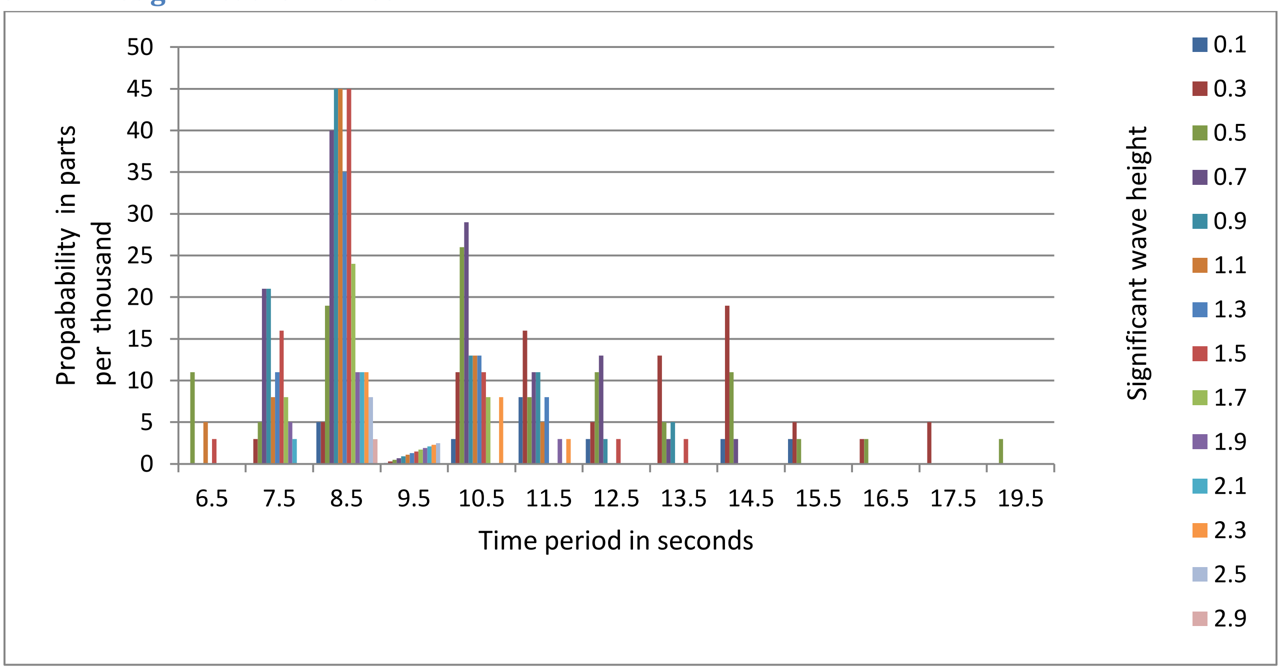

Figure 3.6.1 Scatter diagram for Rough weather Time period and significant wave height

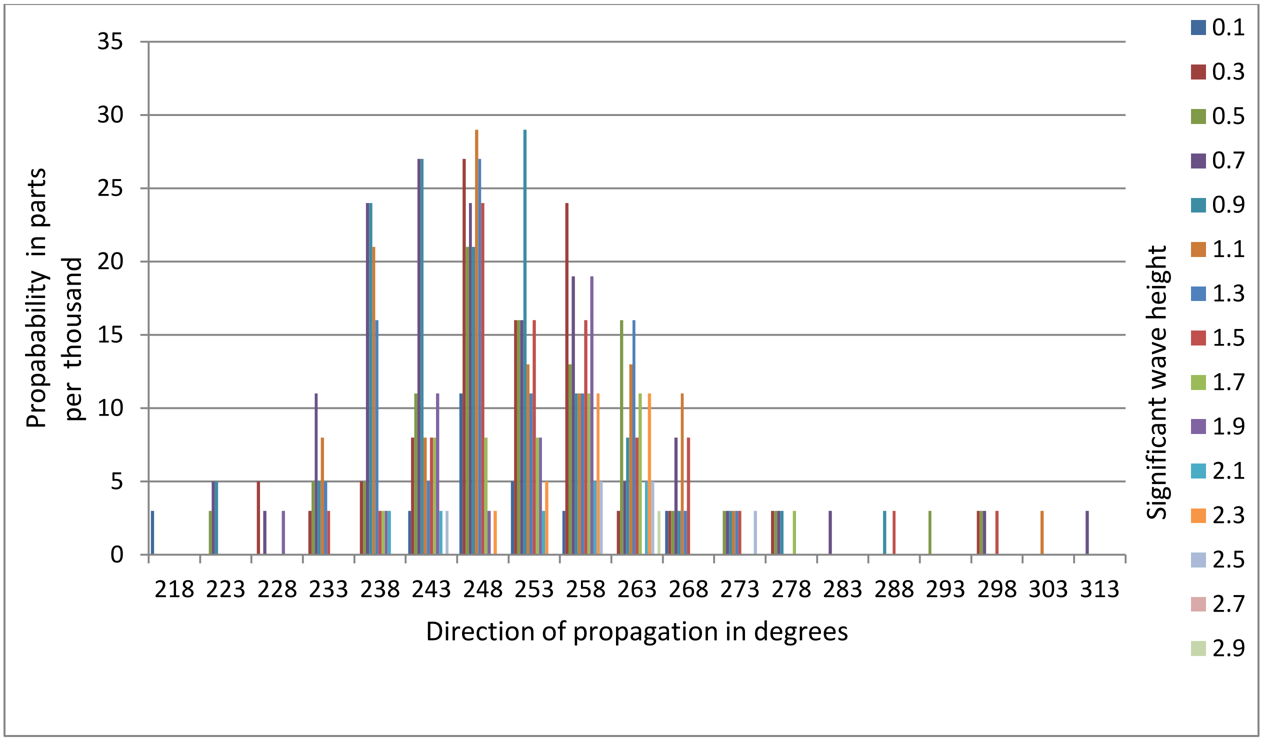

Figure 3.6.2 Scatter diagram for Rough weather Direction and significant wave height

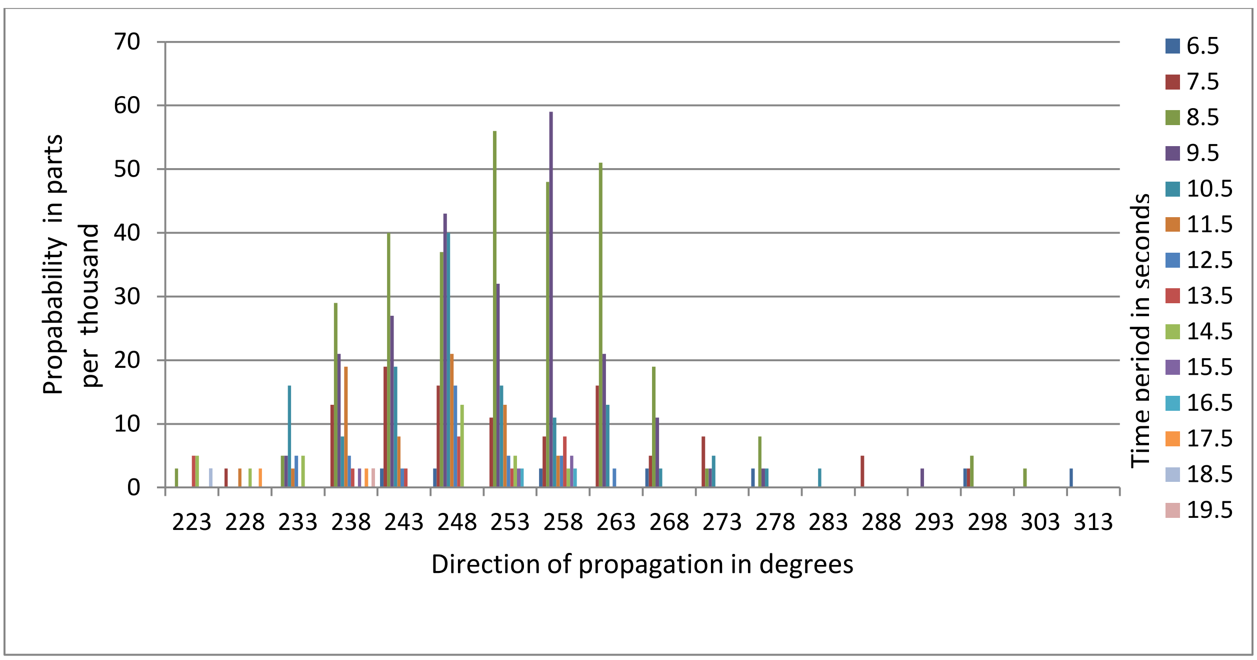

Figure 3.6.3 Scatter diagram for Rough weather Direction and significant wave height

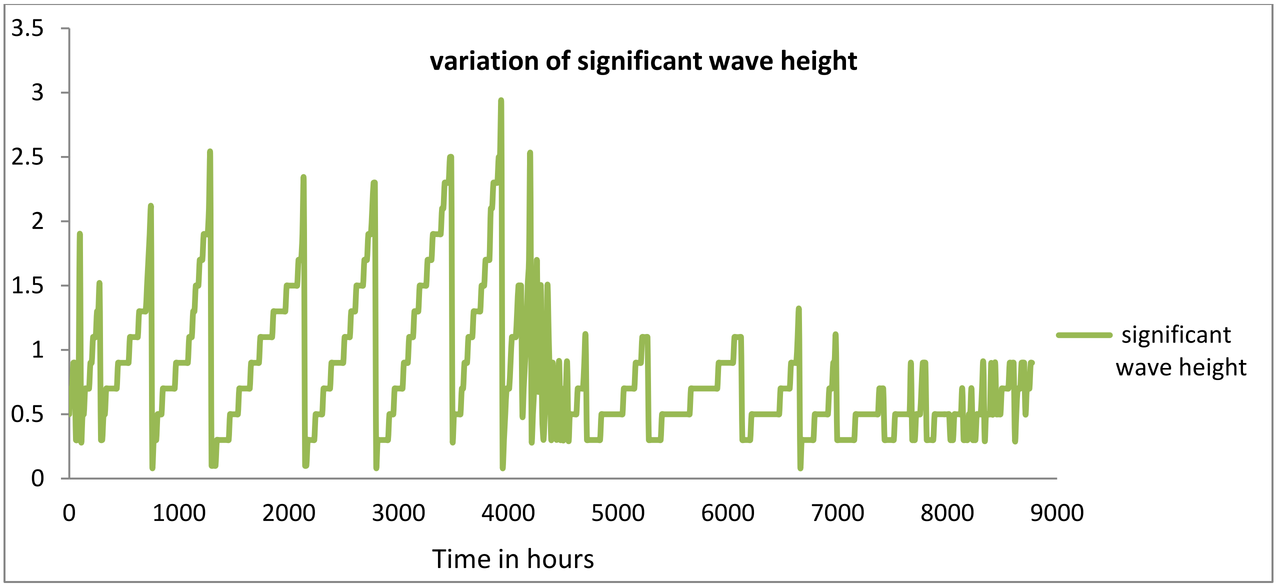

Figure 3.7 variation significant wave heights with time the waves applied at the boundary

3.3 Model setup and parameters

The CMS Flow and CMS Wave are run interactively wherein the data from one model is fed to another model for the calculation of morphological changes due to littoral drift. The wave model is run for every 12 hours for duration of one year. Radiation stresses from the wave module are input to the flow model every 12 hours to calculate the littoral transport. The values used for various model parameters are given below.

Manning's N -0.025

D_{50} size of the sediment -0.2mm (from the review by T.S. Shahul Hameed)

sediment density –2650 Kg/m^3

Water density- 1025 kg/m^3

Sediment Porosity-0.4

Sediment transport formulation- Non equillibruim formulation

3.4 Results and discussion

Annual changes in the depth for the present without any development, with development of attached break water and detached breakwater are shown in figures 3.8, 3.9 and 3.10 respectively. It was seen that the littoral activity is more vigorous during the Rough season when compared to that of fair season. This means that most of the annual changes in the depth are happening during the rough season only. The fair season component of the annual bed change is very low.

3.4.1 Sedimentation in the approach channel and turning basin

3.4.1.1 Option-D with detached break water

The annual sedimentation in the approach channel and the turning basin is around 12612 m^3 of which the sedimentation in the turning basin is 5970 m^3 and the maximum height of deposition is around 0.13 meters.

3.4.1.2 Option-C with connected break water

The annual sedimentation in the approach channel and the turning basin is around 209459 m^3 of which the sedimentation in the turning basin is 44144 m^3. Sedimentation in the channel is 165315 m^3, out of which 75 percent of sedimentation happens within a distance of 500 meters from the basin entrance and the maximum height of deposition is around 3.2 meters.

3.4.2 Effect of breakwater on surrounding area

It can be seen from figure 3.8 that in the existing condition there is a tendency of erosion in the surrounding areas of the proposed breakwater except for a patch of deposition happening at a distance of 2.5 kilometers to the North of the proposed break water because at this location there is formation of an eddy by the currents generated due to wave circulation near the coast which is causing the sediments to get deposited there.

The construction of a detached breakwater increases the tendency of deposition in the vicinity of the area. This is due to the fact that in the absence of the break water the long shore currents due to inclined waves tend to move along the shore thereby carrying away the sediments with them whereas after the construction of the jetty there is formation of an eddy in the region between the break water and the beach which is causing the deposition of sediments to occur.

In the case of attached break water the eddy is formed at the entrance to the basin which is causing the sediments to deposit in the channel. It can be seen that the erosion to the south of the attached break water is increasing whereas the general tendency of increase in the deposition or decrease in erosion is seen to the North of the attached break water when compared to the existing situation without any development.

In view of the above observations it can be said that construction of detached break water is helping in arresting the erosion of beach in the vicinity of the detached break water.

It should be that these studies were conducted with the data available. If more specific data can be available on various site parameters like wave heights, sediment concentrations etc, more accurate prediction can be made with regards to morphological changes and wave climate.

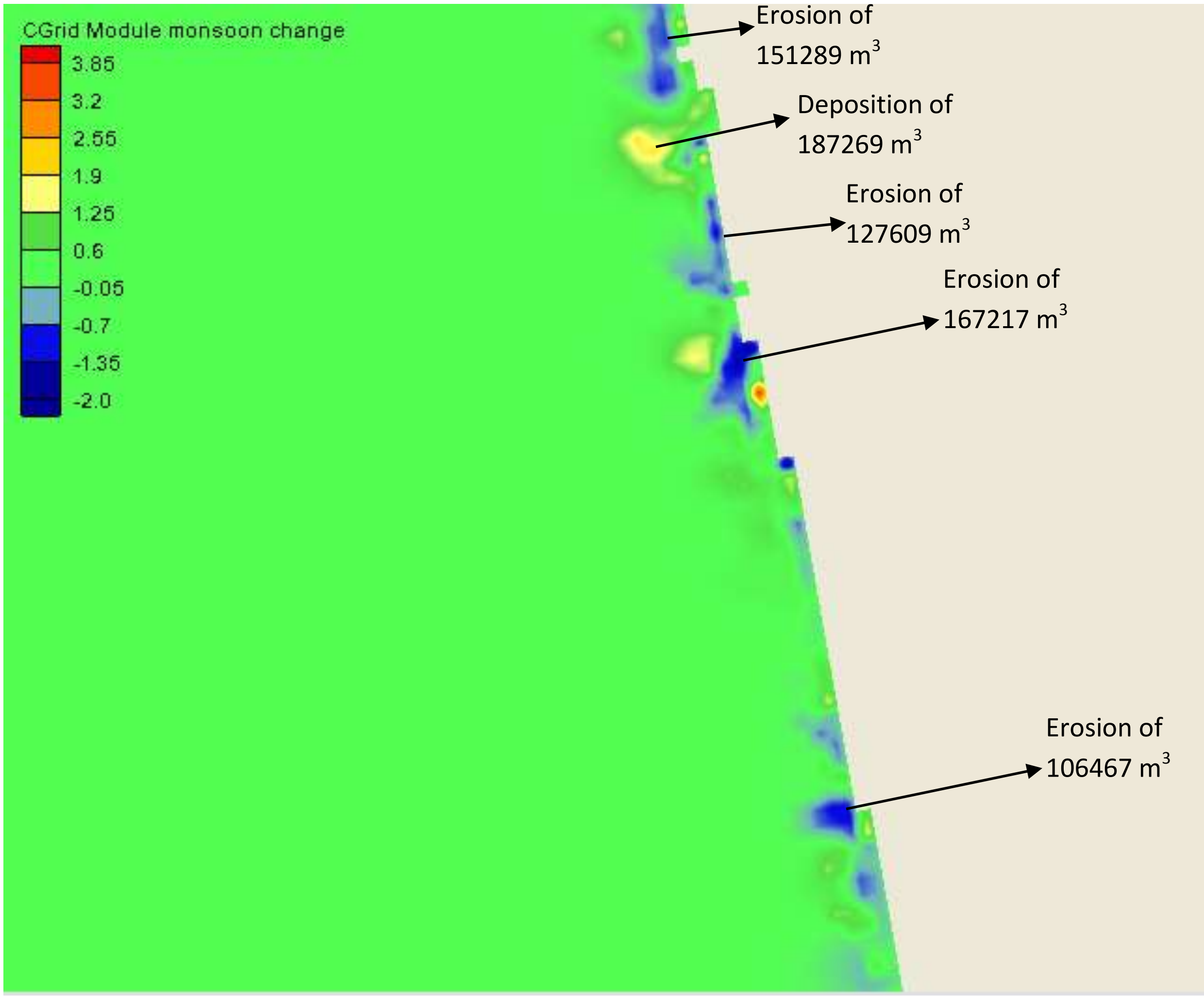

Figure 3.8. Annual changes in bathymetry without any development

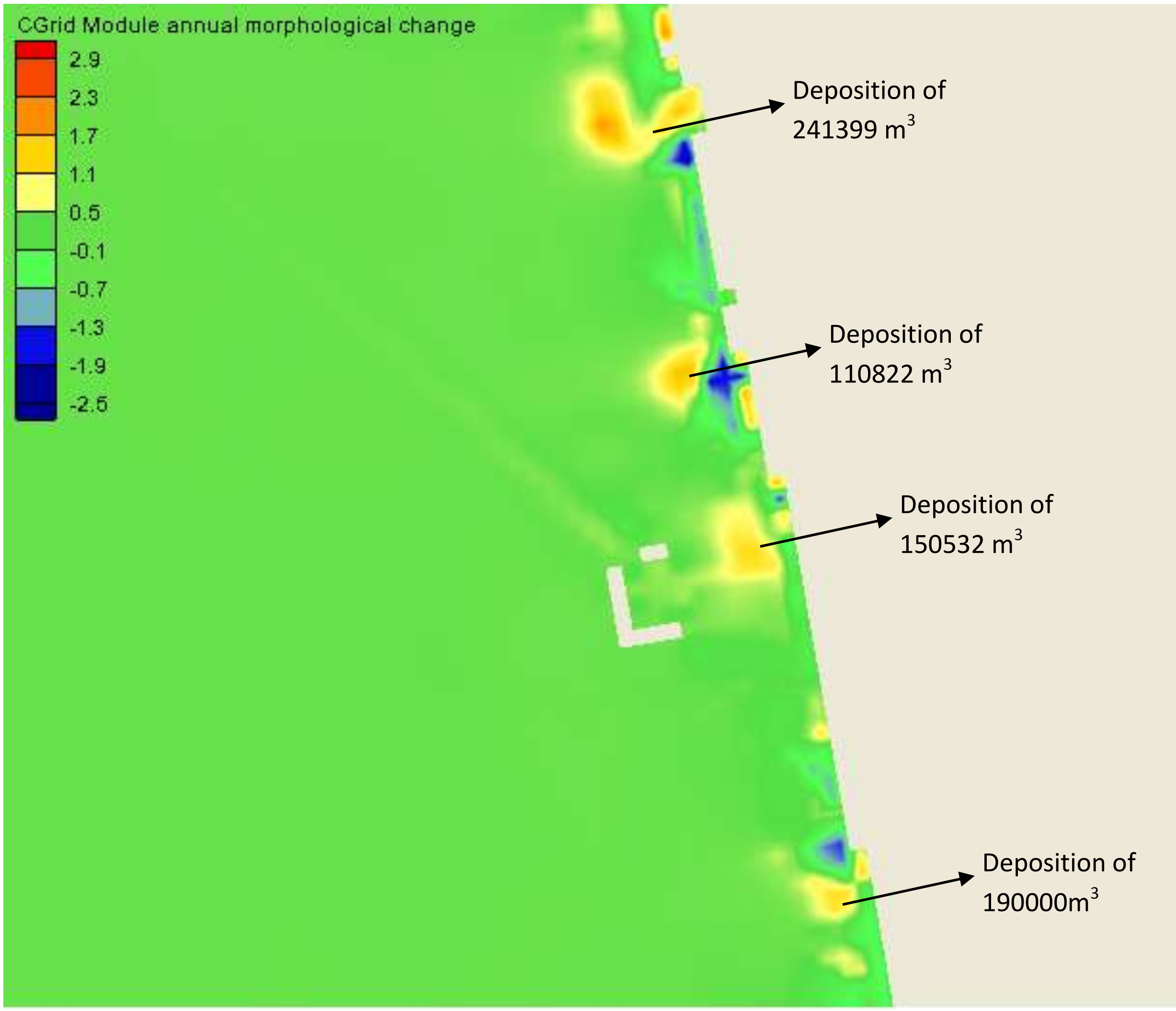

Figure 3.9. Annual changes in bathymetry with Detached break water in Place

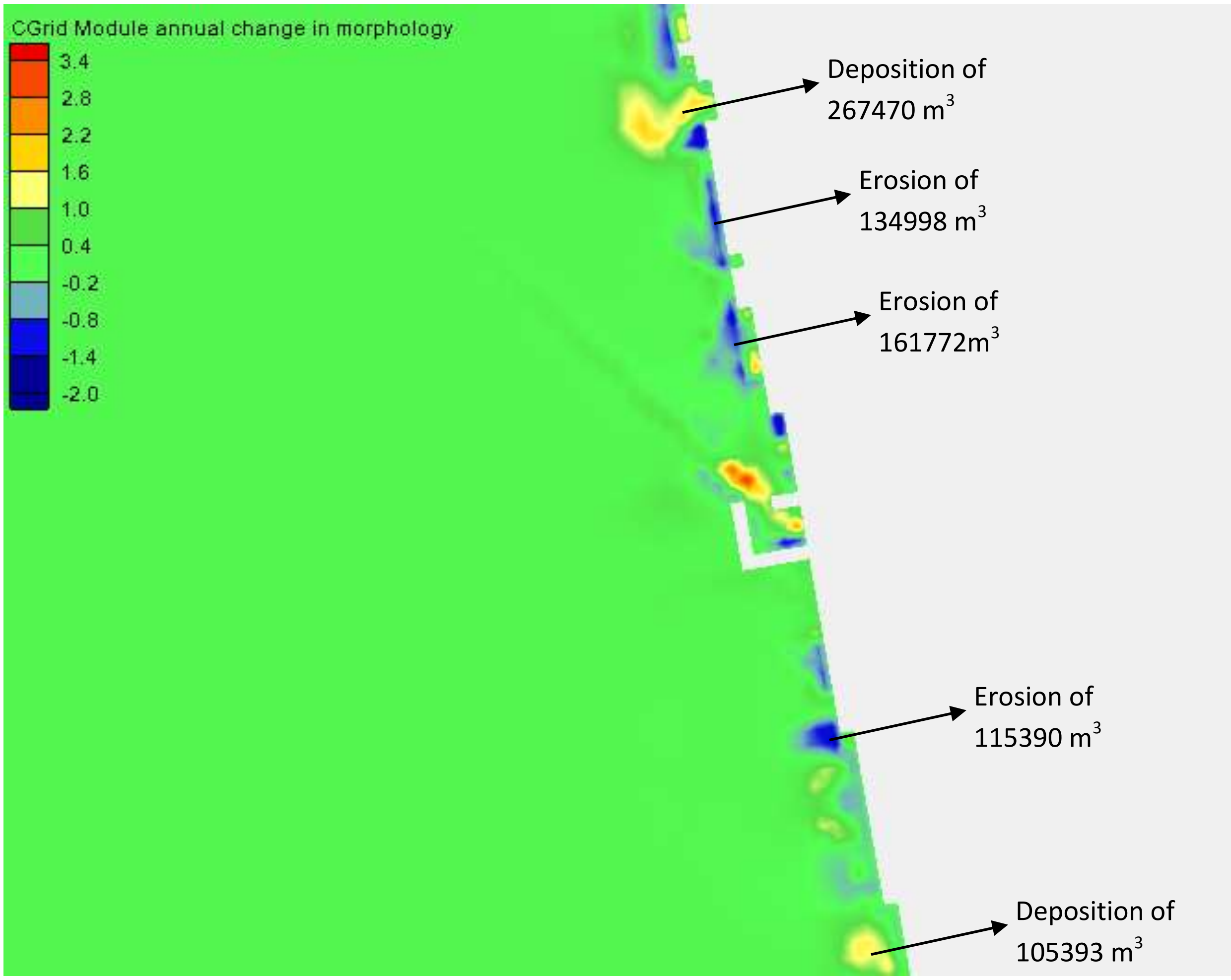

Figure 3.10. Annual changes in bathymetry with attached break water in Place

4 Wave Disturbance study:

4.1 Bathymetry

Wave disturbance study was conducted to check the efficacy of the detached breakwater to reduce the wave disturbance in the basin. The study was conducted using Bouss2D. The bathymetry used for this study is shown in figure 3.1. It should be noted here that negative depth is depth below CD and positive depth is depth above CD. The grid size is taken as 4 meters x 4 meters. The extent of the domain is taken as 2300 meters x 2150 meters. The bathymetry chart containing the survey conducted in April-May 1999 is digitized and used for generation of the bathymetry for the model.

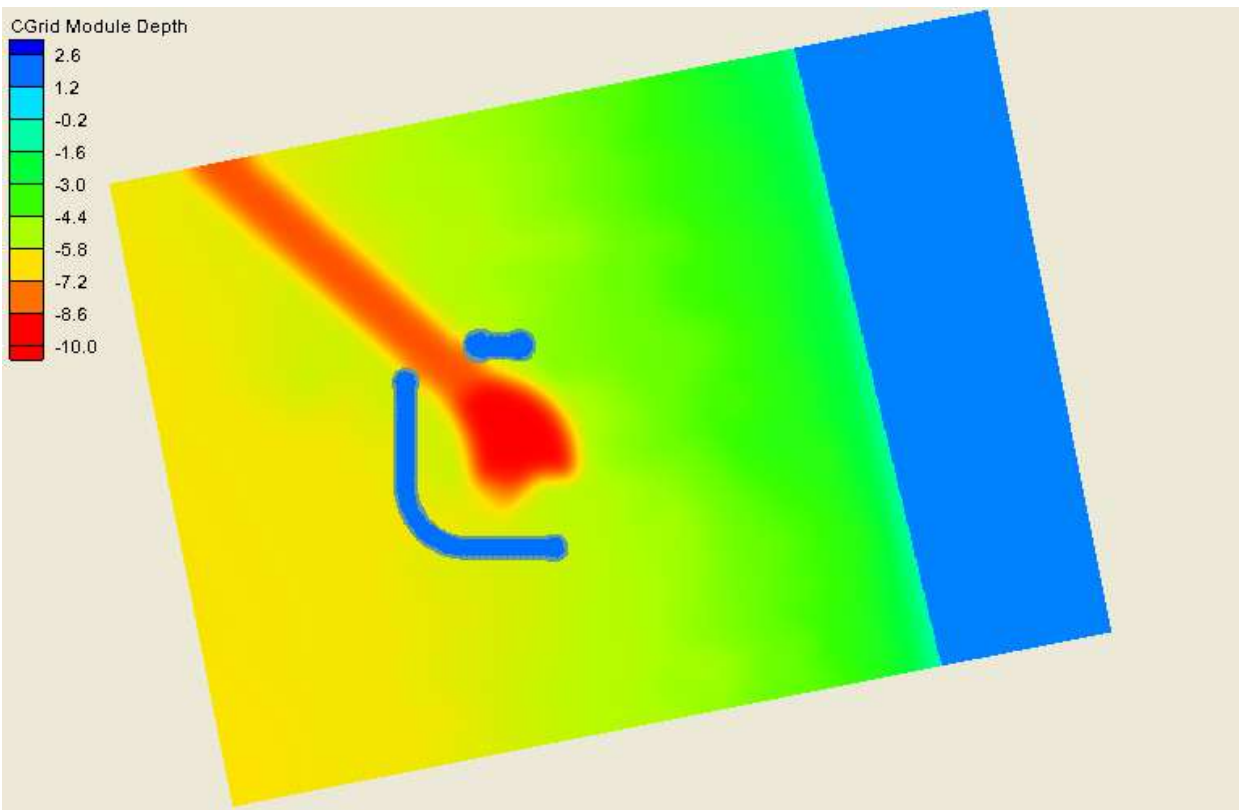

Fig 4.1 Bathymetry used for the wave disturbance model

4.2 Boundary conditions

Boundary conditions are applied in this model using cell strings. Figure 4.2 shows the cell strings used for applying various conditions for the model. The Green colour line shown in the western edge of the model domain is the wave generator boundary condition. Incident wave conditions are applied all along this boundary. The direction of the incident wave is perpendicular to this boundary. Incident wave conditions were taken as a significant wave height of 3 meters with a time period varying from 3.3

seconds to 10 seconds. Damping layers were applied along the lateral boundaries, coast line and the perimeter of the break water.

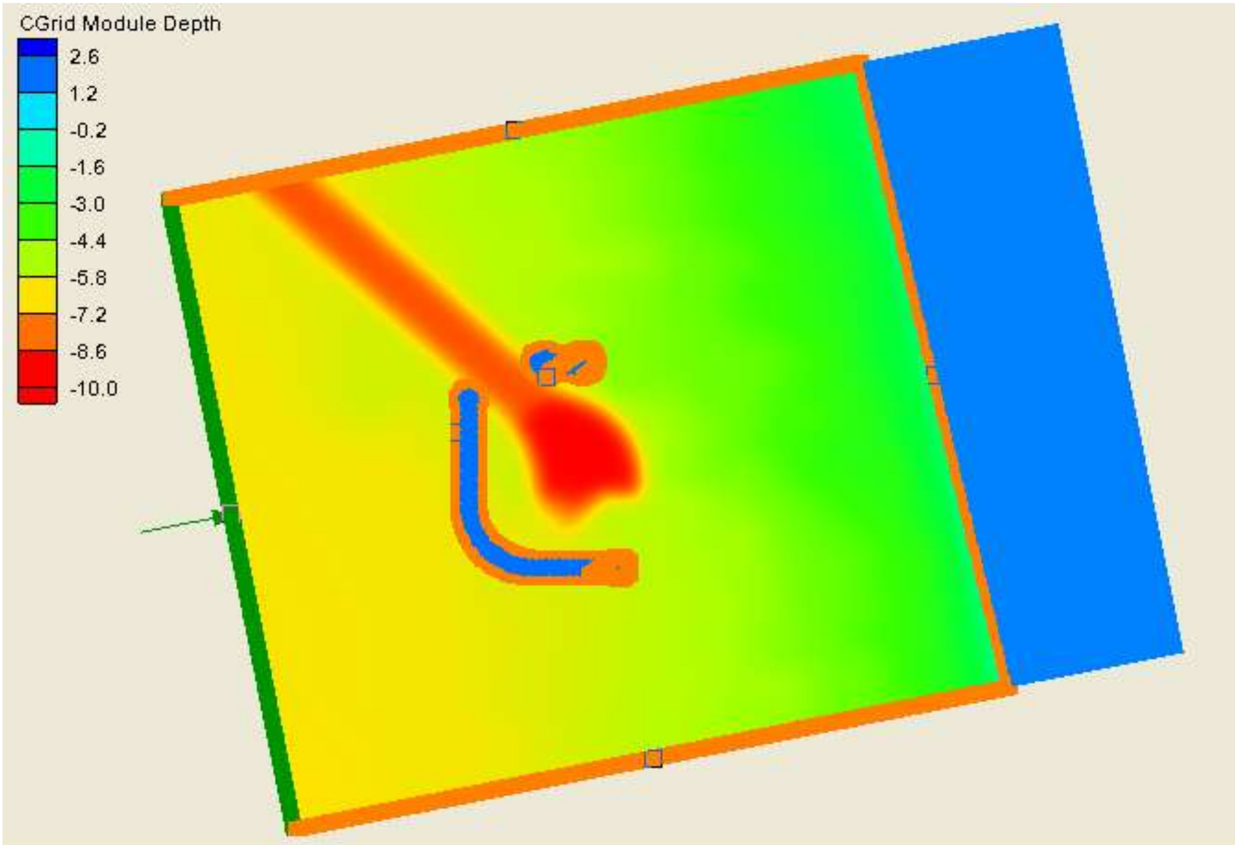

Fig 4.2. Boundaries and damping layers

The damping layers are denoted in orange colour. Damping layer are applied at the lateral boundaries to subdue the reflected wave coming back into the domain which helps in simulating the effect of waves leaving the boundary. The damping layer on the coast line is applied to simulate the effect of gradual dissipation of wave along the gradually sloping beach. In the similar fashion a damping layer is applied along the break water to simulate the dissipation of the wave along the breakwater boundary. The model was run for duration of 44 minutes with a time step of 0.2 seconds and the surface elevation was taken as 0.5 meters for the run.

4.3 Results and Discussion

The variation of significant wave height over the domain is given in figure 4.3. It can be seen from the figure that the figure that the input wave which is 3 meters in height reduced to 0.6 meters at the entrance to the basin and is getting reduced to a height of 0.2 meters in front of the berth.

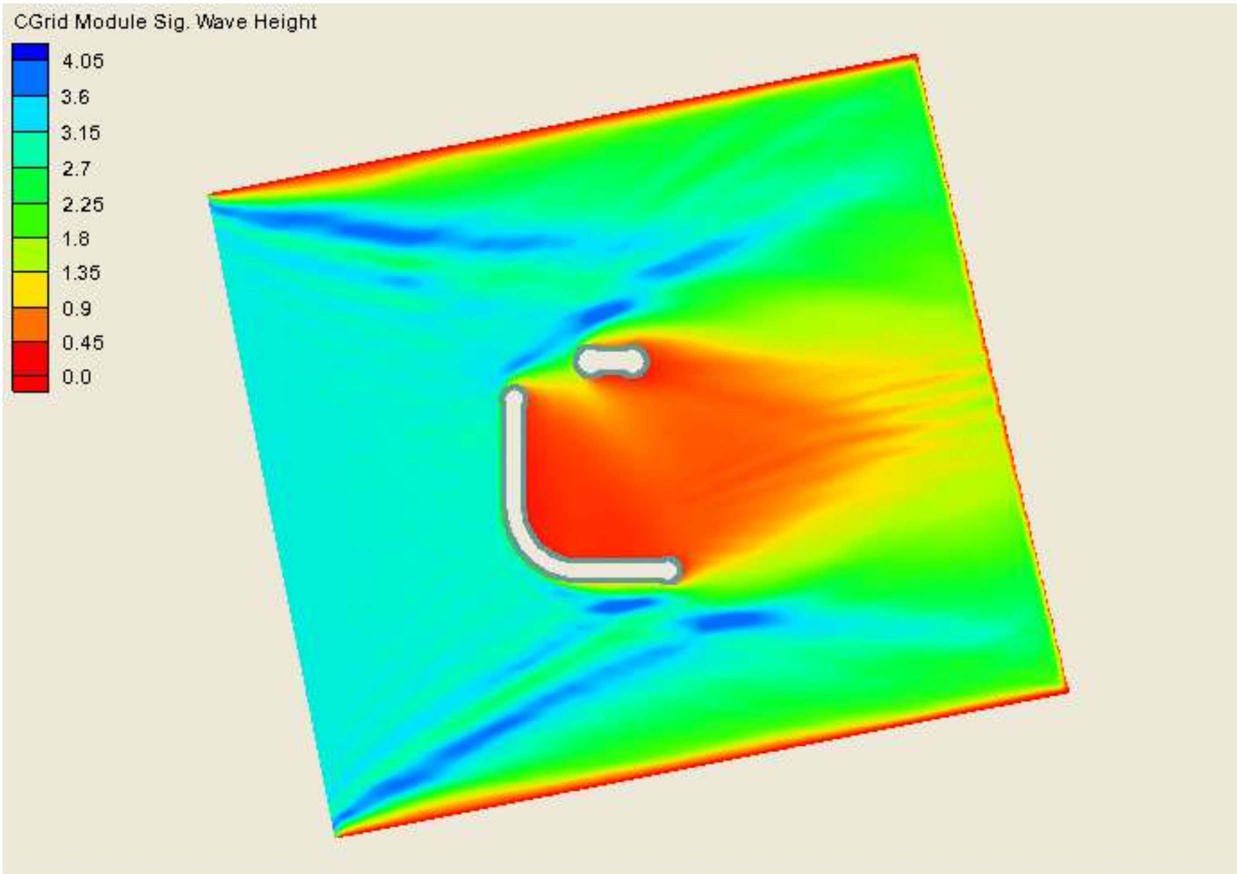

Fig 4.3. Boundaries and damping layers

The approximate acceptable wave heights for different types of vessels recommended by the International Association of Ports and Harbours are given in the following Table.

Table-4.1 allowable wave heights at Berths (IPA Norms)

Type of Vessels	Acceptable wave heights-HS (m)	
	Incident angle –00 Head on	Incident angle(450-900)
Conventional general cargo vessels	1.0	0.8
Container vessels	0.5	0.4
Dry bulk carrier 30-100,000 DWT(loading)	1.5	1.0

Dry bulk carrier 30-100,000 DWT(unloading)	1.0	0.8-1.0
Tankers 30-200,000 DWT	1.5-2.5	1.0-1.2

As per IS 4651 (Part V), the wave disturbance within the harbor should not exceed the tranquility conditions given in following Table.

Table-4.2 allowable wave heights at Berths (IS-4561)

Type of Vessels	Approximate acceptable wave heights - Hs(m)	
	At berth	Turning circle
General Cargo	0.65	0.9
Bulk Cargo	0.9	1.2
Container Cargo	0.65	1.2
Dredgers	-	0.45-2.0

This incident wave height which is taken as 3 meters is not exceeded 97.7 percent of the time. Keeping in view the above discussion it can be safely concluded that the break water is sufficient enough to allow operations throughout the Year. As seen in fig 4.3, it can be observed that the wave at the entrance of the breakwater is a bit high. This is due to the reflection of the wave from the breakwater. So the breakwater edge which is nearest to the channel entrance should be constructed in such a way that the wave reflection is minimal. Such a design of break water can be taken up later at the detailed design stage.