PHYSICAL MODEL STUDIES ON WAVE PROPAGATION ALONG APPROACH CHANNEL AND ITS EFFECTS ON HARBOUR TRANQUILITY

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

By

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OCTOBER 2018

D E C L A R A T I O N

By the Ph.D Research Scholar

I hereby *declare* that the Research Thesis entitled "**Physical Model Studies on Wave Propagation along Approach Channel and its Effects on Harbor Tranquility"** which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfillment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering** is a *bonafide report of the research work carried out by me.* The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

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C E R T I F I C A T E

This is to *certify* that the Research Thesis entitled "**Physical Model Studies on Wave Propagation along Approach Channel and its Effects on Harbor Tranquility"** submitted by **Jagadeesh H.B.**, Register Number: 121154 AM12P01, as the record of the research work carried out by him is *accepted as the Research Thesis submission* in partial fulfillment of the requirements for the award of degree of **Doctor of Philosophy**.

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ACKNOWLEDGEMENTS

With deep sense of gratitude, I express my heartfelt thanks to Dr. Subba Rao, Professor, Department of Applied Mechanics and Hydraulics, NITK, Surathkal, for supervising my research work, timely guidance, encouragement and motivation during the period of research work. The moral support and critical guidance have been priceless which has given me an invaluable opportunity to publish my research work in international/national journals/conferences which is a matter of great satisfaction.

I thank Prof. Karanam Uma Maheshwar Rao, Director of NITK, Surathkal, and formar Directors Prof. Swapan Bhattacharya and Prof. K.N.Lokesh of NITK, Surathkal for granting me permission to carry out research work in the prestigious Institute. I thank Dr. (Mrs) V.V.Bhosekar, Director CWPRS, Pune and the former Directors of CWPRS, Pune for granting me permission to carry out my research work at NITK, Surathkal and permitting me to use the model facilities and data for the research work. The time to time help and support extended by Shri T. Nagendra, Scientist-E, CWPRS is acknowledged.

I am grateful to Research Progress Committee members, Prof. G. S. Dwarakish and Prof. Vijay Desai, for their critical evaluation and useful suggestions during the progress of the research work.

I am greatly indebted to Prof. Amai Mahesha,, Head of the Department of Applied Mechanics and Hydraulics, NITK, Surathkal, and, Prof. M. K. Nagaraj, the former Head of the Department and Prof.A.V.Hegde for extending their cooperation and valuable suggestions during the course of research work.

I sincerely acknowledge the help and support rendered by all the Professors, Associate Professors and Assistant Professors of Department of Applied Mechanics & Hydraulics.

I gratefully acknowledge the support and all help rendered by all staff members of Department of Applied Mechanics & Hydraulics., the Post-Graduate and research scholars at NITK during the research work.

I take this opportunity to thank Late Shri U.V. Purandare, Retd. Additional director, and Shri V.B.Joshi, Retd. Chief Research Officer, CWPRS for their valuable guidance during the initial period of my work at CWPRS on hydraulic physical models. Thanks are due to Shri A.S.Chalawadi, Assisstant Research Officer, and the supporting staff, members in Ports and Harbours-II division at CWPRS for rendering help during the conduction of studies on the physical model.

My special thanks to Dr.Prashanth J., Dr.Bhojaraja B.E., Ms. Geetha Kuntoji, Mr Amit V Wazerkar and Nawin Mohan for their timely help extended at NITK for my research work.

I express heart felt gratitude to authors of all those research publications which have been referred in this thesis.

Finally, I wish to express gratitude, love and affection to my beloved family members, my wife Mamatha, daughter Namratha and son Vinay without their cooperation this research work would not be possible. The encouragements provided by my father Sri. H. G.Basavaraj, mother Smt Jayamangala, Father-in-law Sri Mruthyunjaya and mother-in-law Smt Late Madalambike, is greatly acknowledged here. Thanks are also due to all my brothers, sisters, their family and all my friends for their encouragement, moral support during the course of my research work.

Jagadeesh H.B.

ABSTRACT

Development of harbour basin along open coasts need to be connected to the sea with sufficient depth by dredging approach channels of required base width, side slopes and length . The side slopes of such channel also need to be maintained for no sliding condition under water. The side slopes of the channel mainly depend on the seabed material and its stable slope in the submerged conditions. The length of the channel depends on the sea bed bathymetry of the site proposed for development. While finalizing the channel alignment and its geometry, an in-depth understanding of the factors responsible for stability and the effects on the wave propagating along the approach channel and its effects on harbor wave tranquility are very important. 3-D shallow basin hydraulic physical models studies provides a very useful support for the decision to be taken in this regard under complex hydraulic conditions. The wave attenuation along port approach channel is responsible for the dispersion of the wave energy along the side slopes of the channel to outer regions. Due to this phenomenon total amount of wave energy entering into harbour basin will be reduced and this results in achieving the overall wave tranquility inside the harbor basin.

This natural phenomenon of wave attenuation can be effectively utilized for optimization of the harbour layout resulting in reducing the lengths of breakwaters, reduction in the lengths of approach channel by orienting it perpendicular to the sea bed contours wherever possible. This effectively reduces the overall cost of investment of the project and in some case reduces the recurring cost of maintenance dredging as well.

Hydraulic physical model studies are conducted to investigate the wave propagation along port approach channel by varying the side slopes of the channel, which effects of wave attenuation. The effects on wave propagation for regular and random waves are studied. The comparison of studies with regular and random wave generations are made. It is observed that the wave attenuation effectively results in 87%, 90%, 92% and 93% for channel side slopes of vertical, 1:5, 1:10, 1:20 slopes respectively. The wave energy dispersion increases with the increase in the channel dimensions.

The available model studies data at CWPRS for the stage wise development of port over a period of about fifty years was collected .This data has revealed the effects of wave

attenuation for different lengths and depths of channel. It is observed that with the increase in channel dimensions there is increase in wave attenuation effects. Based on these studies the suggestions made for the port development is successfully working in field. This can be effectively used by port planners for arriving at good conceptual layouts.

Apart from wave heights the direction of the wave is another very important factor at the berthing locations inside the harbour basins for effective loading and unloading operations. Broad side wave incidence on the berthed vessels reduces the efficiency of operation and in some extreme cases damages the vessels also. The cost of the mooring devices mainly depends on the ship motions at berth, this in turn depends on wave incidence angle on the vessels. Thus wave directional spread studies for finalizing the berth alignment is an essential part of studies for port development.

Studies are conducted to observe the directional spread of waves generated by 2-D wave generator in the model along the approach channel and adjacent shallow regions. The directional spread for waves approaching from different directions making different incident angle with the sea bed contour are observed by sketching the wave crests in the model. The use of 2-D long crested wave generators in 3-D shallow basin model for wave tranquility studies is widely adopted till date. This is mainly due to its amicability and ease with which it can be used and its low cost effective as compared to 3-D paddle type wave generators. Model studies conducted by various experts have revealed that use of such wave generators are fully justified from the point of port planning Hughes (1993). The 2-D wave generated will propagate over the complex bathymetric conditions in the model and gradually develops into 3-D wave state and resembles the prototype. The factors effecting the change of wave from 2-D to 3-D status viz., wave approach angle with the sea bed contours, presence of port structures like breakwater, approach channel, berthing structures etc., Based on these studies some useful conclusions were drawn for a model engineer in deciding the size of the model for port development studies.

The wave tranquility studies conducted on the model for development of berths at different locations in the harbour basin, the effects of wave height for waves approaching from different directions, effects of wave incident angle at berthing location, the usefulness of

wave incidence angle on decision making about the berthing alignment are discussed based on model results. The effects of providing sloped surface on the rear side of a berthing face, maintaining spending beaches within the harbour basin effectively facilitating wave run up and wave energy absorption.

From the findings of the model studies it is suggested to utilize the natural wave attenuation phenomenon in aligning the port approach channel. This effectively increases the wave tranquility in the harbour basin. Based on wave tranquility studies for different berths in the harbour the advantages of providing sloping face behind berthing structure and effect of maintaining spending beaches within harbour basin in maintaining good wave tranquility are highlighted. The results of directional propagation studies conducted are helpful for a model engineer to select the boundary of physical model to be simulated depending on the sea bed contour and wave incidence angle. The studies conducted will be very useful for port engineers in general planning of the port layouts.

Keywords: *Approach Channel, Breakwater, Directional Spread, Regular Wave, Refraction, Reflection, Random Wave, Sea Bed Contours, Wave Spectra, Wave Tranquility, Wave Attenuation.*

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NOMENCLATURE

Symbols	Explanation
H	Wave Height
T	Wave Period
Hmax	Maximum Wave Height
Tmax	Maximum Wave Period
C	Wave celerity
C_o	Deep water wave celerity
Cs	Group velocity
F	Frequency
g	Acceleration due to gravity
d	Water depth
L	Wave length
T_p	Peak period
T_z	Mean Zero crossing period
Ho	Deep water depth
α	Wave direction
Δf	Basic frequency increment in discrete Fourier analysis
$\eta(t)$	Surface elevation referred to the mean water level
η c	Crest elevation referred to mean water level
η T	Trough elevation referred to mean water level

σ	Standard deviation
Θ	Direction of wave propagation as used in directional spectra
ω	Angular frequency
fp	Spectral peak frequency, l/Tp
Hs	Significant wave height
Ts	Significant wave period
Tp	Spectral peak period, l/*fp*
Tm	Average wave period
a_c	Zero-crossing wave crest height
a_T	Zero-crossing wave trough excursion
$a_{c,max}$	Maximum zero-crossing wave crest height
$a_{T,max}$	Maximum zero-crossing wave trough excursion
H_d	Zero-downcrossing wave height
$\hat{H}_d$	Average zero-downcrossing wave height
Hu	Zero-upcrossing wave height
$\hat{H}u$	Average zero-upcrossing wave height
H_σ	Estimate of significant wave height
$R\eta(\tau)$	Auto correlation function of $\eta(t)$
$\eta_{\min}$	Minimum surface elevation in a wave record referred to the mean water level
m_n	n^{th} moment of spectral density
H_{mo}	Estimate of significant wave height

CHAPTER 1
INTRODUCTION

1.1 IMPORTANCE OF PHYSICAL MODELS IN COASTAL ENGINEERING

Coastal engineers mainly rely on three complementary techniques to find solutions to complex fluid flow regimes in coastal projects. These techniques are field observations and measurements, laboratory observations and measurements, and numerical modelling. The laboratory studies are generally termed as Physical models often they are miniature representation of the physical system. Apart from physical models are numerical model which is a mathematical representation of physical system; here the governing equations are discretized and solved by computational techniques. Many experts have pointed out that field studies provide the best data, but they are expensive and too many difficulties during the data collection and many times during extreme weather conditions it's not possible to take measurements which are rather very important. In contrast physical models are smaller, less expensive, easily controllable for studies and simpler but they include most important aspects of the problem to be studied. Here input conditions can be controlled systematically for studies whereas this is not possible in nature during field studies and due to extreme conditions it may not be possible to collect data in field for critical events. Numerical models are feasible for simple processes like wave refraction, shoaling and diffraction and considerable success have been achieved in this field with advanced computers and computing techniques. However many flow conditions in coastal engineering are not amenable to mathematical analysis because of the nonlinear character of governing equations of motions, lack of information on wave breaking, turbulence, bottom friction, internal reflections, complexities of wave run up and rundown at berm breakwaters, simultaneous occurrence of different phenomenon etc. These makes it difficult to accurately simulate the near shore water circulation in mathematical models. In these cases it is necessary to resort to physical models for prototype behavior, Hughes (1993). Also growing use of numerical models in coastal engineering has not made physical model obsolete. They keep pace with mathematical model and most of the mathematical models needs experimental verifications as well. The concept of hybrid modeling where the results from physical model of complex region are used as input or boundary condition for numerical models and vice versa. Thus it is expressed by the experts that theory cannot cover all the complications that are encountered in practice. Thus most major coastal engineering projects are tested on physical models for optimization.

In coastal engineering, physical model studies with shallow wave basin model having long crest wave generation facility have played a very vital role to support the harbour layout design and optimization. Basically these are 3-D models constructed to geometrically similar scale generally having rigid bed simulated to study the wave propagation from off-shore region to near shore region. These types of models provides less expensive tool for examination of 3-D wave process in sea condition as 2-D process in the model at wave generation. These wave basins provide the capability to study the wave transformation process over complex bathymetric situations. These wave basins are the traditional tools to study evolution of the wave field over a non-uniform bathymetry at specific project sites as waves undergo refraction, diffraction, shoaling, breaking, reflection, internal diffraction etc and more important is many of them occur simultaneously. The most important application of the short wave models is to support harbour layout design. The following design aspects are examined in a short wave shallow basin harbour model, Hughes (1993), Jenson (1993).

- Optimization of the breakwater configuration for most economic design with adequate protection against short wave effects.
- Optimal design and alignment of the approach channel with comfortable entry and exit conditions.
- Quantification of wave disturbances in and around the harbour region.
- Wave induced currents, wave reflection and diffraction effects near the harbour entrance.
- Qualitative information on littoral processes through tracer studies.

The concept of Hybrid modeling where the results of a complex phenomenon from a physical model are used as the input conditions for the mathematical model alternatively numerical model result may be used to provide input boundary conditions for a physical model makes these two modeling techniques complimentary (Beranger,1986).

Thus physical models constructed and operated at reduced scale offer an alternative for investigating a coastal phenomenon that is beyond our analytical skills. Dalrymple (1985) pointed out two distinct advantages of physical models to replicate near shore processes.

1. The physical model integrates the appropriate equations governing the processes without simplifying assumptions that have to be made for numerical model.
2. The small size of the model permits easier data collection throughout the regime at a reduced cost, whereas field data collection is much more expensive and difficult, and simultaneous field measurements are hard to achieve.

The other advantages are the following:

- Degree of experimental control that allows simulation of rare environmental conditions at the discretion of the researcher.
- Visual observations from physical models give a very good qualitative impression of the physical processes which is highly beneficial in conceptual thinking for a coastal engineering project proposal.

1.2 TYPES OF PHYSICAL MODELS IN COASTAL ENGINEERING

1.2.1 Rigid bed models

They have solid boundaries that cannot be modified by the wave hydrodynamics processes in the model. They are mainly used to study waves, currents or similar hydrodynamic phenomena in laboratory. They are also used to study the interaction of hydrodynamic forces with solid surfaces. The scaling effects of rigid-bed models are generally well understood and much confidence can be achieved by continuous usage of these types of models. These models are further classified into different types each of this type of model is developed with an aim to study certain factors to arrive at an optimal design within the required factor of safety, Hughes (1993).

2-D models: These include wave flume tests of breakwater stability, hydrodynamic forces on structures etc.

3-D models: These include shallow basin wave models to study wave penetration into harbour basin, transformation of directionally spread random waves, interaction of oblique waves and currents etc.

1.2.2 Movable-bed models

These have a bed comprising of a materials that can be influenced by the hydrodynamic forces. 2-D movable bed models are used to study the beach profile evolution, scour at the toe of coastal structure, estimating the efficacy of sand traps for the ports constructed in the littoral drift prone areas etc. 3-D movable bed models are costly and rarely used. This includes littoral drift studies, sand spit formation, scour holes in the vicinity of structures. Generally a compromise between rigid and movable bed is made by conducting injection studies using a suitable tracer material of calculated grain size depending on the material density and the model scale.

Scale models are cost effective considering the sizes of coastal engineering projects.

- Inherent limits of deterministic fluid mechanics due to turbulence makes the laboratory experiments most useful one.
- Availability of new techniques allows discovery of physical relationships of fluid flow beyond what is already known. Data processing techniques allows handling of large quantity of data, hence more complex relation between variables of the processes can be established.
- Accuracy of mathematical models is limited to accuracy of functional mathematical relationships on which they are based. Mathematical models point out the most important deficiencies and physical models offer chance to monitor and measure the physics in a controlled environment.

- Scale models allow reproduction of complex boundary conditions like berm breakwater, piled structures, wave absorptive surfaces like spending beach, sloping surfaces behind berthing faces etc. Thus the Convective and dissipative nonlinear effects which are the major difficulties in case of mathematical models are also in near similitude on physical models.
- Physical contact with the fluid element remains the best guide for the intuitive discovery and it stimulates the imaginations in creative engineering.

1.3 A PREAMBLE TO THE PRESENT RESEARCH WORK

It is important to understand the complexity of the wave propagation in the near shore region with complex conditions like varied bathymetry due to incorporation of approach channel, port basin, breakwater and other pertinent port related structures. Understanding these through experimental studies on the physical model and developing standard documentation is much useful to the scientific and engineering community using physical models for evolving the port design. Proper understanding of this complex physical process will lead to arrive at different alternative conceptual layouts during harbour planning which can be tested on the model for refining and finalization based on the merits of the proposal before implementation.

In the present research work an attempt is made to understand the wave propagation in a harbour basin along long approach channel, effects of channel dimensions viz its size-length and depth, side slopes, directional spread of the waves on the model in the harbour basin and adjacent regions for waves arriving from different directions, effect of different types of berthing structures, effects of spending beach and slopes underneath berthing structures with in a harbour basin on the wave tranquility. Wave tranquility in the harbour basin is a very vital aspect since the port operational efficiency depends on this.

Also some very useful physical model study results for different stages of the development of a port over last five decades are collected and some important conclusions are presented in the report. All these are beneficial for port planners involved in making master plan for stage wise port development of a port based on the increasing traffic demands at a port.

Experiments are conducted on 3-D rigid bed shallow basin physical model having the facility of regular/ random wave generation along with computerized multi-channel wave data acquisition facility. The experiments are conducted for wave attenuation studies during the process of wave propagation along approach channel with different side slopes. The results of wave propagation for regular and random waves and the advantages of utilization of random waves for wave tranquility studies are highlighted.

1.4 NEED AND SCOPE OF THE PRESENT STUDY

Ports developed along open coast need to be protected against waves approaching from different directions during different seasons. The waves travelling from off-shore region approaching towards the port basin propagates over the dredged navigation channel and has impact on other port structures. During this passage waves undergoes transformation before entering the port basin. By understanding the wave transformation process while travelling towards the port basin, it is possible to estimate its effects on port basin wave tranquility. Also by understanding the process of natural phenomenon of wave attenuation along port approach channel, it is possible to beneficially utilize this phenomenon in port design and optimize the breakwater layout effectively reducing the capital investment for the port development. These type of studies are desirable to be carried out on large scale physical models with sophisticated facilities like generation of random waves and multichannel wave data collection simultaneously at desired salient locations. Since the process of wave transformation along artificially dredged channel along with the interference of various port structure are very complex to be precisely studied by mathematical model and also physical models have advantages of visual observation and feel of the phenomenon as explained earlier. Hence it is desirable to study on physical models before finalizing the design and executing the plan which involves a huge investment. Many times it is noticed a simple idea made by physically observing on the running model will have a very huge positive effect in port development works.

The present physical model studies on wave propagation along approach channel to port basin using a sophisticated physical model is useful in understanding the changes that are occurring to the waves while travelling along dredged approach channel, understanding the phenomenon of wave attenuation along the channel and beneficially utilizing this in the optimization of port planning. This will effectively reduce the cost of investment and also the understanding of the waves will be useful for the port planners in general.

Overall understanding on the hydraulic processes on physical model with visual observation will aid engineers for alternative possibilities thereby improving the existing model setup. It will also help the person without advanced knowledge of hydraulic engineering also to understand the phenomenon. Thus it is beneficial to stakeholders like operators who use the facility of harbouring vessels

The present research is an experimental work, based on the experiments carried out a shallow basin random wave model the wave heights at different locations of harbour, the directional spread of the waves along the shallow sea bed regions, effects due to refraction, diffraction due to complicated nearshore structures. The effects of abruptly varying depths near harbour approach channel and the effects of sloping sides of the channel based on the direction of wave crest approach from deeper sea are studied and the effective utilization of this phenomenon is highlighted in the thesis. By doing so it is possible to optimize the cost of the port development structure and also to provide comfortable berthing conditions for the port users.

To study the wave transformation effects along port approach channel, channel with different side slopes are reproduced and effects of regular and random waves on wave tranquility is studied. For directional propagation waves are generated from different directions and wave crest pattern is recorded at different locations of importance.

The berthing conditions for the port users is mainly dependent on wave tranquility and wave direction at any berthing facility. While planning a berth it is very important to consider both these aspects with due weightage. The physical model studies conducted for port development for various alternatives, the effects of its implementation on field are very useful and this gives a good confidence for engineers while taking decisions in port planning.

1.5 OBJECTIVES OF THE PRESENT STUDY

Following are the objectives of the study

1. To study the wave attenuation along a port approach channel on a scaled physical wave model.

2. The effects of change in approach channel dimensions, varying slopes of channel on the channel wave attenuation and wave tranquility in the harbour basin.

3. The refraction and diffraction effects on the directional wave spread in physical models.

4. The effects of wave absorptive surfaces in the harbour basin on the wave tranquility.

1.6 ORGANISATION OF THESIS

The research work is presented in six chapters. The first chapter gives introduction to the physical model studies in coastal engineering in particular for port and harbour design and development, advantages of physical models are highlighted compared to mathematical models, types of physical models and its specific application is given. The scope of objectives of the present research is presented. The Chapter two discusses on wave propagation along shallow water regions, its complexity in particular while travelling across approach channel and literature review on wave theories importance of wave spectra are presented. Chapter three discusses about experimental investigations, physical model facilities, experimental methods, methodology of study. Chapter four includes the details of model studies for wave attenuation, directional spread and wave tranquility studies with different breakwater configurations and discussions of model results with explanation and application of the research work in modeling and field practices are presented. Chapter five provides the concluding remarks, recommendations and scope for further studies.

CHAPTER 2
LITERATURE REVIEW

2.1 INTRODUCION

Coastal Engineering traces back to the beginning when man began building harbours to protect the navigational vessels and improve its operative conditions. Only after 1930, this branch of engineering recognized as distinct subject and particularly in the last three decades much work has been done in this field to develop systematically the reasons for various complex phenomenon associated with the development of coastal infrastructure across the globe. Understanding of sea waves is one of the most important topics in this branch of engineering. Since the wave form moves forward with significant amount of energy from the location of origin, its properties are very important for researchers, scientists and engineers.

A practicing coastal engineer must have a basic and relatively easy to use theory that defines the important characteristics of two-dimensional waves. This theory is required in order to analyse changes in the characteristics of a wave as it propagates from the deep sea towards near shore regions. Also, this theory will be used as a building block to describe more complex sea wave spectra. The random nature of sea waves has become much clearer and sea waves are described and analysed by statistical theories. The ocean wave spectrum is the working tool of oceanographers and researchers in coastal and offshore engineering. However engineering application of the random wave concept is yet limited to a small number of researchers and engineers (Goda 1999).While it is known beyond doubt that the random wave method need to be applied for any design or planning of near shore structure it is also equally important to understand the wave theory and basic wave characteristics before switching into random wave applications. One such theory is the small amplitude wave theory which is the most important development in understanding sea waves, which is described in the next section.

2.2 SMALL AMPLITUDE WAVE THEORY (SAWT)

Small amplitude wave theory might be best described in practical terms as “a first approximation to the complete theoretical description of wave behaviour”(Ippen 1966). As is the case in many physical problems, the first approximation is very rewarding in that it yields a maximum of useful information for a minimum investment in mathematical endeavour. Since it is an approximation of the true behaviour, however it is necessary to have an estimate of the magnitude of the error involved in its use. We shall see that in many practical situations this error is negligible; while in other cases (such as wave breaking) SAWT does not even predict the existence of the phenomenon of interest.

The small-amplitude theory for two-dimensional, freely propagating, periodic gravity waves is developed by linearizing the equations that define the free surface boundary conditions. With these and the bottom boundary condition, a periodic velocity potential is sought that satisfies the requirements for irrotational flow. This velocity potential, which is essentially valid throughout the water column except at the thin boundary layers at the air–water interface and at the bottom, is then used to derive the equations that define the various wave characteristics (e.g., surface profile, wave celerity, pressure field, and particle kinematics). Specifically, the required assumptions are:

1. The water surface is considered as an ideal fluid, and surface tension forces are negligible. Thus, there are no internal pressure or gravity waves affecting the flow, and the surface waves are longer than the length where surface tension effects are important (i.e., wave lengths are greater than about 3 cm).

2. Flow is irrotational. Thus there is no shear stress at the air–sea interface or at the bottom. Waves under the effects of wind (being generated or diminished) are not considered and the fluid slips freely at the bottom and other solid fixed surfaces. Thus the velocity potential must satisfy the Laplace equation for two-dimensional flow:

$$\frac{\partial^2 \emptyset}{\partial x^2} + \frac{\partial^2 \emptyset}{\partial z^2} = 0 \tag{2.1}$$

$$\text{i.e.} \quad \nabla^2 \emptyset = 0 \tag{2.2}$$

Where x and z are the horizontal and vertical coordinates, respectively.

3. The bottom is stationary, impermeable, and horizontal. Thus, the bottom is not adding or removing energy from the flow or reflecting wave energy. Waves propagating over a sloping bottom, as for example when waves propagate toward the shore in the near shore region, can generally be accommodated by the assumption of a horizontal bottom if the slope is not too steep.

4. The pressure along the air–sea interface is assumed as constant. Thus, no pressure is exerted by the wind and the aerostatic pressure difference between the wave crest and trough is negligible.

5. The wave height is small compared to the wave length. Since particle velocities are proportional to the wave height, and wave celerity (phase velocity) is related to the water depth and the wave length, this requires that particle velocities be small compared to the wave celerity. This assumption allows one to linearize the higher order free surface boundary conditions and to apply these boundary conditions at the still water line rather than at the water surface, to obtain an easier solution. This assumption means that the small-amplitude wave theory is most limited for high waves in deep water. And in shallow water and near wave breaking zone, where the wave peaks up and wave crest particle velocities approach the wave phase celerity. Given this, the small-amplitude theory is still remarkably useful and extensively used for wave analysis. (Ippen 1966)

Figure 2.1 depicts a monochromatic wave travelling at a celerity C on water of depth d in an x, z coordinate system. The x axis is the still water position and the sea bed is at z=-d. The wave surface profile is defined by z= η, where η is a function of x and time t. The wave length L and height H are as shown in the Figure 2.1. Since the wave travels a distance L in one period T,

$$C = \frac{L}{T} \tag{2.3}$$

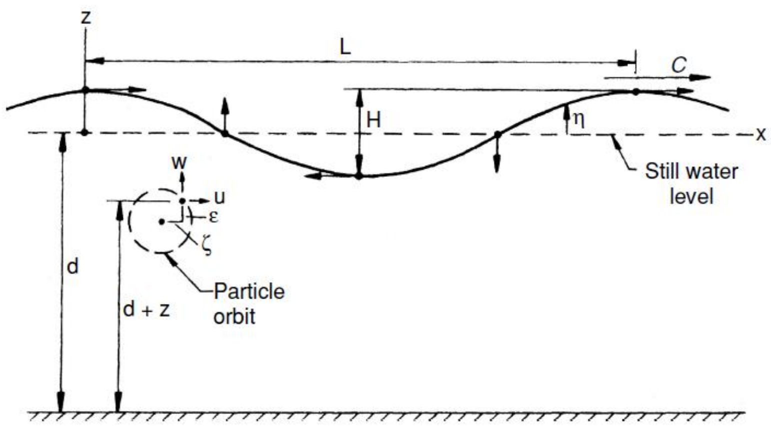

Figure 2.1 Small amplitude wave system, definition sketch.(Ippen,A.T.,1966)

The small-amplitude wave theory is developed by solving Eq.2.1 for the domain depicted in Figure.2.1, with the appropriate boundary conditions for the free surface and the bottom. At the bottom there is no flow perpendicular to the bottom which yields the bottom boundary condition (BBC):

$$w = \frac{\partial \emptyset}{\partial z} = 0 \quad at\ z = -d \tag{2.4}$$

At the free surface there is a kinematic boundary condition (KFSBC) that relates the vertical component of the water particle velocity at the surface to the surface position:

$$w = \frac{\partial \eta}{\partial t} + u\frac{\partial \eta}{\partial x} \quad at\ z = \eta \tag{2.5}$$

The Bernoulli equation for unsteady irrotational flow may be written

$$\frac{1}{2}(u^2 + v^2) + \frac{p}{\rho} + gz + \frac{\partial \emptyset}{\partial t} = 0 \tag{2.6}$$

Where g is the acceleration of gravity, p is the pressure, and ρ is the fluid density. At the surface where the pressure is zero the dynamic boundary condition (DFSBC) becomes

$$\frac{1}{2}(u^2 + v^2) + gz + \frac{\partial \emptyset}{\partial t} = 0 \quad at\ \ z = \eta \tag{2.7}$$

The KFSBC and the DFSBC have to be linearised and applied at the still water level rather than at the a priori unknown water surface. This yields for the KFSBC

$$w = \frac{\partial \eta}{\partial t} \ \ at\ z = 0 \tag{2.8}$$

And for the DFSBC

$$g\eta + \frac{\partial \emptyset}{\partial t} = 0 \quad at\ \ z = 0 \tag{2.9}$$

Employing the Laplace equation, the BBC, and the linearized DSBC, we can derive the velocity potential for the small-amplitude wave theory. The most useful form of this velocity potential is

$$\emptyset = \frac{ag}{\sigma}\frac{\cosh k(z+d)}{\cosh kd}\cos(kx - \sigma t) \tag{2.10}$$

Where

$$k = \frac{2\pi}{L} \quad and\ \ \sigma = \frac{2\pi}{T} \tag{2.11}$$

By using velocity potential function we can find out the velocities, accelerations and displacements.

Velocities

Horizontal velocity $u = -\frac{\partial \phi}{\partial x}$ (2.12)

Vertical velocity $w = -\frac{\partial \phi}{\partial z}$ (2.13)

Accelerations

Horizontal acceleration $a_x = \frac{\partial u}{\partial t}$ (2.14)

Vertical acceleration $a_z = \frac{\partial w}{\partial t}$ (2.15)

Displacement

Horizontal displacement $\xi = \int u\, dt$ (2.16)

Vertical displacement $\zeta = \int w\, dt$ (2.17)

2.3 CLASSIFICATION OF WATER WAVES ACCORDING TO RELATIVE DEPTH

Relative depth is the ratio of the water depth'd' to the wave length L. It is convenient and useful to classify according to this ratio. When a wave propagates from deep water offshore into shallower water near shore the wave length decreases. Thus, the relative depth decreases as a wave approaches the shore (Eagleson et al. 1971). Based on relative depth the classification are given in the Table below

Table 2.1 Classification of waves based on water depth and wave length. (Ref: Dean R.G.1970)

Range of d/L	Range of kd = 2πd/L	Type of wave
0 to 1/20	0 to $\pi/10$	Shallow water waves (long waves)
1/20 to1/2	$\pi/10$ to π	Intermediate depth waves
1/2 to ∞	π to ∞	Deep water waves (short waves)

2.4 WAVE PRESSURE

Substitution of the velocity potential into the linearized form of the equation of motion [without the velocity squared terms] yields the following (Ippen 1966) equation for the pressure field in a wave:

$$p = -\rho g z + \frac{\rho g H}{2}\left[\frac{\cosh k(z+d)}{\cosh kd}\right]\cos(kx - \sigma t) \qquad (2.18)$$

The first term on the right hand side gives the normal hydrostatic pressure variation and the second term is the dynamic pressure variation owing to the wave-induced particle acceleration. Since particles under the crest are accelerating downward, a downward dynamic pressure gradient is required. The reverse is true under a wave trough. Halfway between the crest and trough the acceleration is horizontal so the vertical pressure distribution is hydrostatic.

In deep water, the dynamic pressure reduces to near zero at z = -L/2. A pressure gauge at this depth would essentially measure the static pressure for the given depth below the still water line. The period of the pressure fluctuation is the wave period which can be used to calculate the wave length from the dispersion equation (L_o = 1.56 T^2). The wave height can then be calculated from equation assuming the position of the gauge, the wave period and length, and the water depth is known. Note that the term in brackets differs from the terms in brackets for the particle velocity, acceleration, and orbit displacement equations. At the deep and shallow water limits we have,

$$\frac{\cosh k(z+d)}{\cosh kd} = e^{kz} \quad (deep\ water) \qquad (2.19)$$

$$= 1 \text{ (shallow water)} \qquad (2.20)$$

Thus, from the small-amplitude wave theory, in deep water there is also an exponential decay in the dynamic pressure with distance below the still water line. In shallow water the total pressure distribution is given by

$$p = \rho g(\eta - z) \qquad (2.21)$$

2.5 WAVE ENERGY

The total mechanical energy in a surface gravity wave is the sum of the kinetic and potential energies. The kinetic and potential energies are equal and the total energy in a wave per unit crest width E is:

$$E = E_k + E_p = \frac{\rho g H^2 L}{8} \tag{2.22}$$

Both the kinetic and potential energies are variable from point to point along a wave length. However, a useful concept is the average energy per unit surface area given by

$$\bar{E} = \frac{E}{L*1} = \frac{\rho g H^2}{8} \tag{2.23}$$

This is usually known as the energy density or specific energy of a wave. (Ippen 1966).

2.6 WAVE POWER

Wave power P is the wave energy per unit time transmitted in the direction of wave propagation. Wave power can be written as the product of the force acting on a vertical plane normal to the direction of wave propagation times the particle flow velocity across this plane. (Ippen 1966) The wave-induced force is provided by the dynamic pressure (total pressure minus hydrostatic pressure) and the flow velocity is the horizontal component of the particle velocity. Thus

$$p = \frac{1}{T}\int_0^T \int_{-d}^0 (p + \rho g z)\, u\, dz\, dt \tag{2.24}$$

Where the term in parentheses is the dynamic pressure. Inserting the dynamic pressure and the horizontal component of velocity and integrating leads to

$$P = \frac{\rho g H^2 L}{16T}\left(1 + \frac{2kd}{\sinh 2kd}\right) \tag{2.25}$$

Or

$$P = \frac{E}{2T}\left(1 + \frac{2kd}{\sinh 2kd}\right) \tag{2.26}$$

Let

$$n = \frac{1}{2}\left(1 + \frac{2kd}{\sinh 2kd}\right) \tag{2.27}$$

There for

$$P = \frac{nE}{T} \tag{2.28}$$

The value of n increases as a wave propagates toward the shore from 0.5 in deep water to 1.0 in shallow water. The above Equation indicates that n can be interpreted as the fraction of the mechanical energy in a wave that is transmitted forward each wave period.

As a train of waves propagates forward the power at one point must equal the power at a subsequent point minus the energy added, and plus the energy dissipated and reflected per unit time between the two points. As a first order approximation in the analysis of waves propagating over reasonably short distances it is common to neglect the energy added, dissipated, or reflected, giving

$$P = \left(\frac{nE}{T}\right)_1 = \left(\frac{nE}{T}\right)_2 = constant \tag{2.30}$$

As the two-dimensional wave travels from deep water to the near shore the energy in the wave train decreases at a rate inversely proportional to the increase in n since the wave period is constant.

As waves approach the shore at an angle and propagate over irregular hydrography they vary three-dimensionally owing to refraction. If we construct lines that are normal or orthogonal to the wave crests as a wave advances and assume that no energy propagates along the wave crest (i.e., across orthogonal lines) the energy flux between orthogonal can be assumed to be constant. If the orthogonal spacing is denoted by B, then the equation can be written as:

$$\left(\frac{BnE}{T}\right)_1 = \left(\frac{BnE}{T}\right)_2 = constant \tag{2.31}$$

Inserting the wave energy

$$\frac{H_1}{H_2} = \sqrt{\frac{n_2 L_2}{n_1 L_1}} \sqrt{\frac{B_2}{B_1}} \tag{2.32}$$

The first term on the right represents the effects of shoaling and the second term represents the effects of orthogonal line convergence or divergence owing to refraction. These are commonly called the coefficient of shoaling Ks and the coefficient of refraction Kr respectively. It can also be written as

$$\frac{H}{H_0} = \sqrt{\frac{L_0}{2nL}} \sqrt{\frac{B_0}{B}} \tag{2.33}$$

Or

$$\frac{H}{H_0} = \frac{H}{{H^1}_0} \sqrt{\frac{B_0}{B}} \tag{2.34}$$

Where the prime denotes the change in wave height from deep water to the point of interest considering only two-dimensional shoaling effects.

The nature of the effects of energy transfer to and from waves by surface and bottom are discussed below. Bottom effects, of course, require that the water depth be sufficiently shallow for a strong interaction between the wave train and the bottom.

2.6.1 Wave Reflection

If the bottom is other than horizontal, a portion of the incident wave energy will be reflected seaward. This reflection is generally negligible for wind wave periods on typical near shore slopes. However, for longer period waves and steeper bottom slopes wave reflection would not be negligible. Any sharp bottom irregularities such as a submerged structure of sufficient size will also reflect a significant portion of the incident wave energy.

2.6.2 Wind Effects

Nominally, if the wind has a velocity component in the direction of wave propagation that exceeds the wave celerity the wind will add energy to the waves. If the velocity component is less than the wave celerity or the wind blows opposite to the direction of wave propagation the wind will remove energy from the waves. For typical non-stormy wind conditions and the distances from deep water to the near-shore zone found in most coastal locations, the wind effect can be neglected in the analysis of wave conditions near-shore.

2.6.3 Bottom Friction

As the water particle motion in a wave interacts with a still bottom, an unsteady oscillatory boundary layer develops near the bottom. (O'Brien and Chaffin 1942). For long period waves in relatively shallow water this boundary layer can extend up through much of the water column. But, for typical wind waves the boundary layer is quite thin relative to the water depth, and if propagation distances are not too long and the bottom is not too rough, bottom friction energy losses can be neglected.

2.6.4 Bottom Percolation

If the bottom is permeable to a sufficient depth, the wave-induced fluctuating pressure distribution on the bottom will cause water to percolate in and out of the bottom and thus dissipate wave energy.

2.6.5 Bottom Movement

When a wave train propagates over a bottom consisting of soft viscous material the fluctuating pressure on the bottom can set the bottom in motion. Viscous stresses in the soft bottom dissipate energy content of the waves**.**

2.7 TRANSFORMATION OF WAVES

The change of wave energy will lead to transformation of wave. Different ways for transformation of waves are

- Gradual changes in the channel geometry
- Abrupt changes in the channel geometry

The transformation of waves due to gradual changes the geometry can be estimated with energy conservation equations and calculations are simpler as compared to abrupt changes in channel geometry.

The transformation of waves due to abrupt changes in the channel geometry will be having many assumptions while estimating the transformation changes along the channel. The details of these two methods are given below.

a. Gradual changes in channel geometry

The flux of wave energy past two vertical sections in three dimensional open channels of arbitrary shape. The waves will be assumed to have a crest length much longer than the wave length in the direction of propagation. We can then write, from conservation of energy

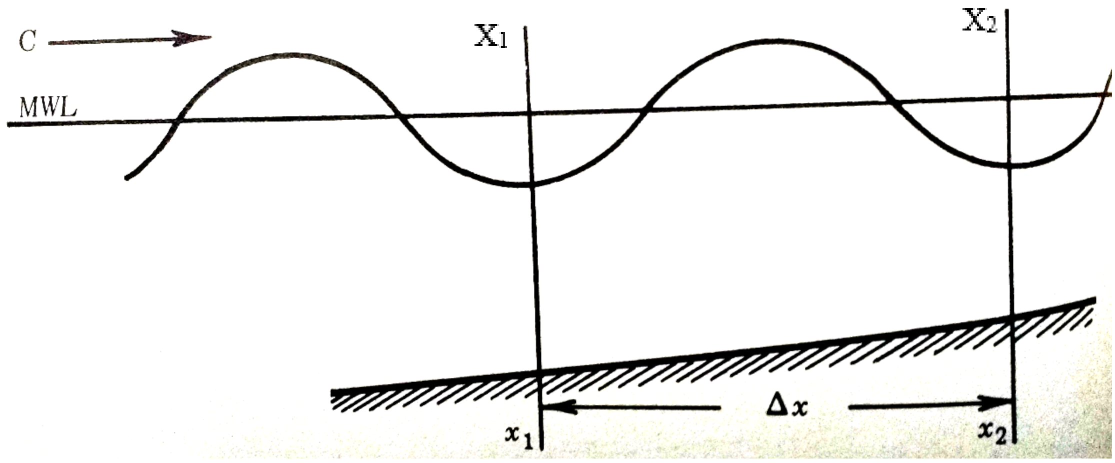

Figure 2.2 Definition sketch for wave transformation.

Incoming flux of energy across x_1x_1-outgoing flux of energy across x_2x_2 = total rate of energy of fluid between x_1 and x_2 .

Letting E= average wave energy per unit of surface area and b=local channel width or distance along a refracting wave crest between adjacent orthogonal, we can express the statement of energy conservation analytically as

$$[EbC_G]_{x_1} - [EbC_G]_{x_2} \; = -\overline{P_g} + \overline{P_r} + \overline{P_d} \qquad (2.35)$$

If it is assumed that over a distance $\triangle x$, regardless of its length, there is no generation, reflection or dissipation of wave energy, therefore the equation becomes

$$[EbC_G]_{x_1} = [EbC_G]_{x_2} \qquad (2.36)$$

Therefore the final equation for the transformation of wave steepness is (from reference 1)

$$\frac{H}{L}\frac{L_0}{H_0} = \left[\frac{b_0}{b}\right]^{1/2}\left[\frac{2\cosh^2 kd}{2kh+\sinh 2kd}\right]^{1/2}\coth kh \qquad (2.37)$$

It should be noted, that for two dimensional channels (b_0=b), the transformation ratios C/C_0, H/H_0 etc may be expressed solely in terms of the parameter d/L_0. This is particularly convenient from the standpoint of practical calculation of transformed wave characteristics at any depth h.

b. Abrupt changes in channel geometry

When changes in channel geometry occur in an abrupt fashion, the assumption of negligible reflection made in deriving equation 2.36 is not valid.

It was seen for reflection of a wave train a reflection coefficient K_r may be defined:

$$K_r = \frac{reflected\ wave\ amplitude}{incident\ wave\ amplitude} = \frac{a_r}{a_i} \leq 1 \tag{2.38}$$

It was also seen that this reflection, either partial or complete, produces a stationary spatial envelope of wave height given by

$$H^*{}_{max} = 2(a_1 + a_2) \tag{2.39}$$

$$H^*{}_{min} = 2(a_1 - a_2) \tag{2.40}$$

The reflection coefficient can then be written in terms of this spatial envelope:

$$K_r = \frac{H^*{}_{max} - H^*{}_{min}}{H^*{}_{max} + H^*{}_{min}} = \frac{a_r}{a_i} = \frac{H_r}{H_i} \tag{2.41}$$

Most practical problems in reflection do not involve simple, regular boundaries and hence can at this time be solved only by experiment.

2.8 DISSIPATION OF WAVE ENERGY

The dissipation of wave energy is due to numerous reasons like bottom friction, viscous nature of the fluid mainly due to suspended material in the water, wave run-up, energy spent on spending beaches, difference in wave celerity due to varied depths near shallow water regions leading to refraction effects, effects of wave reflection at regions like entrance channel slopes, with in the dock arm, along the berthing structures etc., here some are absorptive in nature and some are reflective in nature influencing the wave heights in the region under consideration.

2.9 FINITE AMPLITUDE WAVES

The small amplitude wave theory is based on the assumption that motions are sufficiently small to allow the free surface boundary conditions to be linearized; in particular, terms involving the wave amplitude to the second and higher order are considered negligible. If the wave amplitude is "large", the small amplitude considerations are not valid, and in any theory it is necessary to retain higher order terms to obtain an accurate representation of the wave motion. (Ippen 1966)

The small-amplitude wave theory was formulated as a solution to the Laplace equation with the required for surface (two) and bottom (one) boundary conditions. But the two surface boundary conditions had to be linearized and then applied at the still water level rather than at the water surface. This requires that H/d and H/L be small compared to unity. Consequently, the small-amplitude wave theory can be applied over the complete range of relative water depths (d/L), but it is limited to waves of relatively small amplitude relative to the water depth (for shallow water waves) and wave length (for deep water waves). There is no general solution to the Laplace equation and three gravity wave boundary conditions. All wave theories require some form of approximation or another. Typically, the finite-amplitude wave theories relax the requirement that either H/d or H/L be small to produce a theory that is applicable for finite amplitude waves over some specific range of wave conditions. A finite H/d yields a theory useful in shallow water whereas a finite H/L yields a theory more appropriate for deep water.

2.10 WAVE GENERATION

The term generation of wave is used to define the growth of the wave, increasing the wave height and period, with respect to time and distance, due to the action of wind on the water surface.

The duration of time that the wind act on the water is called *wind duration.* The distance over which the wind blows is called the *fetch.* The minimum duration, which may equal to or less than the actual duration, is that duration of the wind required to establish steady-state generation for a particular wind speed and length of fetch. On the other hand, there is a term, called minimum fetch, which may be equal to or less than the actual fetch for a stationary storm depending on the duration of the wind.

The above definitions are applicable to various theories on wave generation. The transfer of energy from wind to wave, and resulting wave growth, is not completely understood, and all wave forecasting relationships are adjusted by use of actual wave data. When a theory is presented for wave generation, wave data must be used either to verify the theory or to calibrate the relationship, in which case element of ignorance is bypassed.

One of the problems is how to present the wave data which can be used by ocean engineers for design purposes. The most widely accepted method is the use of the significant wave (i e., the average of the highest one-third of the waves) as the design wave. The significant wave may be determined from field measurements during several storms, or it can be predicted by wave fore- casting or hind casting techniques, such as those suggested by Sverdrup-Munk- Bretschneider (1951, 1957), Pierson-Neumann-James (1955) and Darbyshire (1963). Based on the significant wave, many shore structures have been designed with certain safety factors. In recent years, some questions have been raised as to whether or not a structure should be designed for maximum wave height (extreme values), allowing due consideration for the frequency distribution. The distribution of wave periods becomes more important when resonance is of major concern Therefore; the method of presenting wave data requires further investigation in order to provide ocean engineers not only with significant wave data, but with extreme values and frequency distribution as well.

From a practical point of view, the semi-empirical relations for wave forecasting can be classified by two methods

1. The significant wave method
2. The wave spectra method

2.11 PROPAGATION OF WAVES AND SWELLS INTO SHALLOW WATER

When waves or swells propagate into shallow water, a number of modifications take place: refraction, shoaling, and energy losses. These factors will be discussed by considering only a simple sinusoidal wave.

If the waves are long crested and are moving obliquely towards a shore line with depth contours straight and parallel to the shore, those portions of the wave front which efficiently feel bottom first are retarded first, so that the wave become subjected to a progressive curving or refraction which in its overall effect, tends to align the wave front to the depth contours. The spectrum of ocean waves enters a refracting area, the different frequencies or wavelengths are sorted, and the resulting spectrum is changed accordingly.

The orthogonal (wave rays) represent the directions that the wave fronts are taking. They become curved in the process of refraction and in general may tend to diverge or converge. (Sleath 1984) Over a submarine canyon the waves will always tend to diverge, and conversely, over a submarine ridge the waves will always tend to converge.

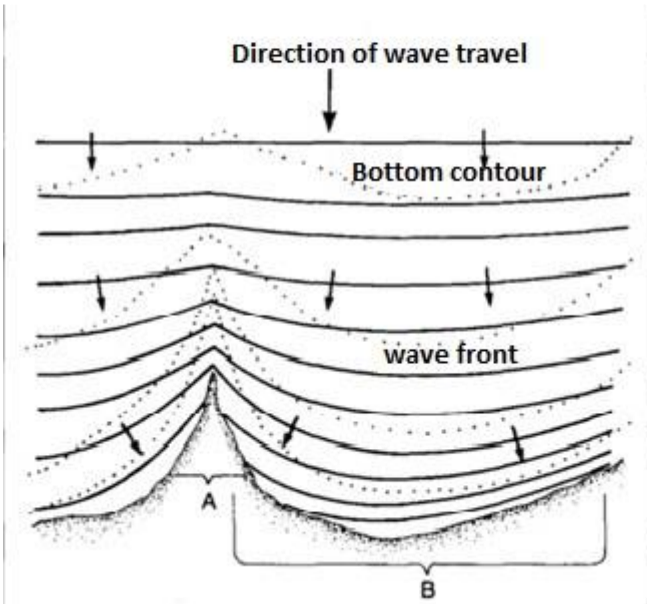

Figure 2.3 Refraction of ocean waves approaching a coast.(ref: Dean R.G.1984)

In the Figure 2.3, bathymetric contours (dotted lines) shoal shoreward. Wave crests (solid lines) propagate in direction of arrows or “rays.” Waves converge toward submarine ridge and associated point of land, A, and diverge outwards within the embankment.

It is generally assumed that the wave energy contained between orthogonal remains constant as the wave front progresses, and this suppose that there is no dispersion of energy laterally along the front, no reflection of energy from the rising bottom, and no loss by other processes. Let b_0 represent the distance between orthogonals in deep water, and b_x the distance between orthogonals somewhere in shallow water, with wave heights H_0 and H_x, respectively. Since the energy of the wave is proportional to the square of the wave height, it follows that

$$b_0 {H_0}^2 C_{G_0} = b_x {H_x}^2 C_{G_x} \tag{2.42}$$

Or

$$H_x = H_0 \sqrt{\frac{b_0}{b_x}} \sqrt{\frac{C_{G_0}}{C_{G_x}}} \tag{2.43}$$

Where $\sqrt{\frac{b_0}{b_x}} = K_r$, called the refraction coefficient. Similarly we can find the shoaling coefficient and friction-percolation coefficient.

2.12 GENERATION OF WIND WAVES IN SHALLOW WATER

Wind-generated water waves are considered the most significant phenomenon confronted by ocean engineers in the design of protective shoreline structures and the prediction of response of offshore floating structures moored in the open sea. This type of wave is distributed in random form compared to the long-period waves generated from an artificial disturbance, such as earthquakes and underwater explosions. The spectrum of the wind-waves varies with wind speed, wind duration, and fetch over which the wind blows. The topography of the local ocean bottom is often the governing factor in the modification of wave spectra in shallow water.(Massel 2013).

If $d/gT^2 < 2.5$ feet/sec^2, then the waves effectively "feel bottom" and the depth and bottom condition enter as additional factors with respect to the heights and periods of wave which can be generated. The effect of frictional dissipation of energy at the bottom for such waves limits the rate of wave generation and also places an upper limit on the wave heights which can be generated by a given wind speed and fetch length.

The establishment of a numerical procedure for computing the wind waves in shallow water of constant depth which can be verified by use of wave data.

2.13 STATISTICAL AND SPECTRAL ANALYSIS OF RANDOM WAVES

The most important concept for random waves is the wave spectrum defined below. Within the approximations which are built into linear wave theory itself, the spectrum basically gives us all properties we need about the waves, as it is in sea state.

Under normal conditions the wave spectrum and hence the sea state is likely to be constant over, say half an hour. The properties of the sea for a constant sea state is covered by what is denoted short term wave statistics. Short term wave statistics deals with the properties of the individual waves, typically the probability distributions of wavelength, period, height and so on. For time periods longer than a few hours, the sea state is likely to vary. Variations in the sea states are covered by a random theory and described by long term wave statistics.

For coastal and ocean engineering, it is very important to know the extreme sea-state conditions the structures are likely to encounter during their lifetime, and this part of the long term statistics is treated by extreme wave statistics. Extreme wave statistics provides methods to estimate how rough conditions are likely to happen at a given location over a time span of, say 100 years.,.

Looking at the realistic, it is composed of number of waves with different directions, frequencies, phases and amplitudes. For an adequate description of sea surface, understanding random wave process and analysis for practical application are important.

2.13.1 REAL FIELD WAVES

Idealization into linear superposition of harmonic waves for all practical purposes with discrete in space.

On the assumption of a random wave field follows stationary and ergodic process, the spatial distribution of wave profile is idealized into time series of sufficiently long, however, enough for the analysis.

Figure 2.4 The random surface as a superposition of plane wave (Ref: Mansard and Funke, 1998)

The stationary property implies the invariant statistical properties if the time window is moving over a random process. And, the probability distribution is same for any sufficiently longer time window over the period of interest.

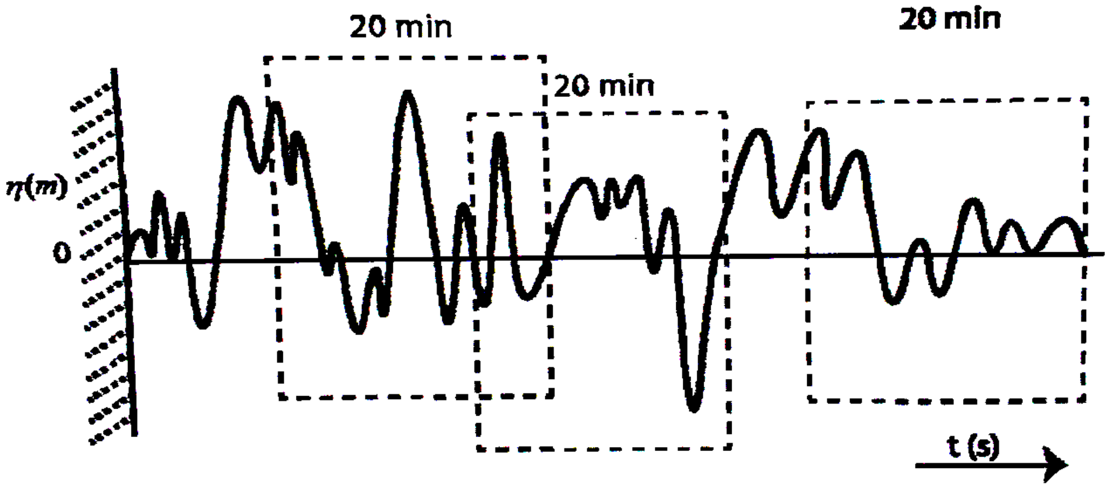

Figure 2.5 A Typical wave record

In real ocean wave field, a typical measurement of 20 min. record is sampled to represent a wave field of 3-hr (Fig 2.5). The time averaged statistics over 20 min times record is equal to the time averaged statistics of every 20min record within the 3-hr time interval. And, in addition, the event averaged statistics is also equal to the time averaged statistics of one frame following the ergodicity. If the stationary,

ergodicity process is not valid, one has to measure for a longer period to take statistical averages to represent a wave climate for 3-hr intervals.
Ergodic: Ensemble (total collection of samples) average (at any time, t_1) is equal to time average of any record.

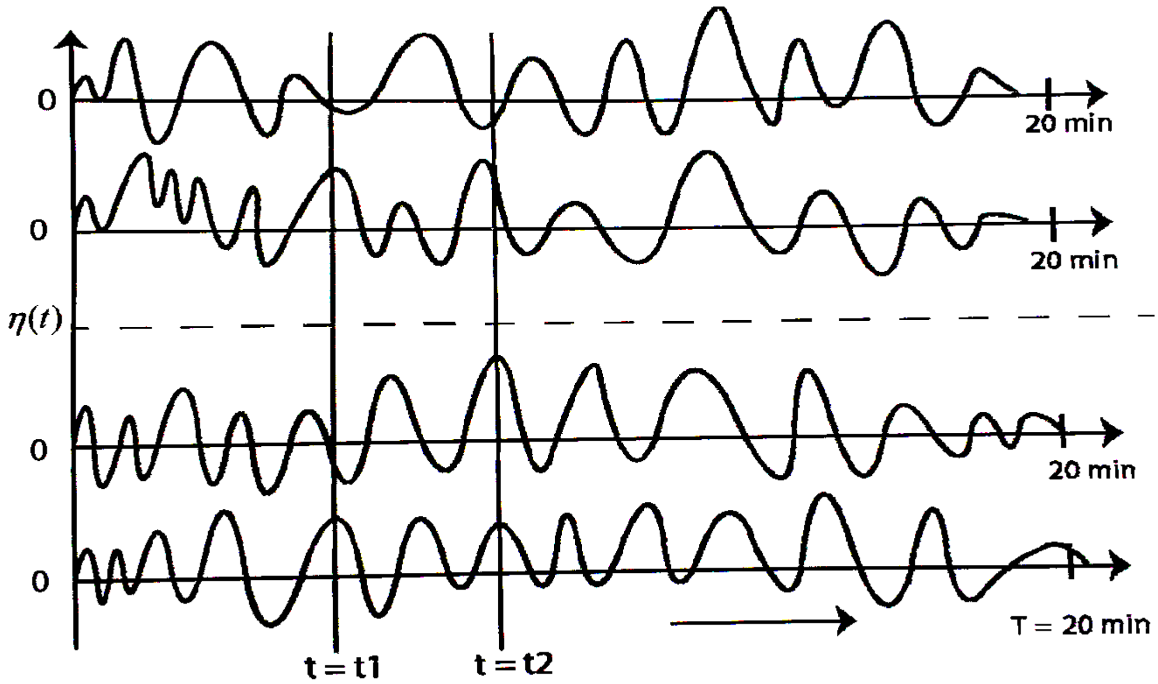

Figure 2.6 Discrete 20min wave record from Fig 2.5

A simple approach to represent such a random event is the concept of the spectrum of ocean waves. The spectrum gives the distribution of wave energy among different wave frequencies on the sea surface. Let us concentrate on statistical and spectral analysis of a random wave process.

The random process such as a random wave, η(t) can be analysed either in the time domain or frequency domain. The assumption of linear superposition (and hence the process is assumed to be linear) makes a good correlation between two type of analysis such as statistical (time domain) and spectral (frequency domain) analysis.

2.13.2 Statistical Analysis

It is the direct analysis without subjecting the time series into any conversion process. Hence, it is valuable and can be taken as primary information as far as the physical representation along the time scale and the ordinate scale (it can be elevation or pressure).

If we measure the length scale of the time series of consecutive waves, we can have,

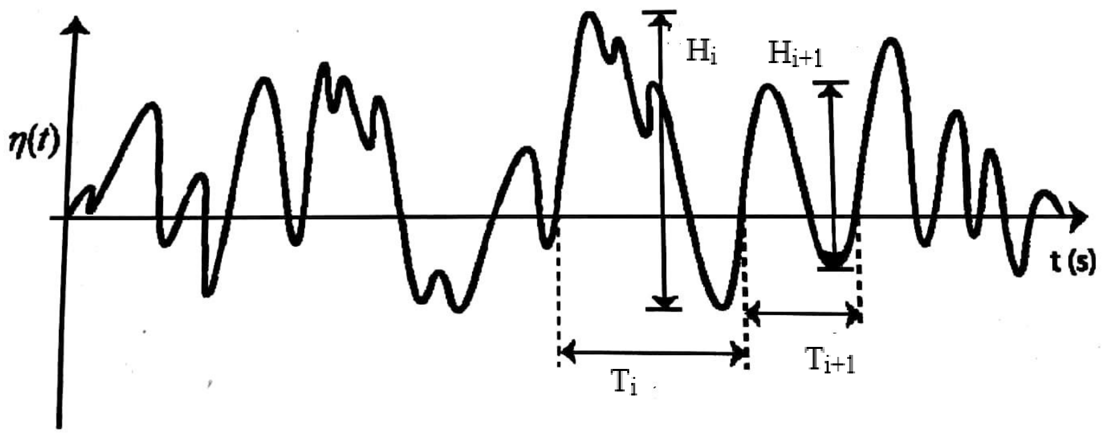

Figure 2.7 Representation of individual wave record within the time series.

Along the horizontal time scale, T_1, T_2 ... T_n for "n" numbers of wave.

Along the vertical wave elevation scale, H_1, H_2 ...H_n for "n" number of waves.

The origin of i^{th} wave is defined from the point where the time series is crossing the mean value (here, zero for all the zero mean process) and the slope of time series has positive value at the origin of i^{th} position. This is called upcross analysis. On the other hand, if the starting point of time series is such that at the zero crossing, at the mean value, the progressive slope is negative, then it is called downcross analysis.

These are two types of analysis and which one to choose depends on the variable under consideration. For the wave surface elevation, if we observe the time progress of the event, η(t), the wave progress from right to left and our time scale is positive from left to right. Hence, in the real field, the trough accompanying a crest crosses the 'time' step forward than the following crest. Hence, downcross analysis is preferred. However, if the convention is different, the upcross analysis can be adopted. (Liu and Frigaard 2001)

There should not be any ambiguity between two users who wants to get the wave field estimate at a same location. So, it is important to define a unique parameter which is also matching with a visual field observer. The estimate of the visual observer in general is found to be correlated with the average of the 1/3rd of the largest wave height among the group. Hence for the general condition, a definition for this value is defined as "significant wave height" which is defined as the average of the highest one third of the wave.

i.e., rewrite the group of values, H_i in descending order of n values. Then,

Significant wave height,

$$H_{1/3} = \frac{\sum_{i=1}^{n/3} H_i}{n/3} \tag{2.44}$$

It is most concern for an engineer to find the maximum value and other statistical properties useful for design and operational purposes.

Different statistical properties can be defined as follows from the list of descending order.

Maximum wave height, $H_{max}=H_1$

Average of highest 0.2% waves, $H_{1/500} = \frac{\sum_{i=1}^{n/500} H_i}{n/500}$ (2.45)

Average of highest one-hundredth, $H_{1/100} = \frac{\sum_{i=1}^{n/100} H_i}{n/100}$ (2.46)

Average of highest one-tenth, $H_{1/10} = \frac{\sum_{i=1}^{n/10} H_i}{n/10}$ (2.47)

Mean wave height, $\overline{H} = H_{av} = \frac{\sum_{i=1}^{n} H_i}{n}$ (2.48)

The above calculation requires a long time series of sufficient record.

Therefore the distribution of wave heights, H_i is found to follow Rayleigh probability distribution which addresses our concern for the estimate. Following Rayleigh distribution, the various statistical parameters can be estimated from the characteristic wave height, i.e., significant wave height, $H_{1/3}$.

Average of highest 0.2% waves, $H_{1/500} = 1.91H_{1/3}$ (2.49)

Average of highest one-hundredth, $H_{1/100} = 1.67H_{1/3}$ (2.50)

Average of highest one-tenth, $H_{1/10} = 1.27H_{1/3}$ (2.51)

Mean wave height, $\overline{H} = H_{av} = 0.63H_{1/3}$ (2.52)

Since, the Rayleigh distribution has no upper bound, the maximum wave height; H_{max} could not be estimated from the characteristic estimate.

Therefore approximate maximum wave height given as, $H_{max} = 2H_{1/3}$ (2.53)

And the root mean square wave height (H_{rms})

$$H_{rms} = \sqrt{\frac{\sum_{i=1}^{n} {H_i}^2}{n}} \tag{2.54}$$

The above process can be used for any variable of interest. However, depending on the distribution of variables, the fitting coefficients have to be carefully chosen.

2.13.3 Definition of Some Wave Parameters

The parameters encountered during analysis of wave records are described below:

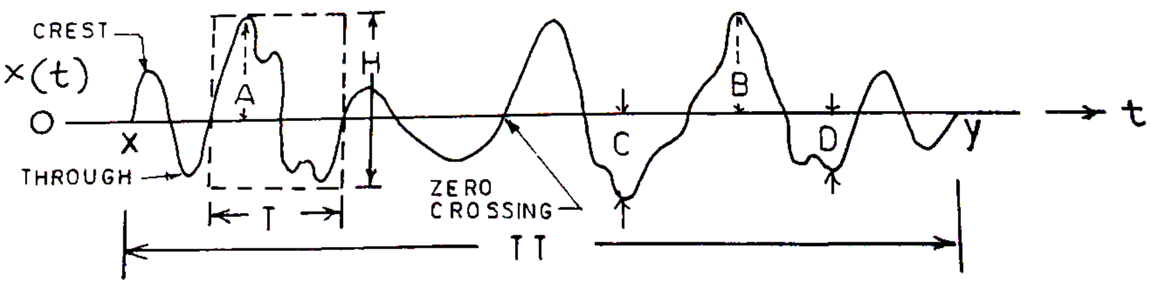

Figure 2.8 Wave parameters for analysis

Mean line (xy): the hypothetical line about which the water surface is assumed to fluctuate due to wave action (drawn by eye judgement).

Total time (TT): the total duration of wave record in sec.

Zero-up-cross: considered to occur when the water surface elevation passes through the mean line in upward direction.

Wave height (H): The vertical distance between the highest and the lowest point of the wave in meters.

Wave period (T): the period between two successive zero-up-crossings of the waves under consideration in sec.

Wave crests: the points in the record, on either side of which the water surface elevation falls.

T_Z = the mean zero-up-crossing wave period

Hs = the significant wave height defined as the average height of the highest one- third of the waves

Hmax = the most probable height of the largest zero-up-cross wave expected to occur during the recording interval of 3 hours

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2.13.4 Tucker's method for analysis of sea wave records

This is an approximate method of wave analysis using drum chart wave records installed with wave raider buoys. It was in use till 1990s and with the advancement with the wave data collection techniques and instrumentations system. The method is obsolete today, however for basic understanding of wave data analysis this is a good exercise.

Notations:

Hmax = Maximum height of the wave in a record.

Ts = Wave period compounding to Hmax.

Nc = No. of crests in a period (i.e. total time TT).

Nz = No. of zero up-crossing waves.

Tz = Mean zero-up-crossing wave period = $\frac{TT}{Nz}$

A = Height of highest crest.

B = Height of 2nd highest crest.

C = Depth of lowest trough.

D = Depth of 2nd lowest trough.

Hrms = Root mean square wave length.

The procedure for the analysis is explained below. (Narashimhan et. al. 2002)

1. Select a typical 20 minute length of record.
2. Draw a mean water level line approximately by eye using judgement.
3. Count the number of crests (Nc). A crest is defined as a point where the water level is momentarily constant, falling on either side
4. Count the number of zero up-cross waves (Nz).
5. Measure the height of the highest crest (A) and the height of the second highest crest (B) from the mean water level.
6. Measure the depth of the lowest trough (C) and the depth of the second highest crest (B) from the mean water level taking both the quantities as positive.
7. H1 = A+C
8. H2 = B+D
9. $\emptyset = \ln(Nz)$

10. Calculate

$$H_{rms1} = \frac{H1}{2(2\emptyset)^{\frac{1}{2}}(1+0.289\emptyset^{-1}-0.247\emptyset^{-2})} \quad (2.55)$$

$$H_{rms2} = \frac{H2}{2(2\emptyset)^{\frac{1}{2}}(1-0.211\emptyset^{-1}-0.103\emptyset^{-2})} \quad (2.56)$$

11. $H_{rms} = (H_{rms1} + H_{rms2})/2$
12. By using the following graph, find out the value of $Hrms_1/H_1$ and $Hrms_2/H_2$ corresponding to the number of waves (N_z)

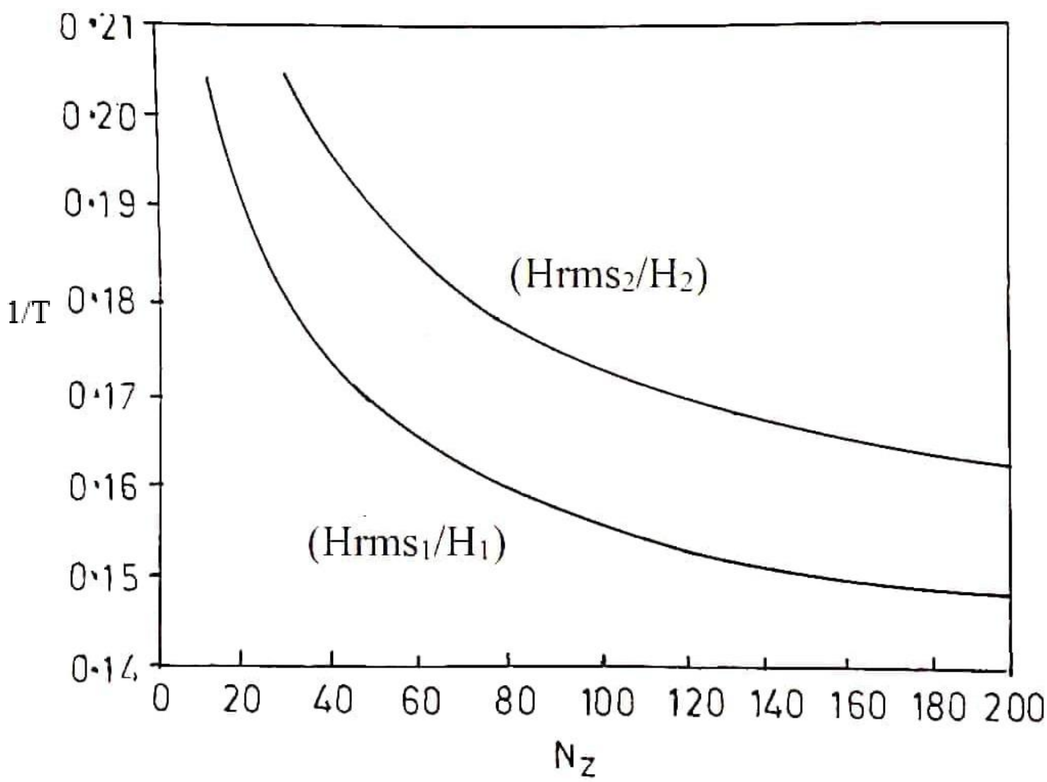


Figure 2.9 Graph for Hrms

13. Now find the value of Hrms corresponding to H_1 and H_2.
14. Hs1 $=Hrms_1*4.0$
15. Hs2 $= Hrms_2*4.0$
16. Significant wave height Hs $=(Hs_1+Hs_2)/2$

2.13.5 Short term analysis

The short term stationarity of the wave statistics, the values of Tz, Hs and Hmax (3hr) are computed from each 20 minute record, which is assumed to characterise the corresponding interval of three hours. The procedure is described below:

1. Mean zero-up-crossing wave period(Tz)

$$T_z = \frac{TT}{N_z} \tag{2.57}$$

2. Significant wave height (Hs):

This is defined as the average height of the highest one third of the waves of the wave record under consideration. Actual procedure for estimation of Hs requires noting down the wave height of each wave of the record, putting them in descending order and taking the average of the wave heights of the first higher one third of the wave.

3. Estimation of Hmax:

A three hour period is considered to be centered on a twenty minute record under consideration and the probability density function of wave heights is used for the purpose of extrapolation in order to obtain the value of Hmax (3 hr) from each 20 minutes record.

Number of waves (N) expected to occur in three hours is given by

$$N = (3 \times 60 \times 60)/Tz \tag{2.58}$$

Making use of the theoretical work of Cartwright and Longuet Higgins (1963), on statistical properties of waves, Hmax is estimated follows:

$$(Y)^2 = \ln N \tag{2.59}$$

$$\Psi = (0.566405 + 0.316548Y + 0.330573Y^2 + 0.073968Y^3 + 0.006361Y^4)^2$$

$$Hmax\ (3hours) = 2\sqrt{2\Psi} *\ Hrms \tag{2.60}$$

Another useful parameter in short-term analysis is the band width parameter (ε) which is very useful in establishing whether the prevailing wave climate is due to sea condition or swell condition. The band width parameter can also be estimated using higher-order moments. The band width parameter tends to unity in sea condition and tends to zero in swell condition and is estimated as follows:

$$\varepsilon = \sqrt{1 - (\frac{N_z}{N_c})^2} \tag{2.61}$$

2.13.6 Long term analysis

The long term analysis is usually required for estimation of 'design wave' expected to occur over the anticipated life span of the proposed structure. The values of Hmax obtained from a number of records are required to be projected over the life of the structure for obtaining the 'life time wave'. It is advantageous to have round-the-year data in order to have a realistic picture of the statistical characteristics of waves occurring in the region since the technique of extrapolation is employed for estimating the life time wave from limited recorded data.

The 'N' values of Hmax (3hr) obtained from 'N' number of 20 minutes record spread over the period of observations are grouped in small ranges to yield number of events corresponding to each group.

$$Prob(H_{max}(3hr) \geq H) = \frac{no.of\ events\ when\ H\ has\ been\ equalled\ or\ exceeded}{N+1}$$

2.13.7 Spectral Analysis

Even though statistical analysis provides a comprehensive direct data analysis, the information on the distribution of concentration of wave energy at different frequency bands is lacking. This is supplemented by the frequency domain analysis by decomposing the time series into various frequency components using Fourier Transform. (Goda 2010)

In simple terms, a wave spectrum is the distribution of wave energy as a function of frequency. It describes the total energy transmitted by a wave-field at a given time. Many times a transformation is performed to provide a better or clearer understanding of such a phenomenon. The time represents a sine wave may be difficult to interpret. By using a Fourier series representation, the original time signal can be easily transformed and much better understood.

Only a brief overview of Fourier Transform to obtain a frequency spectrum has been provided here. The salient aspects that need attention in the analysis, particularly to extract design parameters are dealt here.

2.13.8 Auto correlation function

For a random process, x (t), the auto correlation function is defined as the average value of the product of x (t) and x (t+τ).

$$R_x(\tau) = E[x(t).x(t+\tau)] = \langle x(t)x(t+\tau)\rangle \quad (2.62)$$

If x (t) is stationary, $E[x(t)] = E[x(t+\tau)] = m$ (2.63)

$$.\sigma_{x(t)} = \sigma_{x(t+\tau)} = \sigma \quad (2.64)$$

Correlation coefficient $\rho = \frac{R_x(\tau)-m^2}{\sigma^2}$ (2.65)

2.13.9 Spectrum

The concept of a spectrum is based on work by Joseph Fourier, who showed that almost any function x (t) over the interval (-T/2 < t < T/2) can be represented as the sum of an infinite series of sine and cosine functions with harmonic wave frequencies. Conversion of irregular surface elevation into variance spectrum is not simple. We need to decompose the irregular wave into its linear components. The fourier series is used to represent any arbitrary function of time "t".

$$x(t) = \frac{a_0}{2} + \sum_{n=1}^{\infty}(a_n \cos \mathrm{n}\omega'\mathrm{t} + b_n \sin \mathrm{n}\omega'\mathrm{t}) \quad (2.66)$$

Where,

$$a_n = \frac{2}{T}\int_{-T/2}^{T/2} x(t)\cos n\omega' t \; dt, \quad (n = 0,1,2, \dots) \quad (2.67)$$

$$b_n = \frac{2}{T}\int_{-T/2}^{T/2} x(t)\sin n\omega' t \, dt \;, \quad (n = 0,1,2, \dots) \quad (2.68)$$

Where $\omega' = 2\pi f' = 2\pi/T$ is the fundamental frequency, and nf' are harmonic of the fundamental frequency. This form of x (t) is called a Fourier series; a_0 is the mean value of x (t) over the interval.

The above equation can be written in complex form,

$$e^{in\omega' t} = \cos(n\omega' t) + i\sin(n\omega' t) \quad (2.69)$$

And

$$x(t) = \sum_{n=-\infty}^{\infty} Z_n * e^{in\omega' t} \quad (2.70)$$

Where

$$Z_n = \frac{1}{T}\int_{-T/2}^{T/2} x(t)e^{-in\omega' t} \; dt, \quad (n = 0,1,2, \dots) \quad (2.71)$$

Zn is called the Fourier transform of η (t).

The spectrum S (f) of x (t) is:

$$S_{\eta}(f) = Z_n Z_n^{*} \quad (2.72)$$

Where Z* is the complex conjugate of Z. The computation of ocean wave spectra follows from these forms for the Fourier series and spectra.

Thus in complex form of a random signal, x (t) and its Fourier transform can be written as below.

$$X(\omega) = \int_{-\infty}^{\infty} x(t)e^{-i\omega t}\, dt \tag{2.73}$$

$$x(t) = \frac{1}{2\pi}\int_{-\infty}^{\infty} X(\omega)e^{i\omega t}\, d\omega \tag{2.74}$$

$$X(\omega) - Fourier\ transform\ of\ x(t)$$

$$x(t) - Inverse\ fourier\ transform\ of X(\omega)$$

2.13.10 Spectral density

For a stationary process, x (t) goes on forever and the condition,

$$\int_{-\infty}^{\infty} |x(t)|\, dt \ < \ \infty \text{ Is not satisfied}$$

So that theory of Fourier analysis cannot be applied to a sample function. This difficulty can be overcome by analysing its auto correlation function R_x (τ) since, $R_x(\tau \to \infty) = 0$ .For non-periodic wave with zero mean process and the condition. $\int_{-\infty}^{\infty} |R(\tau)|\ dt < \infty$ is satisfied. (Liu and Frigaard 2001).

Given a random process that is stationary and ergodic, with an expected value of zero and autocorrelation R (τ), the power spectral density, or spectrum of the random process is defined as the Fourier transform of the autocorrelation.

$$S(\omega) = \int_{-\infty}^{\infty} R(\tau)e^{-i\omega\tau}\, d\tau \tag{2.75}$$

Conversely, the autocorrelation, R (τ), is the inverse Fourier transform of the spectrum.

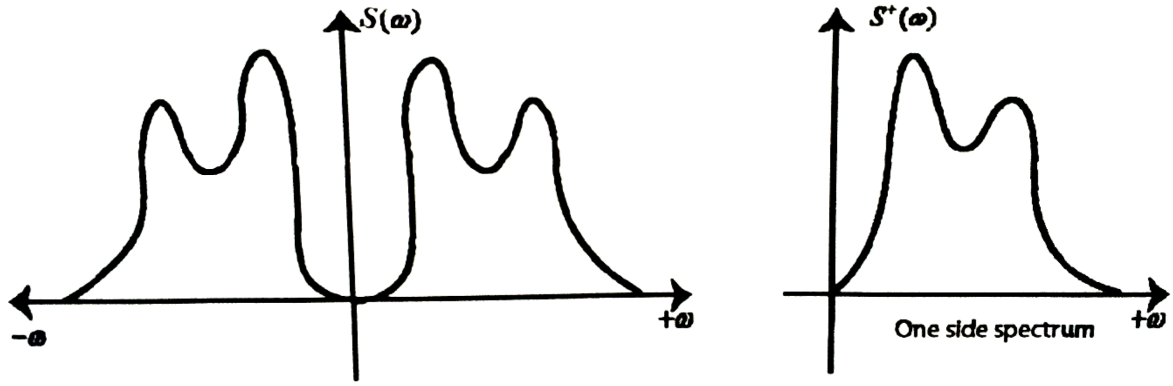
$$R(\tau) = \frac{1}{2\pi}\int_{-\infty}^{\infty} S(\omega)e^{i\omega\tau}\ d\omega \tag{2.76}$$

Properties of the spectrum S (ω) of η(x, t):

- S(ω) is a real and even function, since R(τ) is real and even
- $\int_{-\infty}^{\infty} R(\tau)e^{-i\omega t}\ d\tau = \int_{-\infty}^{\infty} R(\tau)(\cos\omega\tau - i\sin\omega\tau)\, d\tau$
- The variance of the random process can be found from the spectrum: $\sigma^2 = (RMS)^2 = R(0) = \frac{1}{2\pi}\int_{-\infty}^{\infty} S(\omega)\, d\omega$
- The spectrum is positive always: $S(\omega) \geq 0$
- With some restriction it can also be established that

$$S(\omega) = \lim_{T\to\infty}\left|\int_{-T}^{T} x(t, x_k)e^{-i\omega t}\ dt\right|$$

A spectrum covers the range of frequencies from minus infinity ($-\infty < \omega < +\infty$)



S(ω) S⁺(ω)

Figure 2.10 spectral representation

A one sided spectrum, S^+ (ω), is a representation of the entire spectrum only in the positive frequency domain. It can be obtained by folding the energy over ω=0 and introduce the $1/2\pi$ factor we get.

$$S^+(\omega) = \begin{cases} \frac{2}{2\pi}S(\omega) & ,\omega \geq 0 \\ 0 \end{cases} \tag{2.77}$$

This representation for the one sided spectrum comes from the variance, R (0):

$$R(0) = \sigma^2 = \frac{1}{2\pi}\int_{-\infty}^{\infty} S(\omega)\ d\omega = \frac{2}{2\pi}\int_{0}^{\infty} S(\omega)\ d\omega \tag{2.78}$$

Which we can rewrite in terms of one sided spectrum

$$\sigma^2 = \int_{-\infty}^{\infty} S^+(\omega)\ d\omega \tag{2.79}$$

Where, $S^+(\omega) = \frac{2}{2\pi}S(\omega); for\ \omega \geq 0$

The spectrum provides a distributed amplitude, or "probability density" of amplitudes, indicating the energy of the system. Hereafter, S (ω) represent S^+ (ω), i.e., '+' sign is conventionally not added.

We can expand the idea of a Fourier series to include series that represent surfaces η(x, y) using similar techniques. Thus any surface can be represented as an infinite series of sine and cosine functions oriented in all possible directions.

Consider the Radom process, η(x, y) follows stationary and ergodicity process, it is assumed that the expected value of the random process is zero. However it is not always possible. If the expected value equals some constant a_0, the random process can be adjusted such that the expected value is indeed zero,

$$\eta(x, t) = \eta(t, x\,) - a_0 \tag{2.80}$$

The spectrum of the wave-height gives the distribution of the variance of sea-surface height records at the wave staff as a function of frequency. Because wave energy is proportional to the variance the spectrum, it is called the energy spectrum or the wave height spectrum. Typically three hours of wave data are used to compute a spectrum of wave-height.

2.13.11 Window function

In the above frequency representation of a typical time series, it is assumed that the series is continuous. However, in practice, there is a definite time step between successive data. The time step is small enough such that the event is presented as a smooth functional variation over time. This discrete nature of data forces to adopt discrete Fourier transform and in turn add furious noise in the estimate. In addition, a sudden initiation of the event and also, an abrupt end of the event induce higher frequency noises which is otherwise unwanted change in the energy level. This is avoided by introducing a *windowing function*.

In the signal processing, a windowing function is a mathematical function that is zero valued outside some chosen interval. The commonly adopted window functions, w (n) for filtering the noises are

- Cosine tapered window
- Hanning window
- Welch window

2.13.12 Spectral smoothening

Similar to the discrete time series, the derived frequency spectrum is also been specified in discrete frequency step. This resulted in leaking of energy in between discrete steps. This can be rectified by smoothening the spectral curve. 5-point smoothening or higher orders smoothening can be carried out to perform this task.

2.13.13 Statistics

The statistics of η (t) given by the spectrum S_η (ω) needs to be established. The wave heights, Hi and wave periods of interest Ti are the random variable in this problem. As time statistics are equal to the event statistics, if η (t) is a realization of the random process η (t, x) and vice versa. (Liu and Frigaard 2001).

Let the moments of the spectrum are as follows:

Zeroth moment:

$$m_0 = \int_0^\infty S(\omega)\, d\omega = \sigma^2 = variance \tag{2.81}$$

Second moment:

$$m_2 = \int_0^\infty S(\omega)\omega^2\, d\,\omega \tag{2.82}$$

Note, it can be shown that m_1, m_3, etc... are zero (for n odd).

Fourth moment:

$$m_4 = \int_0^\infty S(\omega)\omega^4\ \ d\omega \tag{2.83}$$

The root mean square wave height, H_{rms} or standard deviation, σ_0 is given by,

$$H_{rms} = \sigma_0 = \sqrt{m_0} \tag{2.84}$$

$$H_{s=}4\sqrt{m_0} \tag{2.85}$$

Mean wave height, $\bar{H} = 2.5\sqrt{m_0}$ (2.86)

Average of highest 1/10th waves, $H_{1/10} = 5.09\sqrt{m_0}$ (2.87)

Average of highest 1/100th waves, $H_{1/100} = 6.67\sqrt{m_0}$ (2.88)

The average period, $\bar{T}$ can be found by calculating the centre of the area of the spectrum.

$$\bar{T} = 2\pi\frac{m_0}{m_1} \tag{2.89}$$

The peak period, T_p is the wave period at which the wave energy is maximum. This can be calculated either by differentiating the spectral function or interpreting the spectral values.

$$\overline{T_p} = \sqrt{\frac{m_2}{m_4}} \tag{2.90}$$

The mean zero crossing periods, $\bar{T}_z$ can be estimated as follows.

$$\bar{T}_z = \sqrt{\frac{m_0}{m_2}} \tag{2.91}$$

Hence, $\bar{\eta}(A)$, the average frequency of up crossings past a certain level A (crossing above the threshold elevation A per time) can be estimated as,

$$\bar{\eta}(A) = \frac{1}{T_z} e^{-A^{2/2m_0}} \tag{2.92}$$

$$\bar{\eta}(A) = \frac{1}{2\pi}\sqrt{\frac{m_2}{m_0}} e^{-A^{2/2m_0}} \tag{2.93}$$

And, $\bar{\eta}(0)$, the average frequency of all upcrossings (past a zero level).

$$\bar{\eta}(0) = \frac{1}{2\pi}\sqrt{\frac{m_2}{m_0}} \tag{2.94}$$

2.13.14 Band width of the spectrum

The band width of the spectrum describes how “wide” the spectrum is. For a harmonic signal with one frequency the bandwidth is nearly zero and a narrow spectral peak appears. However in a signal that contains multiple frequencies the bandwidth increases. For white noise the bandwidth approaches one. (Massel 2013)

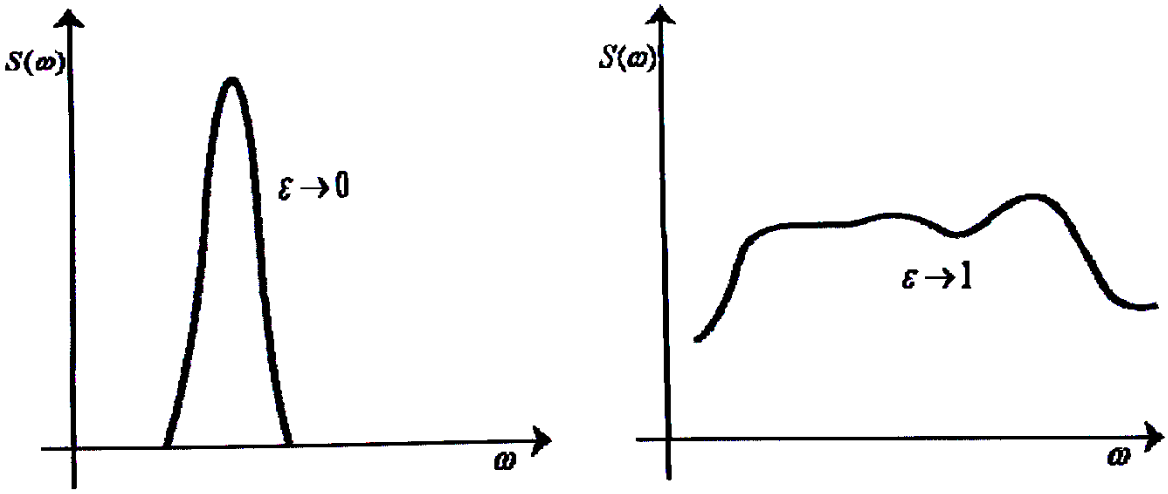


Figure 2.11 Narrow and broad band spectrum

The bandwidth parameter, Ɛ, called the spectral width parameter defines the width of the spectrum. It can be estimated from the spread of energy over the frequencies.

$$\varepsilon^2 = 1 - \frac{\overline{T_p}}{\overline{T_z}} = 1 - \frac{{m_2}^2}{m_0 m_4} \tag{2.95}$$

The value of Ɛ is between 0 and 1. For the ocean wave, a bandwidth between 0.6 and 0.8 is common. In general, Ɛ > 0.6 is called *broad band spectrum* and Ɛ < 0.6 is called narrow band spectrum. In general, it can be said that most sea spectra are relatively narrow banded. It is due to the fact that the very small, high frequency waves (ripple) are of no interest in the prediction of ship response.

It is to be noted that the significant wave height, Hs is dependent on the bandwidth of the spectrum and can be estimated as below.

$$H_s = 4\sqrt{m_0(1 - \frac{\varepsilon^2}{2})} \tag{2.96}$$

On the assumption of narrow bandedness, the significant wave height (Ɛ=0) is given by

$$H_s = 4\sqrt{m_0} \tag{2.97}$$

If the spectrum is wide band

$$H_s = 2.83\sqrt{m_0} \tag{2.98}$$

Maximum wave period obtained from the above procedure may mislead since it would not correspond to maximum wave energy this point we have to account before continuing to analyse in comparison to statistical analysis that we have seen in the earlier section.

2.14 HYDRAULIC MODELLING

A hydraulic model, may be defined as any physical model for the simulation of flow processes, flow states and events, which concern problems of hydraulic engineering or technical hydromechanics.

2.14.1 Hydraulic Similitude

The purpose of hydraulic similitude is to ensure that the model reproduces the behaviour of the prototype as much as possible (Goda 1985), (Hughes 1993), (Young et al. 1997). This similar behaviour includes velocity, acceleration, mass transport, and resultant fluid forces on objects and boundaries. Correspondence between prototype (p) and model (m) is denoted by the scale factor or model scale (NX), which is the ratio of the prototype parameter (Xp) to the model parameter (Xm) defined as

$$N_x = \frac{X_p}{X_m} \tag{2.99}$$

Note that one can sometimes see the inverse of this factor used to represent the scale factor (i.e., model divided by prototype value). This makes the scale factor look like a fraction less than one, as in 1 model-100 prototype. In this the form given in equation 2.99, where the prototype value is divided by the model value, is preferred. This X parameter can represent any derived variable as determined from its dimensions. For instance, fluid velocity (V) has dimensions of length (L) divided by time (t), so the velocity scale (N_V) is given by

$$N_v = \frac{V_p}{V_m} = \frac{(\frac{L}{t})p}{(\frac{L}{t})m} = \left(\frac{L_p}{L_m}\right)\left(\frac{t_m}{t_p}\right) = \frac{N_L}{N_t} \tag{2.100}$$

The three basic laws of similitude are geometric (similarity of form), kinematic (similarity of motion), and dynamic (similarity of forces). In general, the ratio of quantities in the model needs to be the same as in the prototype. For geometric similarity, the ratio of model and prototype lengths must be equal. For kinematic similarity, velocity and acceleration must have the same ratios between model and prototype. For dynamic similarity, the four external forces of gravity (Fg), viscosity (Fv), surface tension (Fs), and elasticity (Fe) must have the same ratios. This requirement arises from Newton's Second Law which states that the inertial force (F_I=ma) equals the sum of these external forces.(Hughes 1993)

In general, almost any modelling situation can be simplified as the interplay between two major forces as the other forces play a minor role. In fluid flow modelling, inertial forces are always present. The ratio of the inertial force to the other forces has led to the development of the similitude criterion that relates the importance of these secondary forces to the inertial force. No fluid can satisfy all the dynamic similarity requirements. Since the same fluid (i.e., water) is usually used for both model and prototype, it is impossible to achieve exact dynamic similarity for water waves. Surface tension and compressibility are generally neglected as they are relatively small. Viscosity can be neglected in most free-surface models if the model is not too small. Thus, the Froude Number (equation 2.101) is the major scaling criterion in coastal and inland models with free-surface flows.

2.14.2 Froude Model Law

The Froude number (Fr) is defined as the square root of the ratio of the inertial force (force due to convective acceleration of a fluid particle) to the gravitational force (weight) as given by:

$$F_r = \sqrt{\frac{F_i}{F_g}} = \sqrt{\frac{\rho L^2 V^2}{\rho L^3 g}} = \frac{V}{\sqrt{gL}} \qquad (2.101)$$

Froude similitude between model and prototype requires that the Froude numbers are equal:

$$F_r = \left(\frac{V}{\sqrt{gL}}\right)_m = \left(\frac{V}{\sqrt{gL}}\right)_p \qquad (2.102)$$

Rearranging and expressing in terms of scale ratios (N_X) of individual variables gives

$$N_v = \frac{V_p}{V_m} = \sqrt{\frac{g_p}{g_m}}\sqrt{\frac{L_p}{L_m}} = \sqrt{N_g}\,\sqrt{N_L} \qquad (2.103)$$

2.14.3 Reynolds Model Law

The Reynolds number (Re) is important when viscous forces dominate, such as laminar boundary layer problems in open-channel or free-surface flows and forces on cylinders in low Re flows. Bottom friction can be significant if the flow depth is relatively small. One way to minimize the influence of viscosity is to ensure that the model flow is in the turbulent range, which occurs for Re above approximately 10^4. The Reynolds number (Re) is defined as the ratio of inertial to viscous forces:

$$R_e = \frac{F_i}{F_v} = \frac{\rho L^2 V^2}{\mu V L} = \frac{\rho V L}{\mu} \tag{2.104}$$

Where ρ is the mass density, μ = dynamic viscosity, and ν (=ρ/μ) is the kinematic viscosity. As done with the Froude number, similitude is achieved when Reynolds numbers are equal in model and prototype:

$$N_\mu = \frac{\mu_p}{\mu_m} = \left(\frac{\rho_p}{\rho_m}\right)\left(\frac{V_p}{V_m}\right)\left(\frac{L_p}{L_m}\right) = N_\rho N_V N_L \tag{2.105}$$

2.14.4 Other Dimensionless Parameters

In addition to the Froude and Reynolds numbers, there are several other dimensionless numbers that can be derived for use in physical models (Young et al. 1997). These include (a) the Weber Number (We) that is important in the study of surface tension, (b) the Cauchy Number (Ca) for the study of elasticity and compressibility in breaking waves, impact forces, and mooring lines, and (c) the Strouhal Number (St) for the study of acceleration effects in unsteady, oscillating flows, and vortices.

2.15 ADVANTAGES AND DISADVANTAGES OF PHYSICAL MODEL STUDY

Some advantages and applications of physical models are listed as follows (Hughes 1993):

- Physical phenomena are often highly nonlinear and not well understood. Physical models usually replicate the physics, both linear and nonlinear, of physical processes very well, without simplifying assumptions.
- Physical models are used to determine empirical coefficients for analytical and numerical models that are not known or only poorly understood.
- Physical models assist in evaluating the effect of simplifying assumptions (i.e., discarding higher order terms) on numerical model predictions.

- Field experiments are very expensive and forcing conditions (i.e., waves, winds, water levels, currents) are difficult to control and systematically vary.
- Models provide repeatability and a controlled environment for calibrating analytical and numerical models. The smaller size permits easier and less expensive data collection of multiple variables such as water levels, pressures, forces, velocities, displacements, etc.
- Physical models provide a hands on look at the processes, allowing one to examine different options relatively conveniently and gain insight (qualitative impression) of the physics governing a process. In spite of the improved graphics of numerical models, most people can relate to the visual feedback and ability to interact with a physical model.
- Prototype construction may be very risky or uneconomical without a model to verify assumptions and performance. In this case, a physical model study may be justified from the standpoint of verifying the prototype design and validating numerical model results.
- Physical models can be used in conjunction with numerical models as a hybrid model to take advantage of their individual benefits. In a hybrid model, one can have numerical physical-numerical or physical-numerical model connectivity. For instance, the numerical model is used to provide input to the physical model which then provides its output as input to the same or another numerical model.

The four main disadvantages of physical models can be listed as follows:

- Scale effects are due to the fact that it is impossible to correctly scale a free-surface hydrodynamic model using water that satisfies all the laws of similitude. To minimize scale effects, one should construct as large a model as possible that fits within time, cost, available facility and space constraints.
- Laboratory effects consisting of model boundaries, instrumentation support, mechanical wave and flow generation losses, etc., are a concern in physical models. In the model, walls are necessary to contain the water, but they induce laboratory effects due to reflection and flow restriction. In the real world, there

are no artificial boundaries to produce these effects. In the model, instruments are typically mounted in some type of rigid support that may have a larger impact on the flow field than a similar instrumentation support in the prototype. However, measures can be incorporated in the model to minimize and mitigate these effects. The generation of waves and currents by mechanical means in a laboratory is not exact between model and prototype. In the real world, waves may have multidirectional characteristics with frequency and directional spreading that are not always possible to simulate as accurately in the laboratory.

- Forcing functions or boundaries in the prototype may not be simulated in the model due to cost and/or practicality. For instance, simulating wind blowing across the surface is rarely included in coastal models. Instead, the wind effect is represented by the equivalent wave conditions at the wave maker or wind effects on the model (i.e., ship heeling due to wind).
- Construction and application of a physical model may not be cost-effective. For relatively straightforward applications, numerical models can be more cost efficient. One must realize that numerical models also have limitations and simplifying assumptions that may significantly affect the results. Trade-offs between physical and numerical model applications should be considered.

2.16 CHOICE OF WAVE THEORIES IN COASTAL ENGINEERING

In coastal engineering the question of which wave theory to be used has been a great academic interest. Linear (Airy, Stokes) wave theory are easy to use. These are adopted in majority of practical application problems. Also in many of the applications it is the appropriate wave theory. However in consideration of shallow water and or higher-order wave theories are substantially complicated. But there is substantial literature and alternative wave theories are available for this domain. The use of alternative wave theories is not easy as the description available for these are complex and sometimes incomplete. This is especially true in prediction of acceleration, pressure components and inclusion of currents.(Sleath 1984)

There is little standardisation in literature regarding;

1) Frames of reference like fixed/moving, located at bed/trough/mean water levels

2) Mathematical formulations (Euler equations, velocity potential function or stream function

3) Definition of dimensionless variables

4) Perturbation parameters of higher order theories.

It is necessary to understand and create correct details to utilise these theories. Many attempts are made by the researchers in this direction and lot many clarifications are made available with little ease with the availability of modern computers in using these theories.

2.17 NEAR SHORE WAVE CLIMATE

It is very difficult to estimate the near shore wave climate because of primary complexities like wave shoaling, refraction, reflection, diffraction, internal diffraction, wave breaking and in addition to this wave decay occurs due to percolation and bottom friction. All these factors add to wave transformation processes in the near shore shallow water regions. Many researchers have worked on these factors and arrived at different methods to estimate the near shore wave climate (Farhadzadeh2011), (Beltami2003), (Ebersole1985). Whereas the estimate of wave transformation parameters due to primary factors like shoaling, refraction, diffraction reflection are well understood with theoretical reasoning and many analytical and mathematical modelling techniques have been developed to study these. The estimates of other complex parameters like internal diffraction, wave breaking, and wave attenuation can be established through sophisticated shallow basin wave modelling techniques. However estimates effects of bed friction and bottom percolation are sensitive to selected values of bottom friction and percolation coefficients, (Galan, 2012). The estimates of these coefficients have shown variation from site to site and also varying at the same site with time. Thus the selection of the coefficient must be regarded as a subject to uncertainty. They have developed a theoretical context for energy losses in surface gravity waves due to spectral transformations in shallow water. To support this physical model data and field data were compared to the predicted values by using the theory developed. Based on the theoretical results and data analysis it was shown a theory based on nonlinear energy fluxes, in which irregular wave breaking is the major energy loss mechanism. This provides a reasonable fit to a wide range of observed wave height reduction without the need for varying any free parameter from site to site. From this it is evident that in shallow basin wave modelling simulation of random waves occupies a great importance.

2.18 WAVE ENERGY LOSES IN SHALLOW REGIONS

A number of investigators have developed theories for energy losses in irregular wave trains in shallow water due to wave breaking (Collins 1970), (Battjes 1974), (Kuo and Kuo 1974) (Goda 1975), (Battjes and Janssen 1978), (Mase and Iwagaki 1982) (Thornton and Guza 1983) and (Galan 2012). In estimating the energy loss rate, a probability density function for wave heights is assumed and the waves greater than some breaking criterion were assumed to break, with a resulting loss in energy. In all these studies wave breaking rather than bottom friction is assumed to provide the primary mechanism for wave height decay in near coastal waters. Thornton and Guza (1983) estimates about 3% of total variation in wave height in their studies is associated with bottom friction.

All studies in this direction have indicated good prediction of wave height decay near a coast at locations inside the surf zones, in spite of general lack in understanding of the precise Physics of wave breaking. The reason for this may be the wave breaking process is treated in probabilistic manner rather than a deterministic one. Thus as long as the formulation for wave breaking are dimensionally consistent and empirical coefficients are approximately correct, the predicted wave heights match with the observed values.

For irregular waves there does not exist an absolute deep water limit where no waves break. Instead for waves of a given steepness, the probability of wave breaking asymptotically decreases as the water depth increases. This presents an interesting question regarding the point, approaching a coast, that wave breaking becomes the dominant energy loss mechanism compared to other mechanisms, such as bottom friction. The bottom friction coefficients are calculated so as to explain the total energy losses , thus the estimates inherently assume that any wave energy losses might be due to irregular wave breaking are negligible. At the regions immediately adjacent to shoreline, the likelihood of breaking induced by instabilities related to the ratio of wave heights to water depth becomes small. But breaking induced by instabilities related to ratio of wave heights to wave length (wave steepness) does not become small very quickly. It is the steepness limited wave breaking that forms the majority of energy sink in most models of irregular wave breaking up to the zone where breaking becomes almost a continuous process (Donald, 1987).

2.19 EFFECTS OF CURRENTS ON WAVES

The effects of steady uniform currents on random waves and the associated water particle kinematics have been investigated by Umeyama (2009). The basic equations describing the interaction between waves and currents were reviewed with special reference to changes in the variance spectra of free-surface displacement and of horizontal water particle velocity. The theoretical predictions and the laboratory measurements of random waves propagating on opposing currents. The theoretical model developed is proved in reasonable agreement with the laboratory observations.

Waves do not propagate on calm water but travel on currents, if the current is positive i.e., it travels with the waves, then the transformation of the random waves as they encounter currents are relatively straight forward to predict. But when the currents are negative i.e. it opposes the waves the effects are very complex in nature. Longuet-Higgins and Stewart (1961) were the first to deal with the interaction between water waves and currents. They introduced concept of radiation stress and showed the existence of energy transfer between waves and currents. Bretherton (1976) later drew attention to quantify which they called wave action. Wave action is important in study of waves on currents as, unlike wave energy, it is conserved in absence of wave generation or dissipation. For linear waves, wave action equals wave energy density/ wave frequency relative to the current. Its introduction led to some simplification in the mathematics of wave current interaction. Instead of employing a relatively complicated energy equation with its physically important radiation stress terms, it became possible to allow for the transfer of energy between waves and currents without the need to explicitly calculate the energy exchange by Zhang (2010).

2.20 MATHEMATICAL FORMULATIONS OF PROGRESSIVE WAVES

Sea waves are progressive in deep water. Whereas near a coastal boundary standing waves are formed by the reflection of incident progressive waves. These are very significant from the estimation of near coast wave climate. Edge waves, both progressive and standing may be also an important role in near coast wave climate prediction. All these waves are periodic because they come in groups or steady wave trains it is possible to identify a period of oscillation. The theory of solitary waves would be of relatively less important for sea bed effects, however as periodic waves move into shallow water their profile becomes progressively more like that of solitary waves. Hence the study of solitary waves is relevant to the dynamics of near shore region Sleath (2002).

Progressive waves of permanent form are steady in frame of reference moving at the phase speed/ celerity of wave C. It is convenient to adopt a steady or moving x, z reference frame that is located at sea bed and moves at speed C with the wave crest rather than an unsteady or fixed reference frame. Assuming the flow as incompressible and irrotational the mathematical formulation may be presented in terms of Euler equations/ velocity potential function/stream function. By choosing the potential function $\varphi(x, z)$ the field equation representing mass and momentum conservation is the Laplace equation.

$$\partial^2 \varphi /\partial x^2 + \partial^2 \varphi /\partial z^2 = 0 \tag{2.106}$$

Where the velocity components (u, w) are $(\partial\varphi /\partial z, \partial \varphi /\partial x)$

This equation is subjected to the following boundary conditions

1. Bottom boundary condition(BBC); No flow through the bed

$$\varphi (x, 0) =0 \quad \text{at } z=0 \tag{2.107}$$

2. Kinematic free surface boundary condition, No flow through free surface

$$\varphi (x, \eta) =-Q \quad \text{at } z= \eta(x) \tag{2.108}$$

Where $\eta(x)$ = free surface; and – Q the constant volume flow rate per unit width under steady wave. Q is positive and this flow is in negative x- direction.

3. Dynamic free surface boundary condition, Constant atmospheric pressure on the free surface,

$$1/2[(\partial\varphi /\partial x)^2 + (\partial\varphi /\partial z)^2] + g\,\eta = R\,, \quad \text{at } z=\eta(x) \qquad (2.109)$$

Where g= acceleration due to gravity and R= Bernoulli constant

4. Wave is periodic in space and time.

The adopted solution method depends on dimensionless parameter $[T^*(g/h)^{1/2}]$.

Where T= wave period and d = mean water depth.

If the values of dimensionless parameter $[T^*(g/h)^{1/2}]$ is less than 10 (deep water), Stokes wave theory is adopted and if greater than 10 (shallow water), Cnoidal wave theory is normally adopted.

Also Fourier approximation theory may be used, regardless of the value of $[T^*(g/h)^{1/2}]$.

The theoretical development of rational and consistent theories from first to fifth order for Stokes and Cnoidal waves, and from first to (theoretically any order) of Fourier approximation waves, have much in common. The major constraints to systematic use of higher order theories in practice have been the uncertainty surrounding the theoretical background and many a times incomplete details of the predictive equations, an experts needs to understand these limitations while adopting these methods. Application of Stokes, Cnoidal and Fourier wave theories is explained by Rodney (1987). Stokes theory for deep water, Cnoidal theory for shallow water and Fourier theory for deep water, transitional and shallow waters are relatively easy to use and give good results within the ranges of validity. Also each of these theories automatically accommodates a uniform flowing current with it, an essential element in any application beyond linear wave theory. Stokes and Cnoidal are available at any order from one to five and Fourier theory at nominally any order, through beyond say, 18^{th} order any corrections are approaching typical machine precision. The range of validity of these is not infinite. It is often possible to achieve a mathematical prediction from a theory outside its range of physical validity. In many cases the inappropriateness of the solution is clear but this is not guaranteed particularly if the entire solution field is not inspected. The problem may be deeply embedded and quite subtle. Identification may be difficult but is facilitated by careful screening of the solutions and comparison with other wave theories on routine basis. Fourier wave theory is not immune from these problems, despite its wider range of physical applicability. The sharp curvatures of shallow water wave profiles require much higher order solution than deep waters. Convergence to higher odd harmonics is also possible in shallow and transitional water, if sufficient care is not exercised. There remains an important role for each of these three theories. Within their respective physical and numerical limits, each theory provides a useful prediction. In most of the cases at least two theories will be reasonably appropriate providing independent confirmation of any wave prediction.

The density of sea water varies to a small extent from one point to other. The variation has a very negligible effect on the dynamics of the surface waves. However density variation has a significant effect on the internal waves. These waves are seldom of importance as far as sea bed is concerned.

Water surface boundary condition is nonlinear, most of the available solutions for the velocities and pressures induced by wave action are based on approximations that are valid only over a restricted range of conditions. The exact solutions to the governing equations have been obtained numerically which require significant computing resources. The available solutions may be grouped as below,

1. Small amplitude theory. The first approximation is often referred to as Airy waves and higher approximations are generally called Stokes higher order waves.

2. Shallow water theory. Under this are Cnoidal waves and a special case solitary waves.

3. Rotational wave theory. The best known example is Gerstner’s theory, referred to as trochoidal theory.

4. Numerical methods. These include the exact solutions of Cokelet and the nonexact but somewhat more manageable stream function and vocoidal theories of Dean and Swart.

The question of which wave theory to use is a choice between convenience and precision. Numerical solutions are laborious but for relatively steep waves give better accuracy than other methods. On the other hand when wave height is not large or when a rough approximation is required small amplitude or shallow water theory is adequate. Rotational theories are rarely used (Sleath, 1984).

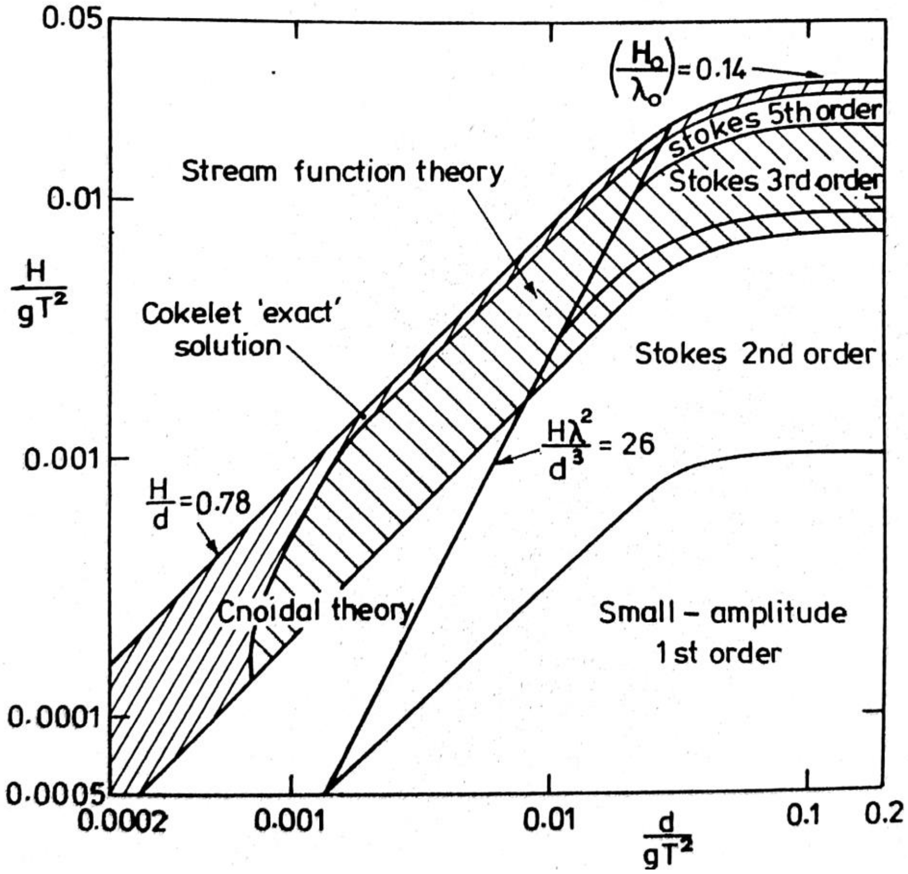


Figure 2.12 Limits of validity of various wave theories. (Dean 1970 and Le Mehaute 1976)

The wave celerity varies with depth according to solitary wave theory, Cnoidal theory and first order small amplitude wave theory. Housley and Taylor (1957) suggested on the basis of experiments in a wave channel that the break even between first order small amplitude and Cnoidal theories was approximately given by

$$H/d = 160 \{d/gT^2\} \tag{2.110}$$

But the Cnoidal theory is inconvenient to use hence many researchers and engineers prefer to use small amplitude and solitary theory. According to Housley and Taylor (1957) the break even between these two theories is approximately

$$H/d = 1600\ (d/gT^2)^{5/4} \tag{2.111}$$

Eqs. (2.110) and (2.111) are based on the measurements of wave speed in wave channel.

2.21 WAVE DECAY IN SHALLOW REGIONS

Umeyama (2008), has reviewed transformation processes and nonlinear properties of internal waves over a uniform slope in a two-fluid system, attempting to reveal water particle kinematics by using particle image velocimetry (PIV). Attenuation and setup induced by wave breaking are predicted using an energy-dissipation model including the radiation stress. In their research the radiation stress is associated with the time-averaged momentum flux owing to the interaction between incident and reflected internal waves. These predictions agree with experimental data obtained by an image processing technique with which the boundary plane between the upper and lower layers could be detected as a density interface. A set of halogen lamps and three high-definition digital video cameras are used to illustrate velocity vector fields during one wave period. Instantaneous velocity fields measured by the PIV system are compared with the calculated velocity distribution by the method of characteristics in combination with the first-order Stokes theory. The rundown behavior is then investigated, and its influence upon the undertow in the lower layer is explained with a simple turbulent-jet model. Furthermore, the theoretical approach for examining mass transport and undertow is discussed in relation to the measured mean velocity by Umeyama (2009).

A theoretical approach was investigated by Sulisz (2011) to describe the diffraction of water waves by a structure in a channel. The solution was achieved by applying the three-dimensional (3D) boundary element method and is valid for a structure of arbitrary shape. The derived model was applied to analyze the effect of the structure geometry and the configuration of the channel on diffracted wave field and on generalized wave-load components. The results show that the wave field in the channel and wave loads on a structure strongly depends on wavelength and the geometry of the structure. The far-field wave amplitudes and wave load components oscillate with increasing wave number, and for selected wave frequencies, the incoming waves are fully transmitted and forces on the structure simultaneously vanish. This result is of practical importance. The oscillations depend primarily on the length of the structure and are associated with the formation of partially standing waves in the sub domain along the structure, which affect wave reflection in the main channel and wave loads on the structure. The effect of the draft and width of the structure on the oscillations is very limited. However, the draft and width of a structure has a substantial effect on the magnitude of wave reflection and transmission, especially for narrow channels. Moreover, the draft and width of a structure has a substantial effect on wave-load components. The analysis of the results indicates that it is difficult to simplify a theoretical description because, even for a channel of a constant depth and ideal vertical walls, once a structure is present in the channel, the problem needs to be described by 3D formulation, which complicates the solution.

A complete nonlinear model for water wave propagation from Deep to Shallow Waters is developed by Galan (2012), and a set of fully nonlinear Boussinessq-type equations (BTEs) with improved linear and nonlinear dispersive performance. The highest order of the derivatives was three in the equations, and they use the minimum number of unknowns, the free surface elevation and the horizontal velocity at a certain depth. The equations allow reduction of the errors both in linear frequency dispersion and shoaling below 0.30% for kd=5, and below 2.2% for kd=10,

Where **k** is the wave number and **d** as the water depth.

The nonlinear performance was also improved for kd=2. A simple fourth-order explicit numerical scheme was derived to test the linear and nonlinear behavior of the model equations against analytical and experimental results.

The effects of external currents on the long shore current and sediment transport in the surf zone are examined by Farhadzadeh (2012), using the cross shore numerical model, which is extended to include the alongshore pressure gradient term in the long shore momentum equation and to allow oblique waves in the swash zone on a beach.

The extended model is compared with five tests conducted in a wave basin with a recirculation system. The cross-shore variations of the long shore current and sediment-transport rate were predicted fairly well for the cases of no and favorable pressure gradients. The cases of adverse and time-varying pressure gradients were computed to extrapolate the experimental results for wider applications.

2.22 BED RESISTANCE ON WAVE PROPAGATION

The near shore radiation stresses are approximated using the monochromatic wave assumption, although this can have significant errors when the incident waves have finite frequency and/or directional spread Goda (1999). Consequently, the resulting wave-driven currents could be predicted erroneously or, when fitting to observations, biased model parameters such as the bottom friction coefficient may result. A moment based reduced spectral wave modeling of frequency and directional spreading effects on wave induced long shore currents were investigated by Zhang et al. (2010), and developed moments-based reduced spectral wave model which includes the leading order effects of wave frequency and directional spreading. This model solves the evolution equations of wave moments, which are integrations of the wave action balance equation multiplied by weighting functions over frequencies and directions. An assumption of analytical Gaussian distribution for the frequency-direction spectrum is made to develop a simple five-parameter system that contains information on the wave height, period, direction, frequency bandwidth, and directional bandwidth. This model is used to investigate the finite bandwidth effects on the wave field and radiation stresses. The directional spreading is found to have a strong impact on the radiation stress, with a larger directional bandwidth resulting in smaller radiation stresses Lee,c. et al (2009). However, the frequency spreading has much less impact. The spectral wave breaking based on the roller concept is considered in this wave model. After coupling with a circulation model based on the shallow water equations Vincent (1985), simulation results compare favorably with field data for waves and long shore currents but showed a strong dependence on the directional bandwidth.

Flow Dynamics and Bed Resistance of Wave Propagation over Bed Ripples was investigated by Galan et al. (2012). The viscous, two-dimensional, free-surface flow induced by the propagation of nonlinear water waves over a rigid rippled bed was simulated numerically. The simulations were based on the numerical solution of the Navier-Stokes equations subject to fully nonlinear free surface boundary conditions and appropriate bottom, inflow, and outflow boundary conditions. The equations were properly transformed so that the computational domain became time-independent. A hybrid scheme was used for the spatial discritisation with finite differences in the stream wise direction and a pseudo spectral approximation with Chebyshev polynomials in the vertical direction. A fractional time-step scheme was used for the temporal discritisation. Over the rippled bed, the wave boundary layer thickness increased significantly, while vortex shedding at the ripple crests generated alternating circulation regions over the ripple trough. The velocity of the Eulerian drift profile was opposite to the direction of wave propagation far above the ripples, whereas close to the bed, its magnitude was influenced by the ripples up to a height of about six times the ripple height above the ripple crest. The amplitude of the wall shear stress on the ripples increased with increasing ripple steepness, whereas the amplitude of the corresponding friction drag force on a ripple was insensitive to this increase. The amplitude of the form drag force attributable to the dynamic pressure increased with increasing ripple steepness; therefore, the percentage of friction in the total drag force decreased with increasing ripple steepness. The period-averaged drag forces on a ripple were very weak, while the influence of form drag increased with increasing ripple steepness Dimas et al. (2010).

Laboratory investigation of low-frequency waves induced by random gravity waves on sloping beaches by Liu et al. (2009) have revealed that for the given water depth, the low-frequency wave energy is proportional to $m_o^{3/2}$, where m_o is the primary wave energy, in agreement with the field observations. They used incident primary waves with a Pierson-Moskowitz (PM) spectrum were mechanically generated at a water depth of 0.7 m and propagated toward beaches with three different slopes 1/20, 1/30, and 1/40 in separate experiments. Four wave conditions with significant wave heights ranging from 0.078 to 0.125 m were studied. The time series of surface elevations were simultaneously recorded at various water depths along the beach. Low-frequency waves were obtained from the data with a low-pass filter. The results showed that the spectra of low-frequency waves on beaches are dramatically changed by the beach slope and the wave height of incident primary waves. The cross-correlation of low-frequency waves with the surface-elevation envelope of the primary waves showed that the incident long waves were dominant. Free low-frequency waves propagate in the offshore direction with very low amplitudes due to high energy dissipation on the beach. The energy ratio between the low-frequency and the primary waves in the shoaling region outside the surf zone, strongly correlates to the local surface skewness.

A general directional spreading function was developed by Lee et al. (2009) that allows for asymmetric directional distributions. For multidirectional random waves that approach the shore obliquely over a planar slope, they demonstrated that directional asymmetry occur due to wave refraction. The asymmetry created by refraction increases with the offshore peak wave direction. The different spreading functions were compared and it was shown to better capture changes in the directional distribution that occur in a refracting, random wave field. The new asymmetric spreading function is compared to a long time series of wave directional spectra measured at a near shore field site. The results demonstrated that refraction-induced asymmetry is common in the near shore and the asymmetric spreading function gives an improved analytic representation of the overall directional distribution as compared to the symmetric function.

Studies with different side slopes indicated that the effect of side slopes on the wave attenuation phenomenon is relatively less as compared to the depth difference from the channel and the adjacent regions.

2.23 FACTORS AFFECTING WAVE ATTENUATION

In order to understand the phenomenon of wave attenuation the effect of the following factors on wave transmission in a dredged channel were analysed by Gole et al. (1966).

2.23.1 Model bed friction

Based on the analysis of the steady, periodic motion of an infinite plate, O'Brien and Chaffin (1942) have developed a method for computing the energy loss for waves in a channel of finite width and depth. It is shown by them that the dissipation of energy per cycle per unit length of channel, per unit width of crest, due to bed friction is

$$N = \frac{\rho}{2} \sqrt{\pi \upsilon T} \; u_b^2 \tag{2.112}$$

Where,

ρ = density of water,

ν = kinematic viscosity of water,

T = wave period and

u_b = maximum velocity of the orbital motion at the bed obtained from potential theory.

$$\text{Now } u_b^2 = \frac{g\, H^2 \operatorname{sech} \frac{2\pi d}{L}}{2L} \tag{2.113}$$

Where,

g = gravitational constant,

H = wave height,

L = wave length and

d= depth of water.

$$\text{Therefore,} \quad N = \frac{\rho}{2} \sqrt{\pi \upsilon T} \; \frac{g\, \pi\, H^2}{2L} \; \operatorname{sech} \frac{2\pi d}{L} \tag{2.114}$$

Now, the rate of decrease of power with respect to the distance of travel of the wave is given by

$$\frac{dP}{dX} = -\frac{N}{T} = -\frac{\rho}{2} \sqrt{\pi \upsilon T} \; \frac{g\, \pi\, H^2}{2L} \; \operatorname{sech} \frac{2\pi d}{L} \tag{2.115}$$

Now,

$$P = \frac{c_{gE}}{L} \tag{2.116}$$

Where c_g is the group velocity and

$$E = \frac{wLH^2}{8} \left(1 - M \frac{H^2}{L^2} \right) \tag{2.117}$$

Where w is specific weight of water and M an energy coefficient.

Now, $$\frac{dH}{dX} = \frac{dP/_{dx}}{dP/_{dH}} \tag{2.118}$$

$$= -\frac{\sqrt{\pi v T}\,\pi H}{nL^{2(\,1-2M\frac{H^2}{L^2})}}\,\text{sech}\,\frac{2\pi d}{L} \tag{2.119}$$

Where $n = \frac{c_g}{c}$ (c being the wave celerity). Separating variables,

$$\frac{dH}{dx}\left(1 - 2M\ \frac{H^2}{L^2}\right) = -\frac{\pi\sqrt{\pi v T}}{nL^2}\,\text{sech}\,\frac{2\pi d}{L}\ dx \tag{2.120}$$

The term involving H^2 can be dropped if the waves are not excessively steep or long. Hence

$$\frac{dH}{H} = -\frac{\pi\sqrt{\pi v T}}{nL^2}\,\text{sech}\,\frac{2\pi d}{L}\ dx \tag{2.121}$$

Integrating,

$$\text{In}\,\frac{H}{H_0} = -\frac{\pi\sqrt{\pi v T}}{nL^2}\,\text{sech}\,\frac{2\pi d}{L}\ x \tag{2.122}$$

Where H_o is the value of H when x=0

The wave attenuation due to bed friction was shown to be about only 6%.

2.23.2 Refraction of wave energy from deep to shallow water

The wave celerity in deeper water is more than that in the shallow water thus, normally wave energy from shallow water cannot enter deeper water unless the angle of approach is greater than a certain critical minimum. Also when a wave approaches a discontinuity in depth, it undergoes a transformation in accordance with Snell's law which states that

$$\frac{sin\alpha_1}{sin\alpha_2} = \frac{c_1}{c_2} \tag{2.123}$$

Where α is the angle between the wave orthogonal and the normal to the discontinuous section, c wave celerity and subscripts 1 and 2 refer to the wave before and after the discontinuity.

When the wave is entering from shallow water into deep water, c_2 is greater than c_1. When the angle α is close to 90° (i.e. the wave approaches at a very small angle to the discontinuity), $\sin \alpha_2$ may become greater than unity, indicating that α_2 becomes a complex quantity. In physical terms, this implies that no wave energy can enter from shallow into deep water, when the wave approaches the channel at an angle close to 90° to the normal.

Thus if the angle of attack of the wave to approach channel is very small, no energy is able to enter the channel from the outside. This is similar to phenomenon of total internal reflection in the theory of light.

2.23.3 Out flow of energy from approach channel

As the wave travels faster in the deeper approach channel, the crest along the side slopes is naturally retarded, therefore taking up a direction such that energy travels out of the channel. This would cause further loss of energy within the approach channel.

2.23.4 Reflection of waves

Reflection of wave energy along the sides of the dredged channel would tend to cause build-up in wave heights at certain points. The precise effect of this reflections in building up and reducing wave heights at particular location is difficult to predict.

2.23.5 Recovery of waves in breaker zone

The recovery of wave heights that is seen as the waves advance further in the approach channel is probably due to the fact that this occurs within the breaker zone. Due to this phenomenon of breaking, it may be assumed that wave energy is radiated uniformly in all directions. Since the wave height in approach channel is smaller than the wave heights in the adjacent shallower contours, the breakers are restricted to the zone outside the approach channel. Consequently there is an influx of energy into the approach channel accounting for the recovery in wave height (Ranasinghe et al. 2011).

2.24 DIRECTIONAL PROPAGATION OF WAVES IN PHYSICAL MODELS

In case of wave tranquility studies for port development using shallow water basins, the 3-D sea state is reproduced as a 2-D process with regular/ random waves propagating from any one direction in the model. These 2-D waves are more amenable to theoretical treatment, hence signal generation for wave board movements is relatively simple and the system is cost effective, Hughes (1993). The direction from which waves are to be generated mainly depends on the port location, field data pertaining to wave which includes wave direction, period and height and berth operative period. Most critical directions are tested to find the feasibility of the layout and the port layout is optimized to increase the berthing comforts and to decrease the cost of the developments.

Wind generated waves are multi directional and random in nature. The wave energy depends on wind speed, duration and fetch. At regions of active generation the peak wave direction is aligned with wind direction and the initial directional spreading is generally symmetric about the peak direction (Kinsman 2002). The wave energy distribution in frequency and direction is given by the directional spectrum S (f, θ) which can be represented as the product of frequency spectrum S(f) and directional spreading function G(f, θ). Analytical models for the shapes of frequency spectrum like Pierson-Moskowitz spectrum, Bretsehneider-Mitsuyasu spectrum, the Joint North Sea Wave Project (JONSWAP) spectrum; Texel Storm- Marsen- Arstoe (TMA) spectrum etc. all these analytical models were developed for deep water conditions The TMA spectrum is unique which was developed as modification to JONSWAP spectrum to include the effects of shoaling on the frequency distribution. But it does not include the effect of refraction. Analytical directional spreading function take the form $Cos^2(\theta - \theta_p)$ (Pierson et al. 1952), $Cos^{2s}[(\theta - \theta_p)/2\ ]$ (Longuet- Huggins et al. 1963) and $Cos^{2s}(\theta - \theta_p)$ (Borgman 1969), where S = directional spreading parameters and θ_p = peak wave direction and is taken to be the same for all frequencies in the direction. Mitsuyasu et al. (1975) suggested the value S based on measured field data. These directional spreading functions impose directional symmetry about peak wave direction (Massel 2005). For many engineering applications measured directional spectra are not available and designers have only estimates of wave height, period and direction (Lee et al. 2009). For these applications analytical models for wave directional spectra can be used to generate representative wave conditions. It is important that these analytic models accurately represent the wave directional spreading in coastal areas, since damage predictions for coastal structures and wave propagation into harbour basins are sensitive to the directional characters of the design wave field as shown by Suh et al. (2002) and Vugts (2005). Also the imposition of directional symmetry will degrade their accuracy in areas where the wave refraction is important. Goda (2010) has showed that the directional spreading function becomes narrower as wave propagates over a planer slope. This directional spreading function assumes directional symmetry. However multi directional random wave spectra will become more directionally asymmetric in shallow water due to difference in refraction of directionally symmetric components. Directional asymmetry is difficult and cumbersome to estimate in the prototype. This process needs installation of directional wave recorders in the near shore regions at several selected locations in the vicinity of the area of interest.

Many experts have developed directional spreading function for multi directional random waves, these allows directional distribution of arbitrary asymmetry (Borgman 1969), (Isaacson 1986), (Tozer 1994). Directional asymmetry can be found in prototype where waves are shadowed by islands or man-made structures or in coastal areas where wave refraction is important. This new directional function will allow more realistic design wave conditions for numerical and laboratory modeling. While testing in laboratory it is important to verify the directional modifications of long crested waves generated from predominant peak directions when propagating towards the study region. This also serves the purpose of validation of analytical methods developed which otherwise very complex in the prototype to measure and validate (Sharman et al. 1986).

Attenuation of random waves by rigid vegetation and comparison of different wave theories (Ling Zhu et al. 2017) have investigated the performance of theoretical wave attenuation models in predicting vegetation induced wave decay. It is mentioned that the existing theoretical models are all based on regular wave theory and cannot describe random waves accurately.

The decision support system for designing integrated coastal berths and entrance channel system is a very complex challenge, considering the stochastic environment and time consuming calculation works Tang et al (2017).A decision support system was evolved for designing integrated coastal berths and entrance channel system by means of simulation and parallel computing. This system provides a 3-D integrated design environment with a broad set of fully integrated functions to evaluate design alternatives and to evolve optimal design.

Some simple physical models were made to simulate the behaviour of buckling type marine fenders Antolloni et al. (2017), these models were used for numerical simulation of the behaviour of the fenders.it is shown that by considering the in-series arrangements of two elementary physical models, the approach can be used to simulate the behaviours of parallel-motion fenders. This indicates the importance of physical models even in the much advanced structural design field as applicable to coastal engineering field studies due to the complex nature of the problem to be studied.

An experimental and numerical study on wave shoaling over a submerged ramp was conducted Srineash et al (2018), shoaling in intermediate and shallow waters is a major process that occurs as the wave travels towards the shore from deep water. Finding shoaling coefficient is important in understanding the nearshore energy distribution and surf-zone dynamics. Shoaling coefficient is normally underestimated using linear wave theory in shallow waters. They have used many wave theories for analysis and compared with physical model, wave flume studies to find the best applicable theory since physical model is next only to the prototype observation even today to understand a natural complex phenomenon.

2.25 SUMMARY OF LITERATURE REVIEW

The sea wave theory is developed over a period of time and many researchers have worked in this direction and contributed for this. The natural sea state is known to be non-stationery random process. It is therefore completely probabilistic in nature and nothing about its characteristics is known with certainty, but only within certain limits of probability. The studies with Numerical models represent the real problem but with some simplifications. Thus, the modeler is forced to make a compromise between the details of the model and the prototype. Several advantages and disadvantages of physical model testing are noticed. The model engineer need to exercise his option based on the factors that need consideration for a study.

An incorrectly designed model always provides wrong predictions, independently of the sophistication of the instrumentation and measuring methods.

The cost of physical modelling is often more than that of numerical modelling, and less than that of major field experiments, but this depends on the exact nature of the problem being studied. Physical modelling has gathered new perspectives due to the development of new sophisticated equipment, allowing the measurement of variables in complex flows, which was previously impossible. New experimental techniques, automated data acquisition and analysis systems, rapid processing and increased data storage capabilities also provide useful information for the validation of numerical models. Physical and numerical model input conditions can be controlled and systematically varied, whereas field studies have no such control. However, many problems in coastal engineering are not amenable to mathematical analysis because of the nonlinear character of the governing equations of motion, lack of information on wave breaking, turbulence or bottom friction, or numerous connected water channels. In these cases it is often necessary to use physical models for predicting prototype behavior or observing results not readily examined in nature. The growing use of numerical models in coastal engineering has not stopped the use of physical models and in some cases they made progress in conjunction with each other. Recent trends have included the concept of "hybrid modelling" where results from a physical model of complex region are used as input or boundary conditions for a comprehensive numerical model covering a wider region of interest. Alternatively, numerical model results may be used to provide input conditions at the boundaries of the physical model. In view of all these complex conditions it is always suggested to finalize the major coastal engineering proposals by sophisticated physical model studies whereas numerical model can be used for initial studies for arriving at few selected alternative conceptual proposals only.

In the present study an experimental investigation of the shallow water region wave transformations and its effects on port development. Due to complex hydrodynamic wave conditions it is very difficult to numerically model this. There are several numerical models predicting different results for a given field condition. The results from this physical model studies can also be utilized for confirmation of results from alternative numerical models. The nearest best numerical model can be identified from this comparison. As seen from the literature review the experts worked in this field are of the opinion that the in coastal engineering studies physical models are next best to prototype observations. No doubt that the quality of mathematical modeling are improving with advancement in computer configurations and capability improving at a rapid pace. But confirmations from the field and physical model data will add confidence to numerical modelling. Thus it is necessary to support modeling techniques with more of field and physical model data. It is expected that the present work will be useful one for the engineers and port planners in understanding arriving at good and economical designs.

CHAPTER 3

DETAILS OF PHYSICAL MODEL TESTING

3.1 MODEL FACILITIES

The Model facilities comprised of 3-D hydraulic rigid bed physical model constructed to 1:100 geometrically similar (G.S.) scale for wave propagation studies along port approach channels. The model is housed in a hangar of size 72m X 45m. (Fig. 3.1), such that the effect of open wind stresses on the water surface can be avoided.

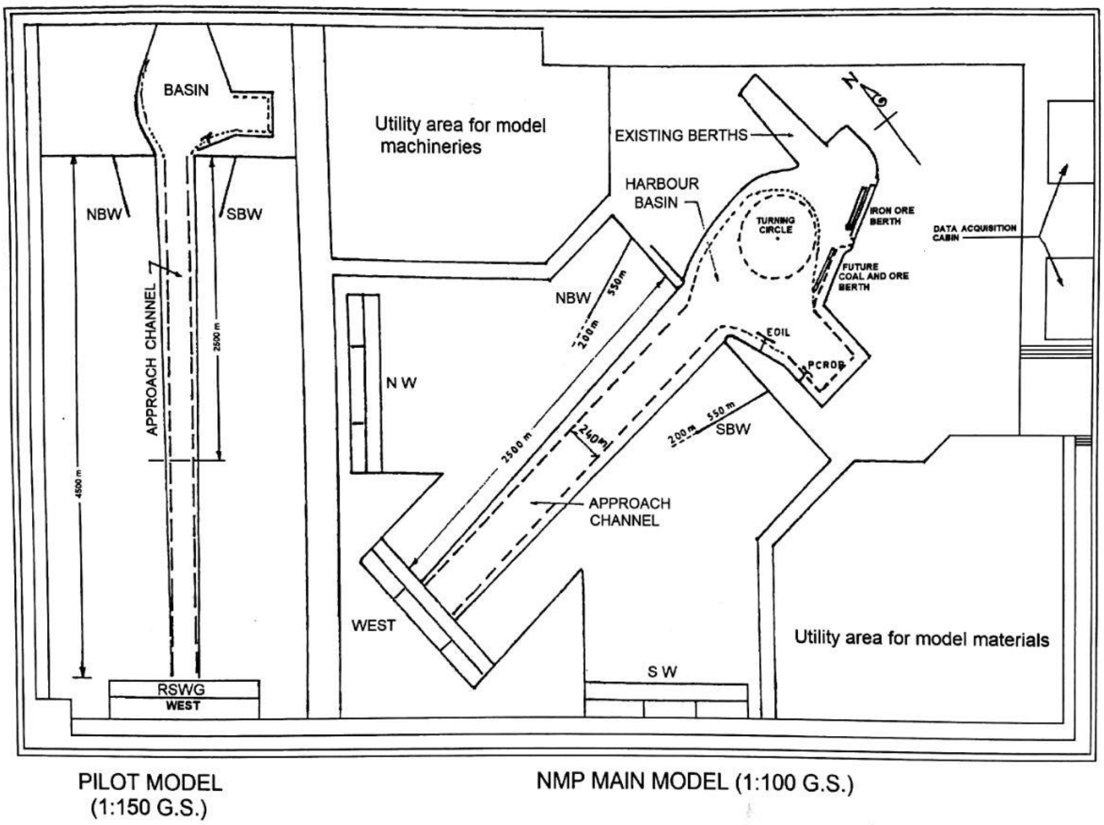


Figure 3.1 Layout plan of hanger showing NMPT (1:100 G.S.) and pilot model (1:150 G.S.)

The covered facilities in the hangar is also necessary for protection of various machineries, sensitive data acquisition equipment, computer system, electronic water level indicators etc and also experiments can be conducted even during rainy seasons without interruption. Sea bed bathymetry is simulated to the model scale all existing port facilities with existing depths are reproduced to scale (Fig 4.9).Model has random wave generation facility from 3 directions namely West, South-West & North-West (Plate 3.1).

During wave attenuation studies adoption of 1:100 G.S. is useful in improving the model study conditions by increasing the widths of approach channel, its side slopes and improved conditions for wave observations along and across the channel. The model scale adopted for directional propagation and wave tranquility studies is 1:120 G.S. The change in scale from 1:100 G.S. to 1:120 G.S. during tranquility studies was useful in simulating larger areas adjacent to port and approach channel to establish the wave field in and around the port area. The model is a truncated wherein only a part of the approach channel is simulated along with the port basin and pertinent structures. This is mainly due to very long approach channel due to mild slope of the sea bed bathymetry. In case if it is required to simulate entire channel, length of model size will increase and becomes very costly for construction and as well as for running the model for studies. Thus a truncated model is very useful in reducing the cost with no compromise on the quality of the study. The model input wave condition at the sea ward boundary of the truncated approach channel is obtained by pilot model and mathematical modeling by calculating the wave conditions from deep water to the model boundary.

Plate 3.1 Overall view of the model.

The model boundaries are simulated with proper wave absorption material like course and fine aggregates so that the undue reflection effects are avoided in the model. The region behind the wave flaps are suitably confined and provided with boulders of suitable sizes to eliminate the reflected waves from these regions which are generated by the backward movement of the wave flaps during the process of wave generation. The harbor basin was suitably simulated with the existing structures and spending beaches are also simulated at the locations as in the prototype for effecting proper absorption of wave energy thus reducing unwanted reflection effects of the waves within the harbor basin. The sea bed contours are almost parallel to the coast line running almost North to South. From analysis of the field data measured at this site west, south – west and north – west are the critical wave direction. The wave approach direction varies seasonally. For all weather port development the wave tranquility testing must be conducted from all the critical direction. The direction of prevailing waves depending on season and waves approach towards port region from any of these locations at a given time. The wave tranquility studies from all three directions will be useful in predicting the wave conditions during different periods of a year depending on seasonal variation.

The model is equipped with computerized wave generation and multi-channel wave data acquisition system (using CWHR) along with necessary software for data analysis. The entire operation of wave generation and data acquisition can be executed from data cabin provided in the model hangar. Electronic water level indicator is provided to monitor water level in the model during the course of experimentation, the least count water level indicator is 0.2 mm. The sea bed counters are marked with color on the model bed. This is to understand the depths at given location on the model and also useful in sketching the wave crests in model. These contours are predominantly visible in the photography and videography as well.

The random sea wave generating system comprises a wave board of (Plate 3.2), servo actuator (Plate 3.3), servo controller, servo valve, hydraulic power pack (Plate 3.4) and personal computer based simulation and data acquisition system.

Plate 3.2 Wave board arrangement with hump.

The wave board is moved by close loop servo hydraulic control system. The servo controller receives the command signal from microprocessor and generates error signal by comparing it with a feedback signal obtained from the displacement transducer (integrated with the actuator).The error signal from the servo amplifier is converted into current suitable for driving a servo valve which in turn controls the oil flow to the actuator and its displacement. The analog command signal acts as a reference input for the system and the actuator moves till the feedback signal becomes equal to the reference input and position feedback is zero.

Plate 3.3 Servo actuator moog system connected to wave board.

The basic working of the wave generator is from the input signal time series of voltage is sent to servo mechanism. At the same time the servo mechanism receives the information on the position of the wave paddle through the displacement sensor (Feedback) .After the comparison of input signal with the peddle position, the servo mechanism sends a control signal to the valve of the hydraulic pump, which converts the output of the hydraulic pump into the movement of the wave paddle (Liu and Figaard 2001). The generation of either regular wave or random wave is dependent on the input time series voltage signal to the system.

Therefore the actuator piston stops until next reference input. This forms the close loop control system. The source of the reference input can be a simple electronic function generator, capable of giving sinusoidal output in the range of frequencies to be generated. For random wave generation, the source is a personal computer where the random wave train synthesized at P.C (Plate 3.5) from the wave spectrum and to be generated in model is stored in voltage form along with software for generation and acquisition of waves. The acquired data is then sent to a P.C. for analysis.

System capabilities: The system is capable of generating frequencies in the range of 0.3 to 3 Hz and maximum wave height of 20cm at a water depth of 50 cm. In the present model setup depth at wave generator is 12.5cms at HWL, a maximum of 5cm wave corresponding to 5m in prototype can be simulated on 1:100(G.S) model and a maximum period of about 33sec in the prototype scale. It has the option for both regular and random wave generation. The various theoretical spectrums like Pierson-Moskowitz (PM), JONSWAP, SCOTT etc or any desired spectra analyzed by field data could be generated.

Plate 3.4 Hydraulic power pack with servo control system.

Plate 3.5 A view of data cabin.

3.2 SYSTEM OPERATION

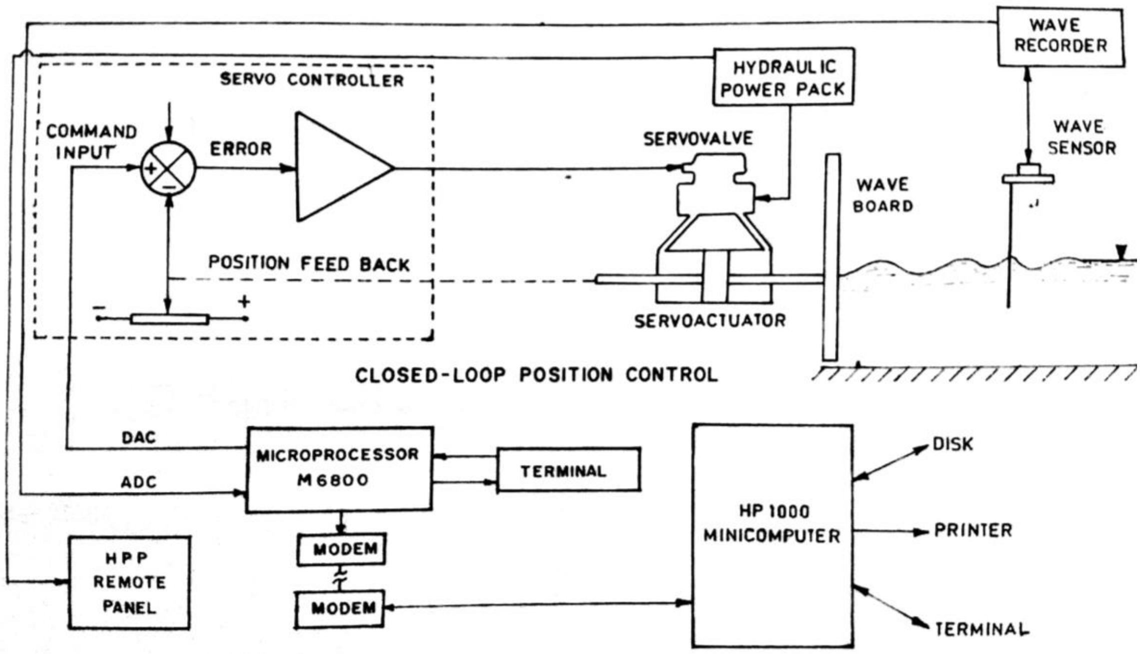


Figure 3.2 Line diagram of setup for the random wave generation (Ref. poonawala et al. 1989)

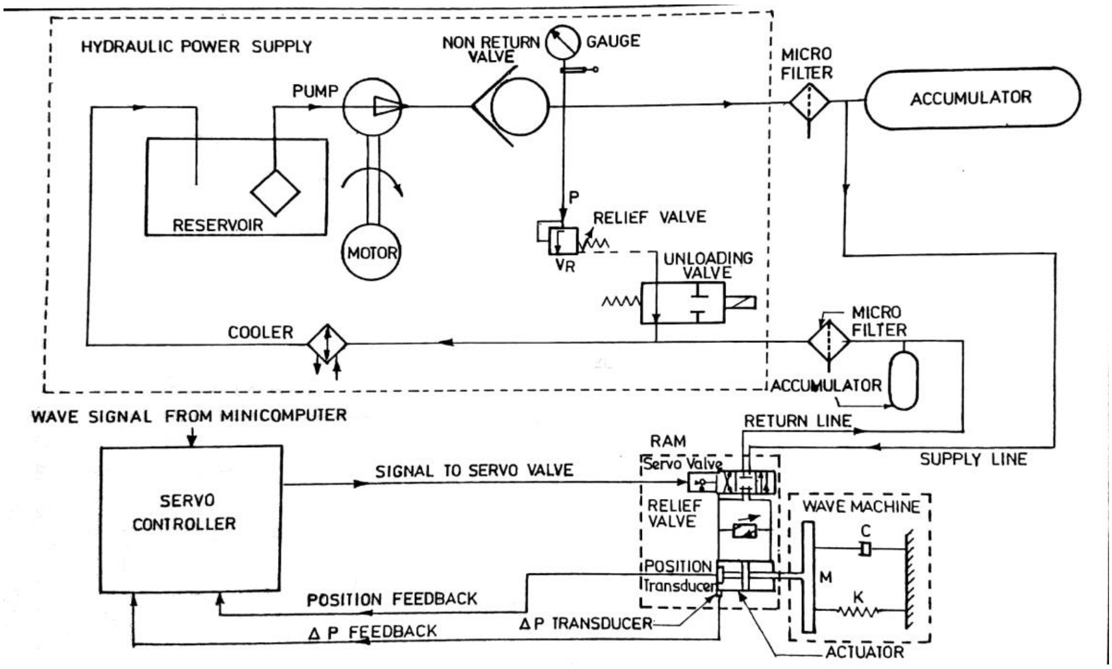


Figure 3.3 Line diagram of servo hydraulic system for random wave generation. (Ref. Poonawala et al. 1989)

In order to confirm the signal input given to the servo system and the output in the form of wave generated the reference standard spectrum will be compared to the spectrum acquired at the wave generated. If the generated and acquired spectra are matching than the wave data can be acquired and analyzed from all the data acquisition channels and will be recorded as a valid result for the model. A typical wave generated and acquire spectra is shown in Fig.3.4. The typical wave spectra acquired in the model is as in Fig.3.5. Practically exact matching and generation of the spectra is very difficult. The fluctuation in the water level in the model and other laboratory effects are the reasons for this. In such cases the normalization of the data acquired will be done to have a standard comparable values of wave tranquility results from experiment to experiment.

The details of wave train based on spectrum are stored in computer. The user at wave generation down the program and wave train into the microprocessor memory using terminal. With the help of interactive program in the microprocessor and with simple commands the operator initiates wave generation by sending the samples of wave train to the servo hydraulic system through digital to analog convertor (DAC) The wave height data is simultaneously acquired by the analog to digital convertor (ADC) of the microprocessor using capacitance type of wave sensors. The acquired data is sent to P.C. for analysis significant wave height, spectrum matching, Rayleigh distribution etc.

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Plate 3.6 Multichannel data acquisition CWHR probes.

Plate 3.7 Wave generation and data acqusition.

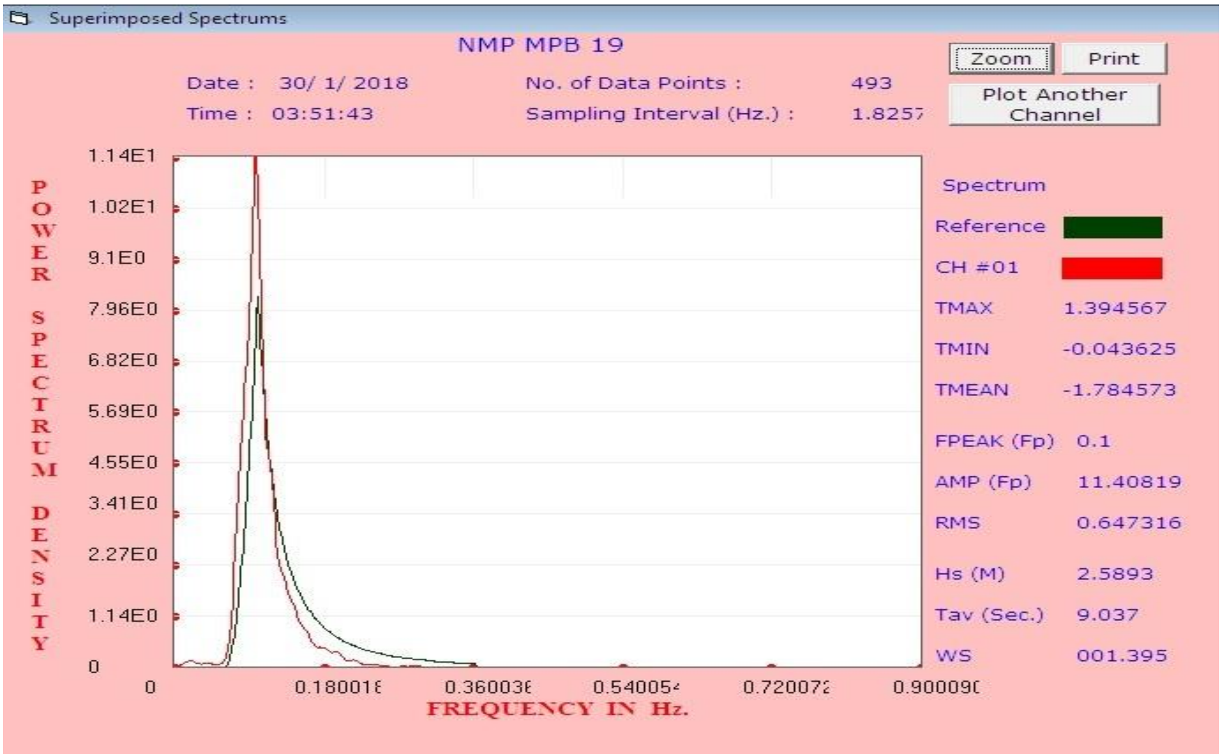


Figure 3.4 Typical wave generated and acquired spectra in the model.

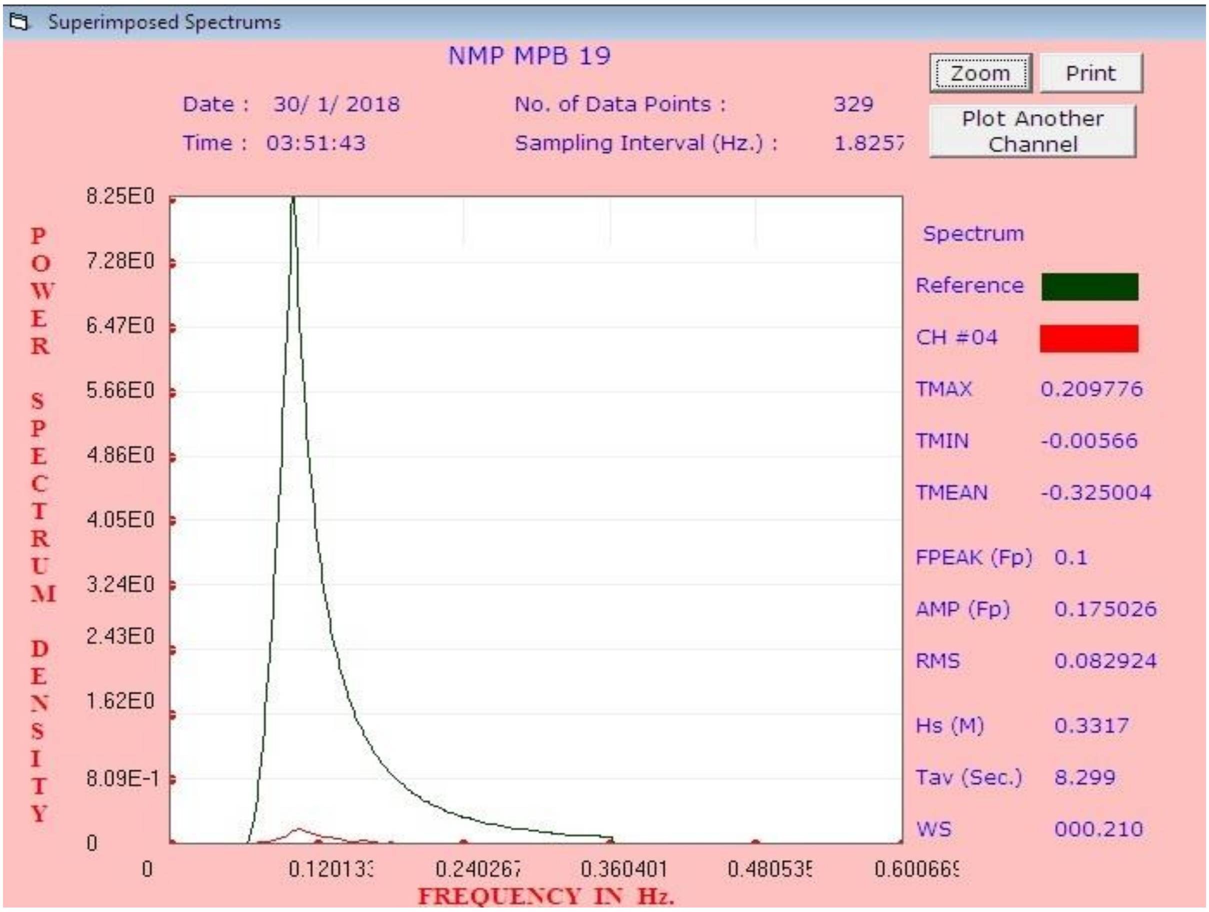


Figure 3.5 Typical wave spectra acquired at the turning circle.

CHAPTER 4

MODEL STUDIES, RESULTS AND DISCUSSIONS

4.1 GENERAL

The experimental results collected for wave attenuation with varied channel slopes were analyzed and are presented in table and graphical forms. The results of the directional spread are presented in table and sketch form. The results of the model studies conducted for wave tranquility of flotilla berths, outer harbour development with breakwaters extension are also presented. The values of wave heights observed in the harbour entrance for studies conducted for the different stages of port development, having varied channel depths and lengths were also collected and presented. These results enable to draw important conclusions on the effect of channel dimensions on the wave tranquility in the harbour basin. Also some important recommendations are made for the port planners based on these long term data of the model based on which the port is developed and successfully operating.

4.2 STUDIES FOR WAVE ATTENUATION

4.2.1 Model studies for different side slopes

The total length of the approach channel simulated is 2250 m from port basin entrance which was extended up to about 9.5m depth contour in the sea as shown in Fig 4.1. In order to estimate the effect of channel side slopes on the wave attenuation for the waves propagating along approach channel the breakwaters are not incorporated. Provision of breakwaters will have diffraction and reflection effects near the reaches of breakwater which affects the wave conditions at the adjacent regions as well. By doing so it is possible to quantify the wave attenuation along the channel. The channel slopes simulated are vertical, 1:5, 1:10 and 1:20. This is to study the effect of the channel slopes on wave attenuation phenomenon. The model scale was kept 1:100(G.S.) for effectively have greater sloping width and depth in the model to study wave propagation along the channel. For each slope experiments are conducted by generating regular and random waves. This is done to find the difference in wave tranquility estimates between regular waves and random waves in the model. For maintaining uniformity and better comparison water level in the model is kept at +1.5m which correspond to high water level at the site.

The experiments are conducted by simulating different side slopes viz., vertical, 1:5, 1:10 and 1:20 in the model (Fig4.1, 4.2). Regular waves of H = 2.25 m, T = 10 sec. and random waves of Scott Spectra with H_s = 2.25m and T_p = 10 sec. are generated during these studies. These are the testing conditions for the berthing operations. This corresponds to 3.66m, 10 sec. in deep waters. Simultaneous wave recording are taken all along the length of the channel and also at the channel surroundings at an interval of 250m at locations as shown in Fig 4.1.

The location of the wave data collection points are selected after carefully observing the directional propagation of the waves on the model, keeping in line with the direction of propagation of wave crests on the adjacent sides of the channel. This is done mainly to optimize the number of wave data points in accordance with the number of wave probes available for the data collection. The locations of the wave probes for data collection are as shown in Fig 4.1. Centre line is along the channel center, two points N1 and N2 are selected, similarly two points S1 and S2 are selected on the South side. Wave data is collected at every 250m interval along these lines.

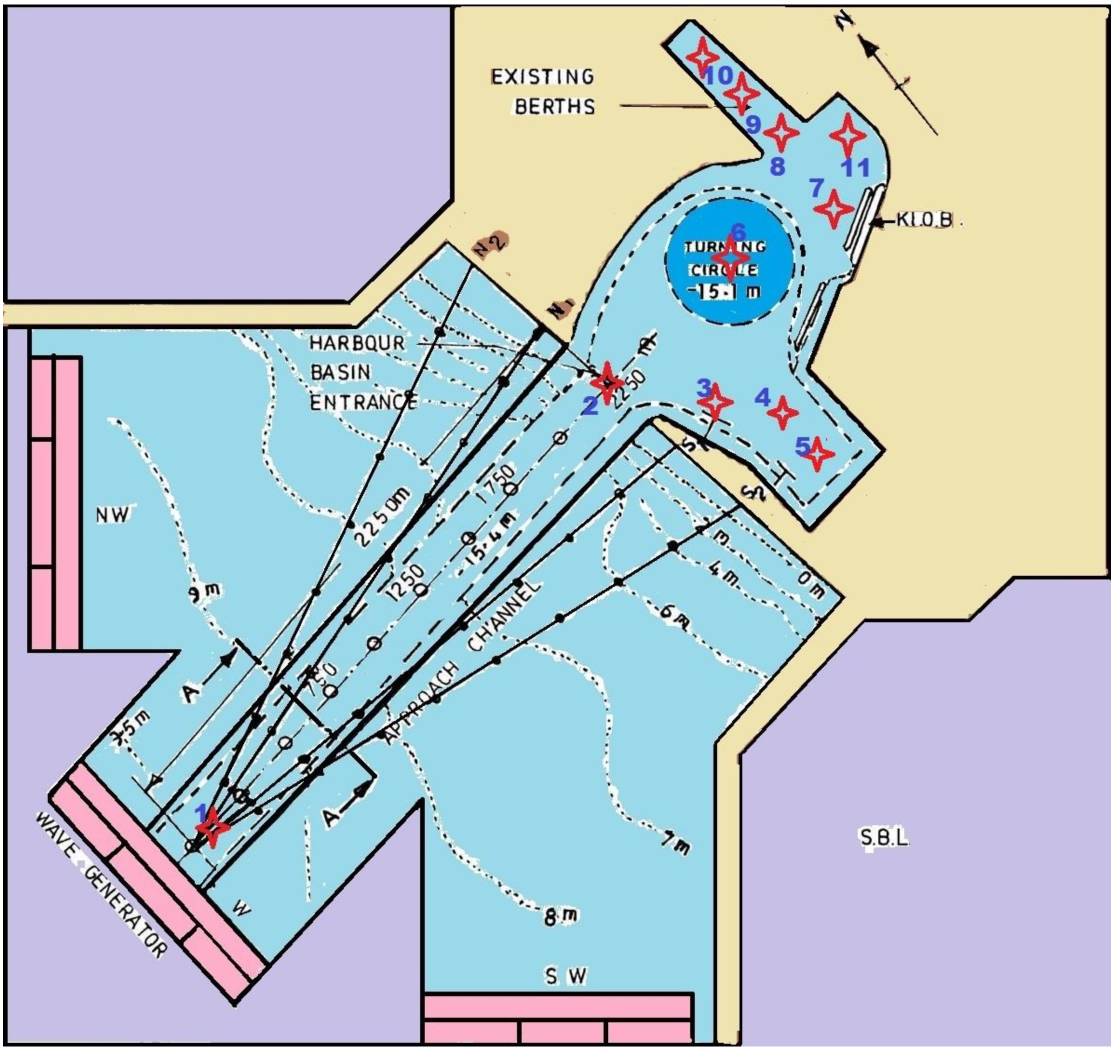


Figure 4.1 Model layout and data point locations

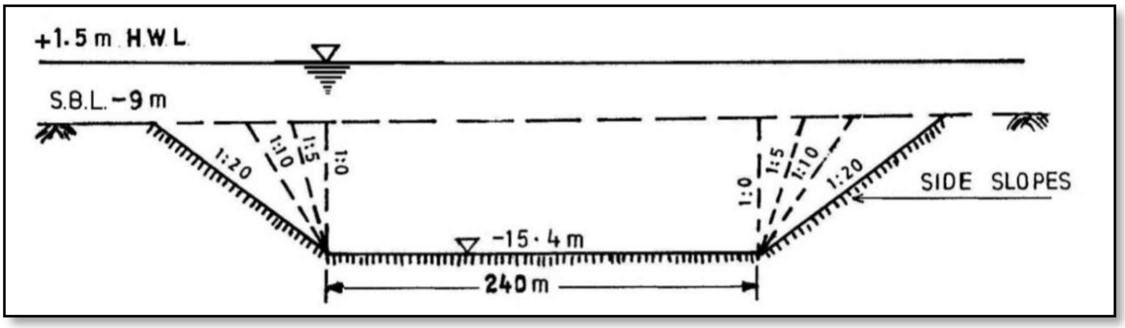


Figure 4.2 C/S @ AA – Channel with different side slopes.

4.2.2 Model results

The results of the wave attenuation along the channel and wave transformation along the adjacent sea bed for different conditions are given in Table 4.1 and 4.2.

Table 4.1 Wave Attenuation Studies with Different Side Slopes of Approach Channel for Regular Waves.

Dist. from generation in meters	Vertical slope	1:5	1:10	1:20
	Waves heights in meters			
0	2.25	2.25	2.25	2.25
250	3.03	2.67	2.27	2.29
500	2.61	2.94	1.75	1.98
750	1.93	1.78	0.97	1.71
1000	1.33	1.58	0.94	1.73
1250	1.03	1.34	1.38	1.38
1500	0.99	1.19	1.22	0.9
1750	0.76	0.94	0.96	0.63
2000	0.7	0.84	0.89	0.65
2250	0.8	0.7	0.67	0.6

For vertical slopes the regular wave of H=2.25 m/ T=10 sec generated at the wave generator suddenly increases to 3.03m after travelling for 250m than reduces to 1.03m at 1250m.Later the rate of change of wave height with respect to distance travelled reduces, gradually it reduces to 0.8 m at 2250m distance from wave generator Table 4.1, Fig.4.3 & fig.4.5. The sudden increase in the wave height may attributed to the reflection of waves from the vertical sides of the channel.

Similarly for 1:5 channel slope increases to 2.67m at 250m, 2.97m at 500m and later gradually reduces from this point to 0.7m at 2250m. The increased sloping width available for the wave front to traverse along may be the main reason for the reduction in wave height in this case as compared to vertical sides of the channel.

For channel slope of 1:10 the wave height gradually reduces from the point of generation as it travels along the channel. Here there is no increase in the wave height noticed for the initial 500m of travel , this may be attributed to lesser reflection effects due to increase in slope width of the channel which effectively allows the waves to refract and pass out of the channel into the adjacent regions. Similar trend in the experiments conducted for 1:20 slopes were observed Here due availability of additional sloping widths the reduction in the wave heights are marginally more.

The sudden rise in wave heights in the initial regions may be attributed to partially developed sea wave state at the wave generator which gradually transforms to fully developed sea wave state as the waves travels away from generation. Therefore the reduction in wave heights is gradual after a certain distance of wave travel from the generator.

The results of the random wave studies conducted for all side slopes of the channel the increase in wave height in the initial reaches of the channel length is only marginal. It increases to 2.5m for vertical, 2.45m for 1:5 slopes, 2.40 for 1:10 slope and 2.35m for 1:20 slopes,(Table 4.2,Fig 4.3).Thus the rise in wave heights is lesser as compared to regular wave studies. This may be attributed to the development fully developed sea state of waves within a short reach. This is mainly due to generation of waves of different heights and frequencies in random wave testing as compared to single wave height and period in regular wave generation.

Thus from the above model result, it is in general we observations that after the waves are generated, the wave height increased rapidly for about initial 500m length of the channel and later it reduced gradually all along the channel length (Fig 4.3). The increase in the wave heights at the generation is more for vertical and 1:5 side slope condition. This may be attributed to higher reflection from the side slopes of the channel as compared to 1:10 and 1:20 wherein the reflection effects are lesser. It is found that even with vertical edges of the channel, there is considerable wave attenuation along the length of the channel and the wave disturbance at port entrance is only 0.8 m for a wave of 2.25m / 10 s at the seaward boundary at a depth of about -9.5m bed contours towards the end of the channel. Studies with side slopes of 1:5, 1:10 and 1:20 indicated a wave disturbance of 0.7m, 0.65m and 0.6m respectively at the entrance of the port basin.

Table 4.2 Wave Attenuation Studies with Different Side Slopes of Approach Channel for Random Waves.

Dist. from generation in meters	Vertical slope	1:5	1:10	1:20
	Waves heights in meters			
0	2.25	2.25	2.25	2.25
250	2.50	2.45	2.40	2.35
500	2.20	2.15	2.00	2.00
750	1.70	1.55	1.35	1.30
1000	1.45	1.40	1.35	1.35
1250	1.10	0.95	0.95	0.90
1500	0.95	0.85	0.80	0.80
1750	0.85	0.70	0.70	0.65
2000	0.75	0.65	0.60	0.55
2250	0.60	0.60	0.55	0.50

The wave heights along the channel length increased initially prominently for channel with the vertical sides and 1:5 side slopes for a small stretch and then gradually reduced towards the port entrance. Over all, it is observed that wave attenuation process is more systematic from 1250 m to 2250 m i.e. the last 1000m stretch of the approach channel for all the cases studied. The studies carried out with random waves also showed similar trends in respect of wave attenuation (Fig 4.4). During the experiments with random waves the difference in wave heights at different locations of wave observations are comparatively less. This may be attributed to the fully developed sea state of waves in case of random wave generation as compared partially developed sea state with regular waves. The wave heights measured along the channel for different slopes clearly show a gradual declination for random waves as compared to regular waves (Fig 4.5, 4.6). In case of random waves since it comprises of waves of different frequencies and heights the chances of abrupt reflection effects within the channel are less. Thus adoption of random waves in wave tranquility studies for port development is advantageous. In case of regular wave studies there are chances of high variation of wave heights due to a single frequency wave. If it is to be studied by regular waves only then it is always advisable to study for different frequencies and average out for better results.

Comparison of wave heights in the channel and lateral regions indicated that with the reduction in wave height in channel, there is increase in wave height along surrounding areas indicating the transfer of wave energy from channel to adjacent outer regions Fig. 4.7 and 4.8, shows that at any point in the channel for the reduction in wave height corresponding increase in wave height just along its adjacent regions along the shallow bed regions.

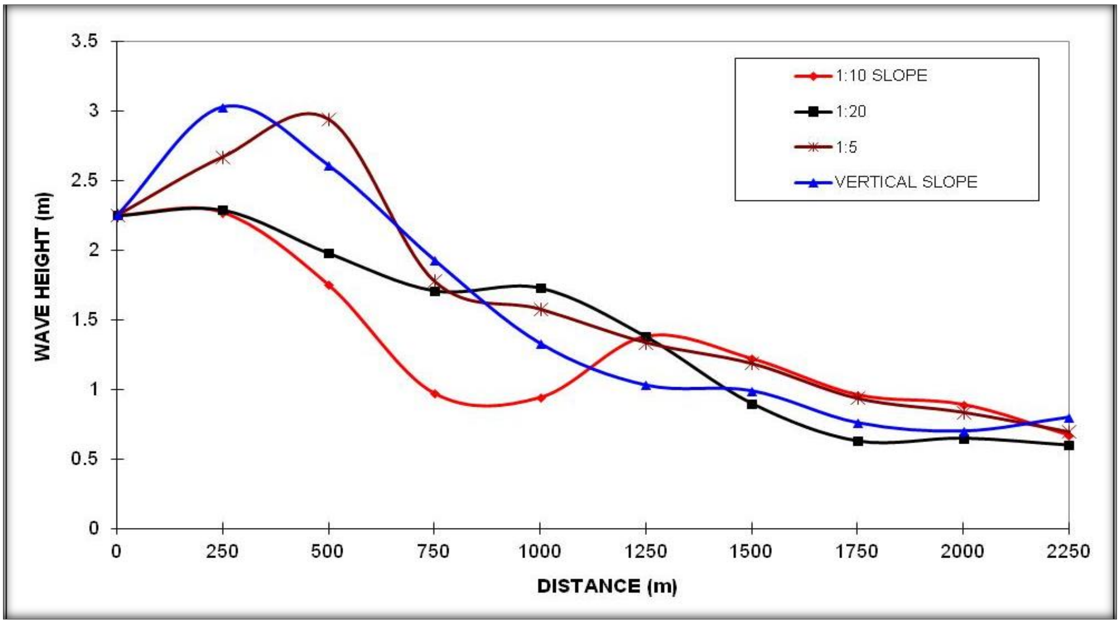


Figure 4.3 Wave Heights for different side slopes (regular waves).

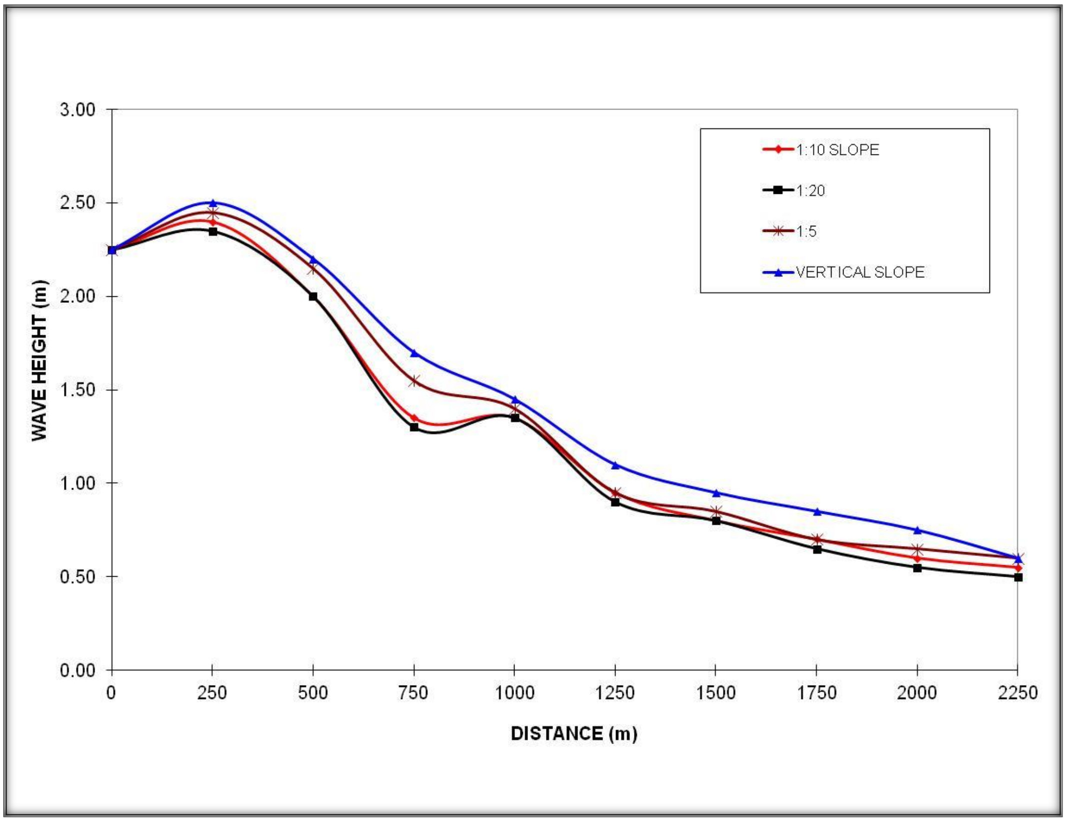
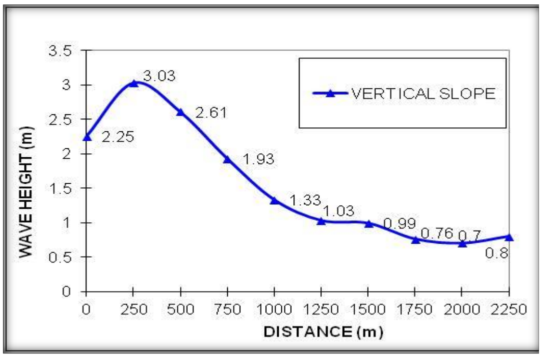
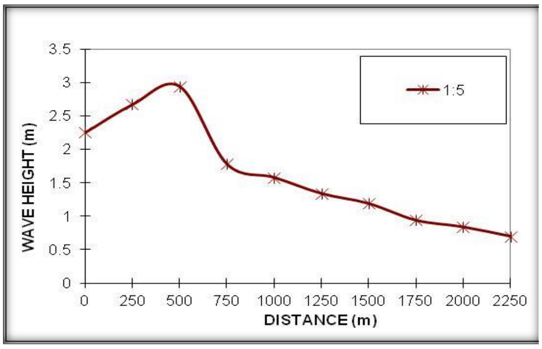


Figure 4.4 Wave Heights for different side slopes (random waves).





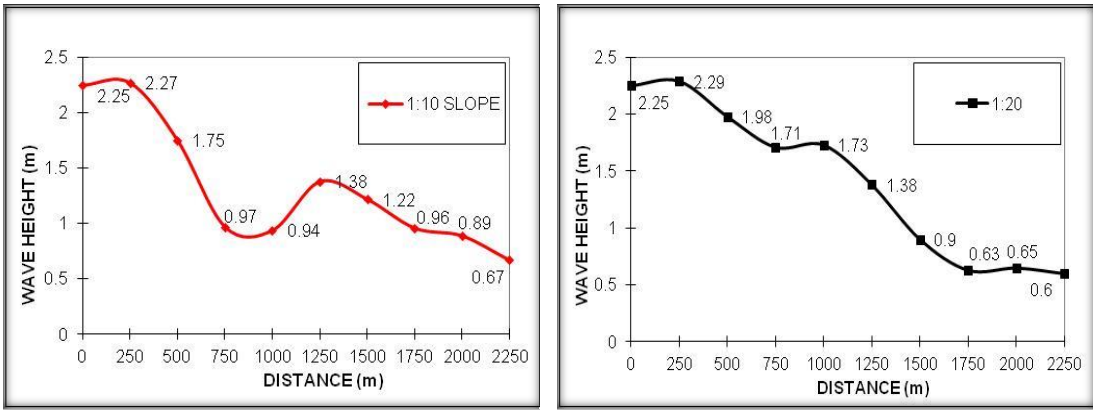
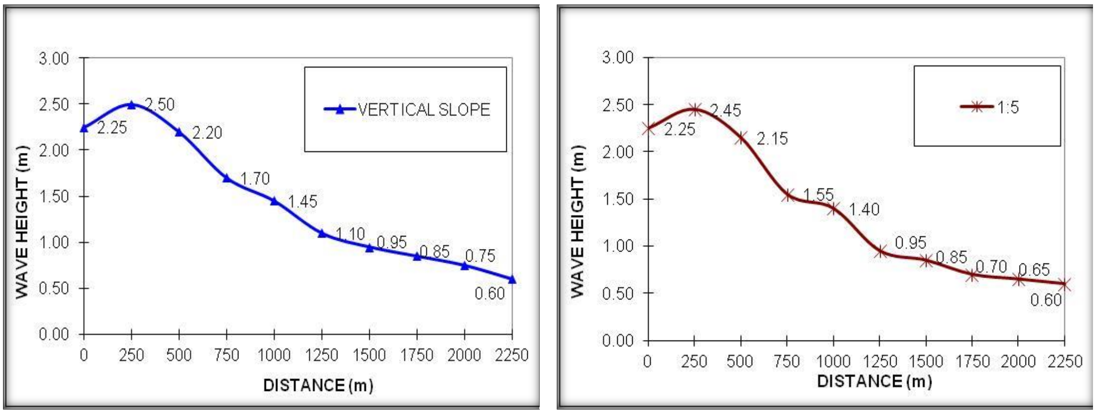


Figure 4.5: Wave height for different side slopes (regular waves)



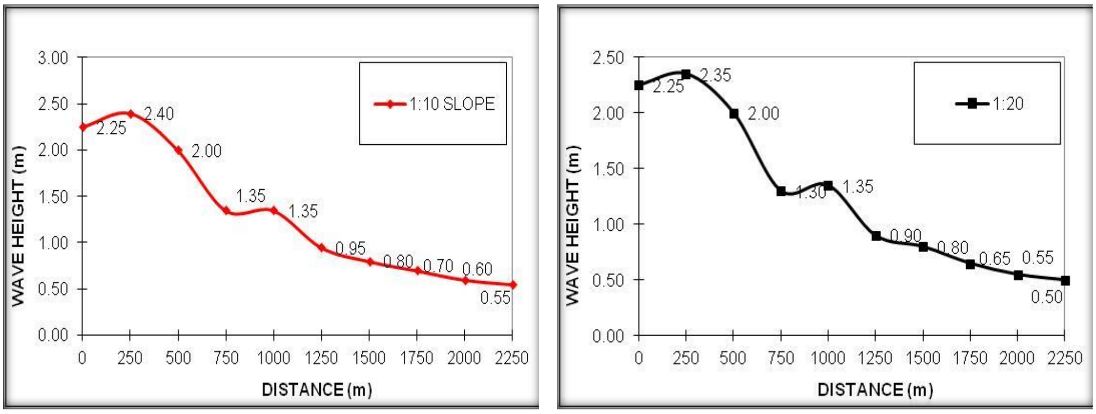


Figure 4.6: Wave height for different side slopes (random waves)

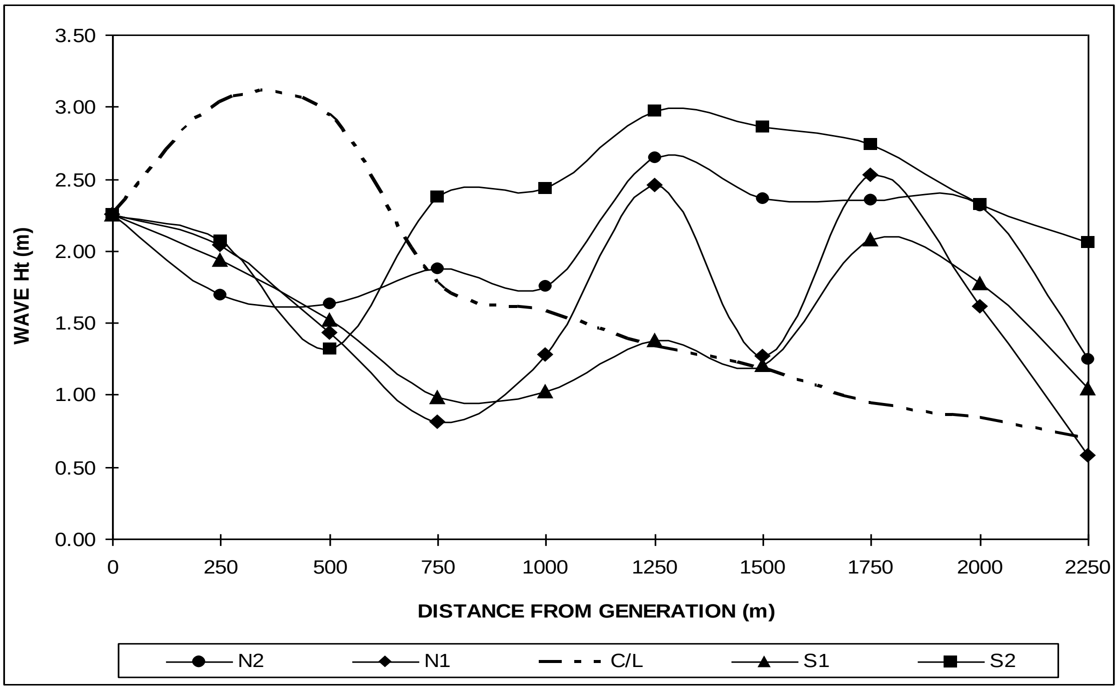


Figure 4.7 Wave Heights along lateral direction with side slopes 1:5 (regular waves).

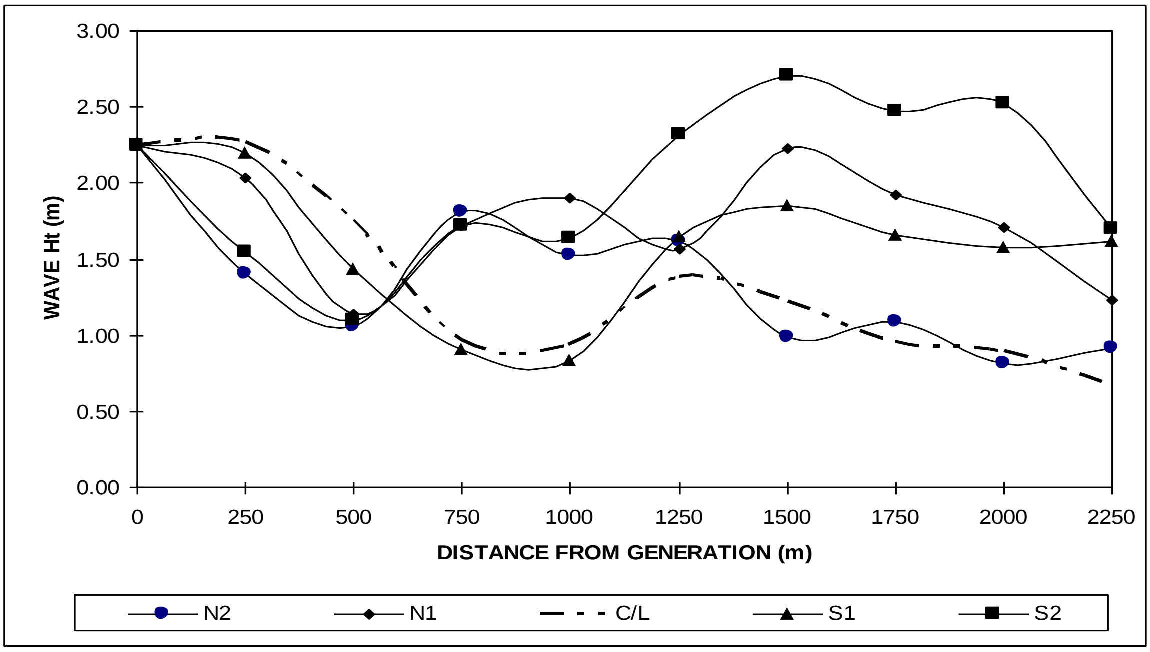


Figure 4.8 Wave Heights along lateral direction with side slopes 1:10 (regular waves)

4.3 MODEL STUDIES FOR DIRECTIONAL PROPAGATION

To study the directional changes of the waves on a 3-D shallow basin model by generating 2-D waves in the model it is desirable to simulate more area for waves to travel and transform. For this reason the model scale was 1:120(G.S.).The approach channel is simulated to a length of about 4000m from the harbour basin entrance, up to sea bed contours of -11.00 m in the adjacent natural depths.

The lengths of each wave flaps are 11m and care is taken to place the wave generators sufficiently far so the wave generated properly attains its shape by the time it reaches the points of interest in the model. The placement of the wave boards are made at the model boundary after suitably considering the wave refraction effects for the direction under study. The wave generated at the model boundary are arrived after studies on a pilot model to scale 1:150(G.S.) by simulating the entire length of the approach channel and mathematical model studies. By pilot model studies (CWPRS Technical report No.3086, 1993) and mathematical model studies (CWPRS Technical report No.3568, 1996) for heights of H= 3.66m tp=10 sec near the deep water conditions(design wave height for wave tranquility studies) are reduced to about 2.5m near the model boundary, this may be attributed to wave transformation from deeper contours to shallower regions undergoes wave attenuation thus losing wave energy by bottom friction, refraction along the side slopes of the approach channel and reflection effects within the approach channel along its side slopes.

By looking to the location of the harbour and the sea bed contours in this region by drawing refraction diagrams for waves in this region, the waves approaching from the western direction are very critical and the wave crests also are parallel to the sea bed contours thus nullifying refraction effects from this direction. Also for the waves from South-West and North-West directions tend to lean towards west by the time they propagate near the harbour breakwater tips. Thus western direction waves are most critical for wave tranquility studies.

4.3.1 Model studies from different directions

The model studies are conducted by keeping the water level at HWL (+1.5m) in the model. Random waves with Hs=2.5m and Tp=10sec are generated. Waves are generated from each of these three directions viz., West, South-West & North-West and the wave patterns are sketched from the generation point till the harbour entrance and along the breakwaters and within the harbour basin as well. Wave crest angles are measured at selected locations in the model with respect to the North direction as whole circle bearing measurement. The changes in the wave crest direction along the north and south of the approach channel are measured. Model photos and videography was useful in finalizing the wave directional changes in the sketches of wave crest at different locations.

4.3.2 Model results of directional spreading

The sketches showing the wave directional changes and wave crest pattern for West, South-West and North-West directions are as shown in the Fig.4.9, 4.10 and 4.11 respectively. Wave direction during propagation on the model is sketched on the model layout having the sea bed contours incorporated on it. After sketching the wave pattern on the layout it is verified with the help of model photos taken during running conditions. The wave directions are later measured by drawing North line at selected locations.

The Change in the wave approach angles along the direction of propagation are measured as whole circle bearing with reference to North direction. These measurements are tabulated in the Tables 4.3, 4.4 and 4.5 for West, South-West and North-West directions respectively.

It is also observed from the records of wave crest patterns that the long crested waves at generation in the model slowly disintegrate into short crested ones while travelling towards the harbour entrance. This can be attributed to complex bathymetric conditions over which the waves propagate. A close observation into the values of rate of changes in the wave angles shows that the rate of change depends on the magnitude of the angle difference between the wave crest and the sea bed contours at that location. Thus it can be seen that the rate of change in the crest angle is minimum for the waves generated from west direction as compared the waves from South-west and North-west directions [Ref. Plate 4.1, 4.2, 4.3, 4.4 & 4.5]

An abrupt change in the wave direction is observed while the wave front crosses the approach channel region in the model. This phenomenon can be attributed to the sudden increased depth in this region which causes abrupt changes in the wave celerity in this zone. The local refraction due to the provision of side slopes of the channel, partial reflection within the channel also contribute to these changes.

Plate 4.1 A model view in still condition

Plate 4.2 Wave propagation from west

The minimum deviation of the wave crest while propagating towards harbour entrance may be attributed to refraction of the waves being lesser due to the minimum angle between the wave crest and the seabed contours in this direction of wave propagation. Thus it leads to minimum changes to wave directional propagation, for this reason the distance of the wave generator to the study area needs to be kept larger so that the long crested wave generated on the model will modify to short crested waves as in the natural sea conditions.

Plate 4.3 A general view of running model

Plate 4.4 Wave pattern near harbour entrance.

Plate 4.5 Wave crest pattern near tip of breakwater

The waves generated from South-West and North-West directions change in their directional propagation with in a very short distance since the angle of wave crest generated and the bottom sea bed contours are more for these directions thus the wave pattern achieving the natural sea state condition in a shorter reach as compared to the waves from west direction.

Thus it is evident that whenever a physical model is to be constructed, the angle between the sea bed contour and wave direction to be simulated needs to be examined very carefully before deciding the distance from the wave generator to the point of observation like harbour entrance etc., more the angle lesser will be the length of wave propagation to achieve the natural sea state condition at the regions of interest and vice-versa. This can be one of the conditions that need to be considered for selection of scale for shallow water wave models for studying wave propagation to harbour basins. Apart from this care must be exercised see the manmade structure in the direction of wave propagation in the model like approach channel, sudden bathymetric variations due to the presence of obstacles like island, local deep pockets etc. The Changes near the tips of breakwaters is one more very important factor that causes the sudden changes in a short distance [Ref. Plate 4.5].

Table 4.3 Waves from West direction

Towards South side of approach channel		**Towards North side of approach channel**	
Distance (in m)	**Angle (in deg.)**	**Distance (in m)**	**Angle (in deg.)**
600	270	600	270
1650	258	1500	248
2700	265	2550	245.
3300	266	3180	254
4200	269	4500	240
4350	286	4050	231

Table 4.4 Waves from South West direction

Towards South side of approach channel		Towards North side of approach channel	
Distance (in m)	**Angle (in deg.)**	**Distance (in m)**	**Angle (in deg.)**
360	220	360	220
1200	230	1800	215
2100	240	3300	225
2400	245	4200	210
3150	255	4500	230
4200	280	5100	225
--	--	4500	253

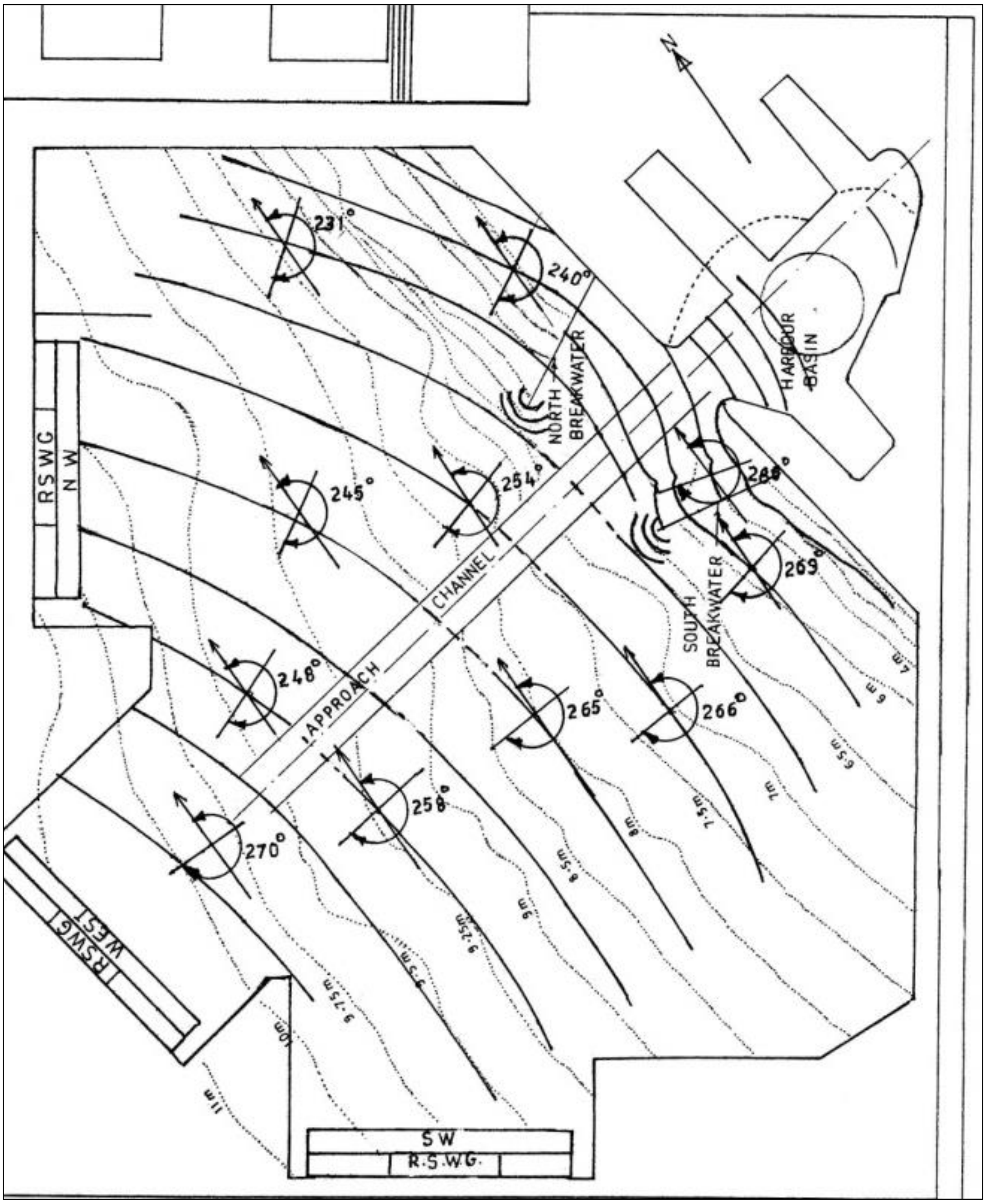


Figure 4.9 Wave crest pattern for westerly waves.

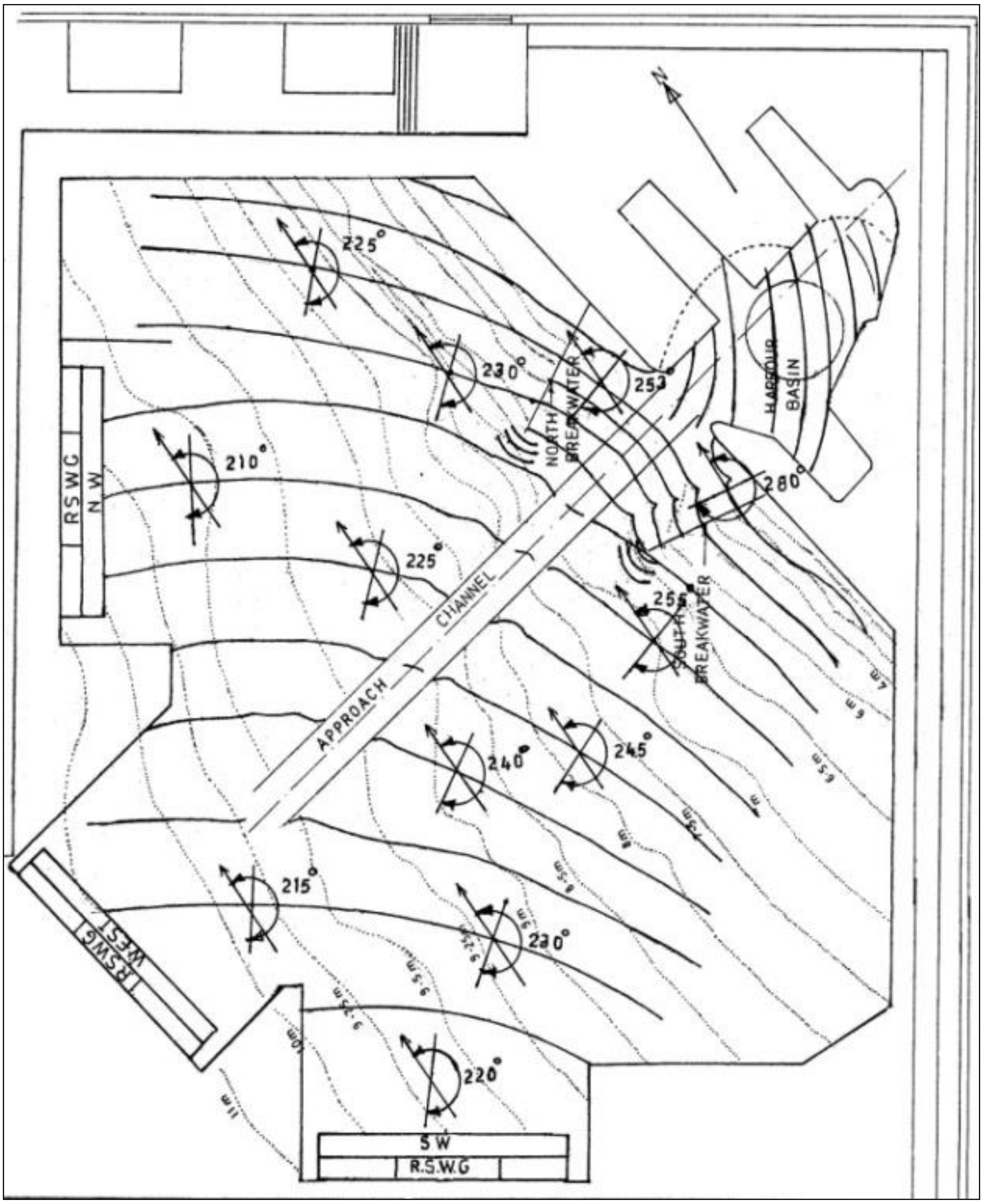


Figure 4.10 wave crest pattern for S-W waves

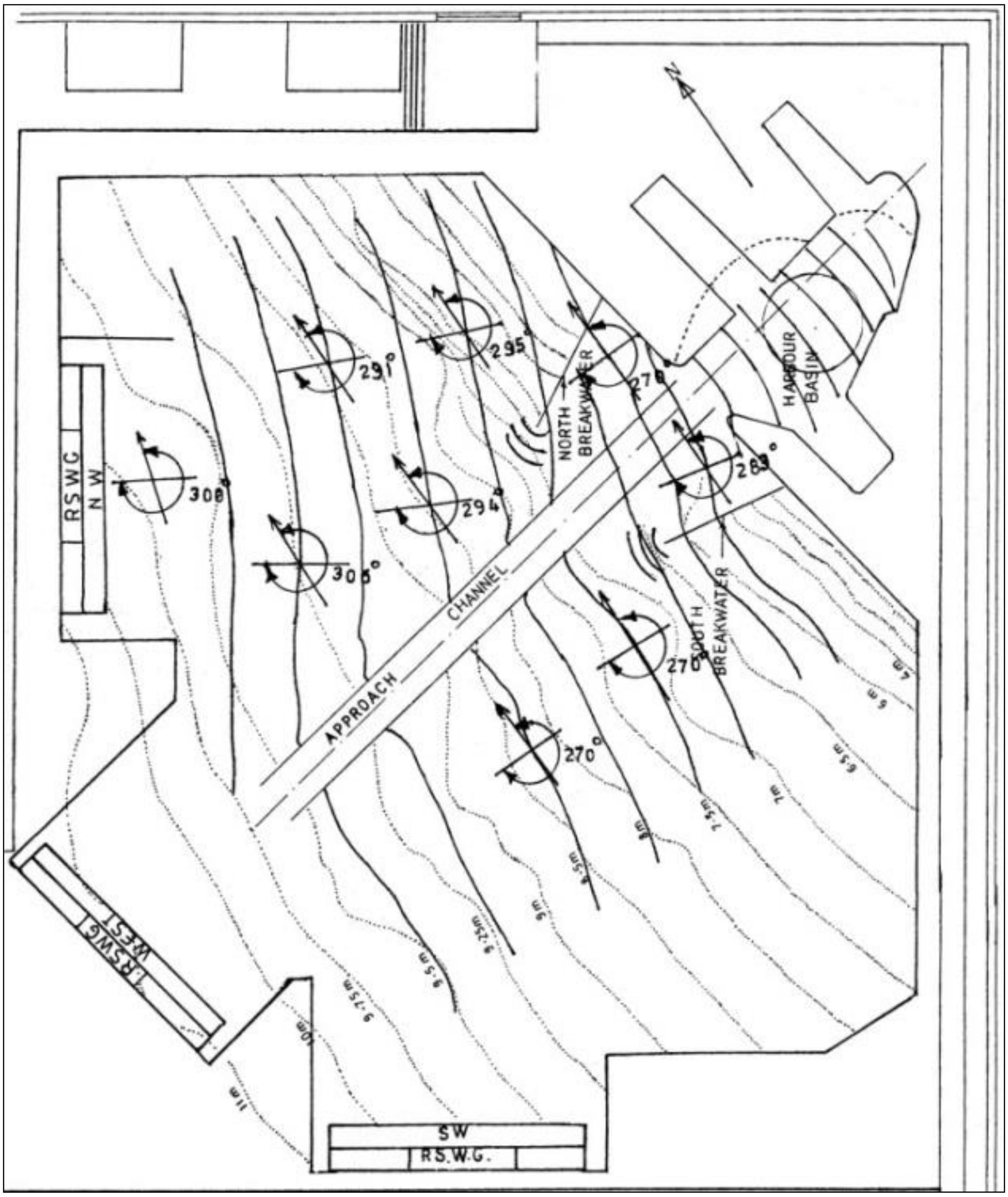


Figure 4.11 wave crest pattern for S-W waves

Table 4.5 Waves from North West direction

Towards South side of approach channel		Towards North side of approach Channel	
Distance (in m)	**Angle (in deg.)**	**Distance (in m)**	**Angle (in deg.)**
300	300	300	300
1200	306	1350	291
2700	270	1950	294
3300	270	2250	295
3600	283	3000	270

4.4 STUDIES FOR OUTER HARBOUR DEVELOPMENT

Studies are conducted for the development of outer harbour by extending the breakwaters from existing 770 m to 1870m each. Two alternatives are tested first by keeping the breakwaters tips straight and second by curved tips as shown in the Fig.4.12 & 4.14 and Plate 4.6 and 4.7. This proposal was to develop a LNG berth and a bulk cargo berth in the outer region of the existing lagoon port enclosed by the proposed breakwaters. The LNG berth is proposed at the extended tip of the southern breakwater. This is mainly to cater to the safety limits of LNG berths. The deep draft bulk cargo berths are to cater to the need of increased ship sizes to sustain competition in the shipping sector as per Sagarmala objectives.The results of these studies are useful in understanding the influence of breakwaters on the wave tranquility inside the harbour basin. Wave tranquility results due to this development at various salient locations are as shown in Table-4.6 in the outer harbour and its effect on the inner harbour was also studied in the model. The wave tranquility at different locations are tabulated in Tables 4.7 & 4.8. Fig.4.13 & Fig.4.15 shows the wave tranquility at various salient locations in the approach channel, harbour entrance and different regions in the basin for waves approaching from West, South-West, and North-West directions for straight and curved extensions of breakwaters respectively. The vertical lines at any location indicates the difference wave height for different wave approach directions in the model studies.

These studies show the wave disturbance in the outer region is higher relatively and this may be attributed to reduced wave attenuation along the port approach channel caused due to the extension of the breakwaters there by reduction of the wave fonts moving out of the channel along the channel side slopes particularly in the shallow water regions. Another important aspect is about losing the spending beaches which are naturally formed on the lee side of the roots of the two breakwaters. These spending beaches are very good wave absorbers; they absorb the wave energy passing out from the channel slopes within the breakwaters. This also contributes substantially in improving wave tranquility in harbour basin. The studies conducted with spending beach for few developments inside the harbour have clearly indicated the positive effects of spending beaches in a harbour basin. Even if the bottom material comprise of soft viscous material the fluctuating pressure on bottom can set the bottom in motion, viscous stresses in soft bottom will dissipate energy content of the waves this is effective in maintaining good wave tranquility. Any absorptive surface in the basin like provision of mild slopes for wave run-up underneath a berthing deck also contributes for achieving better tranquil conditions in the basin. (CWPRS Technical report No.5481).

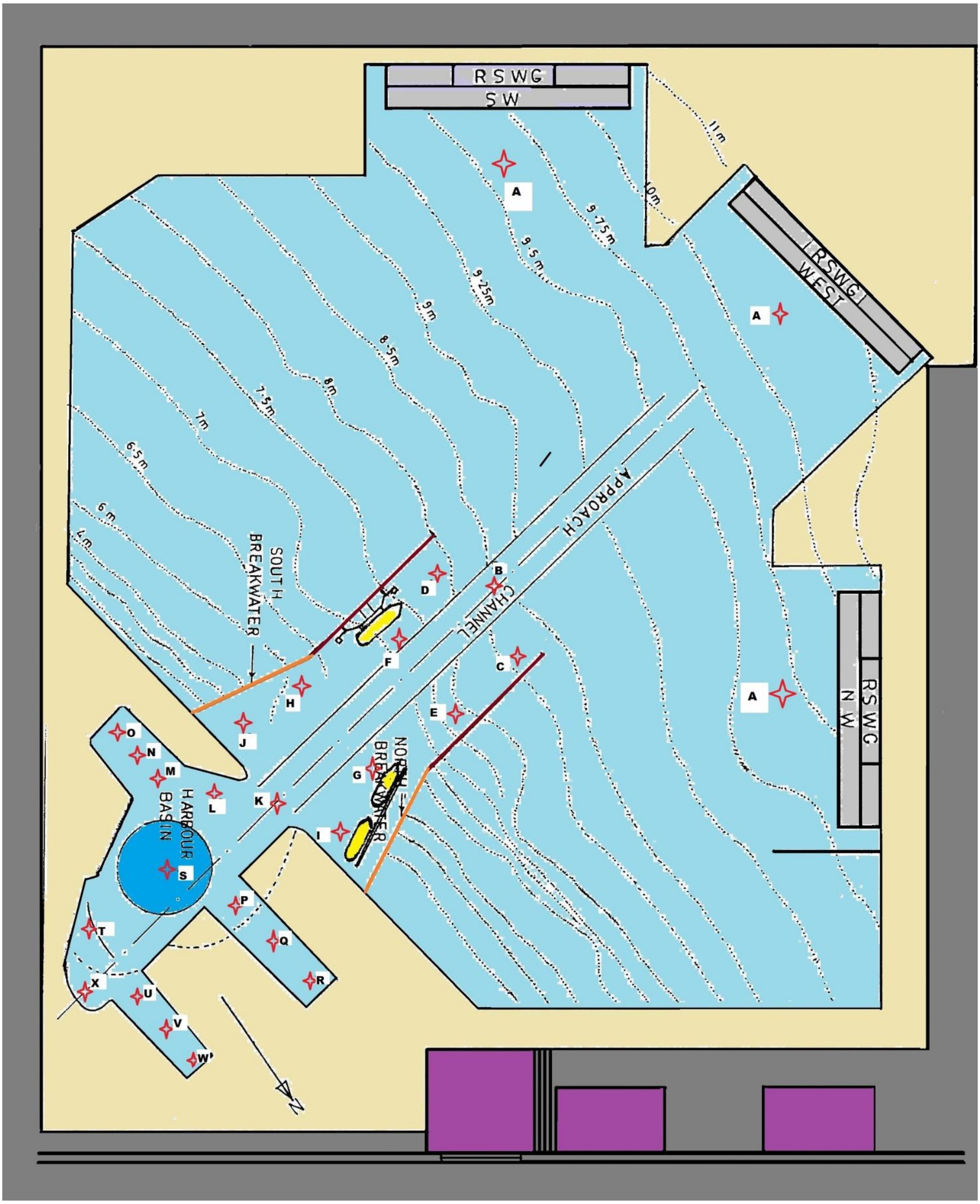


Figure 4.12 Location of Wave data acquisition points for breakwater extension with straight tips

Plate 4.6 A view of model with extended straight breakwater

Table no 4.6 Details of wave observation points

LOCATION NO	LOCATION
A	Wave generation point
B	In the approach channel at the beginning of break water
C	Leeward side of north breakwater
D	Leeward side of south breakwater
E	Lee side of extended North breakwater
F	Proposed LNG berth at leeward side of south breakwater
G	Proposed bulk cargo at leeward side of north breakwater
H	Leeward side of existing breakwater
I	Proposed bulk cargo at leeward side of north breakwater
J	At the end of south breakwater(leeward side)
K	In the entrance of lagoon area
L	Proposed LPG berth
M	Entrance of southern dock arm
N	Middle of southern dock arm
O	Rear end of southern dock arm
P	Entrance of western dock arm
Q	Middle of western dock arm
R	Rear end of western dock arm
S	In the turning circle
T	Kudremukh iron ore berth
U	Entrance of eastern dock arm
V	Middle of eastern dock arm
W	Rear end of eastern dock arm
X	Spending beach on the eastern side

TABLE- 4.7 Wave tranquility in meters for waves from different directions to breakwater extention with straight tips

Sr.No	LOCATION	WAVE HEIGHT IN METERS		
		SW	W	NW
1	A	2.50	2.50	2.50
2	B	1.90	1.80	1.60
3	C	1.75	1.70	1.40
4	D	1.75	1.70	1.75
5	E	1.60	1.50	1.20
6	F	1.40	1.50	1.50
7	G	0.55	0.50	0.45
8	H	0.45	0.50	0.65
9	I	0.75	0.70	0.55
10	J	0.60	0.70	0.75
11	K	0.70	0.65	0.85
12	L	0.60	0.55	0.70
13	M	0.35	0.30	0.40
14	N	0.30	0.25	0.30
15	O	0.20	0.20	0.20
16	P	0.55	0.50	0.60
17	Q	0.45	0.40	0.45
18	R	0.30	0.20	0.25
19	S	0.75	0.65	0.75
20	T	0.55	0.50	0.50
21	U	0.30	0.25	0.25
22	V	0.20	0.15	0.15
23	W	0.10	0.10	0.10
24	X	0.45	0.40	0.45

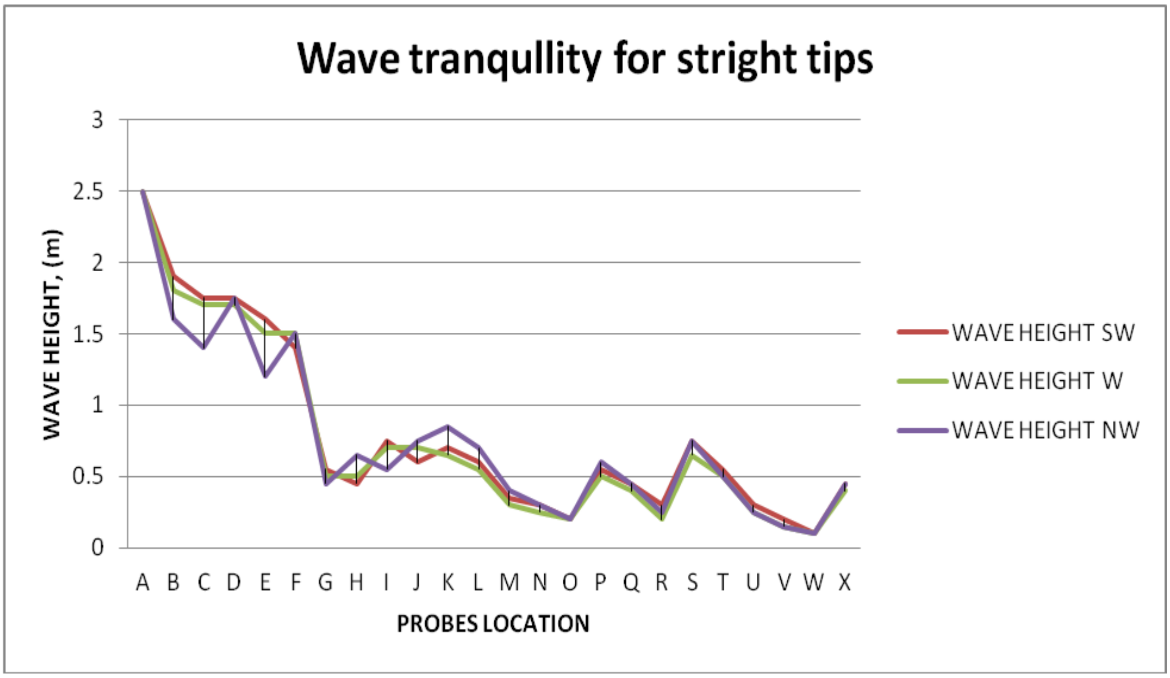


Figure 4.13 Variation of wave heights at salient locations for extention of breakwater with straight tips

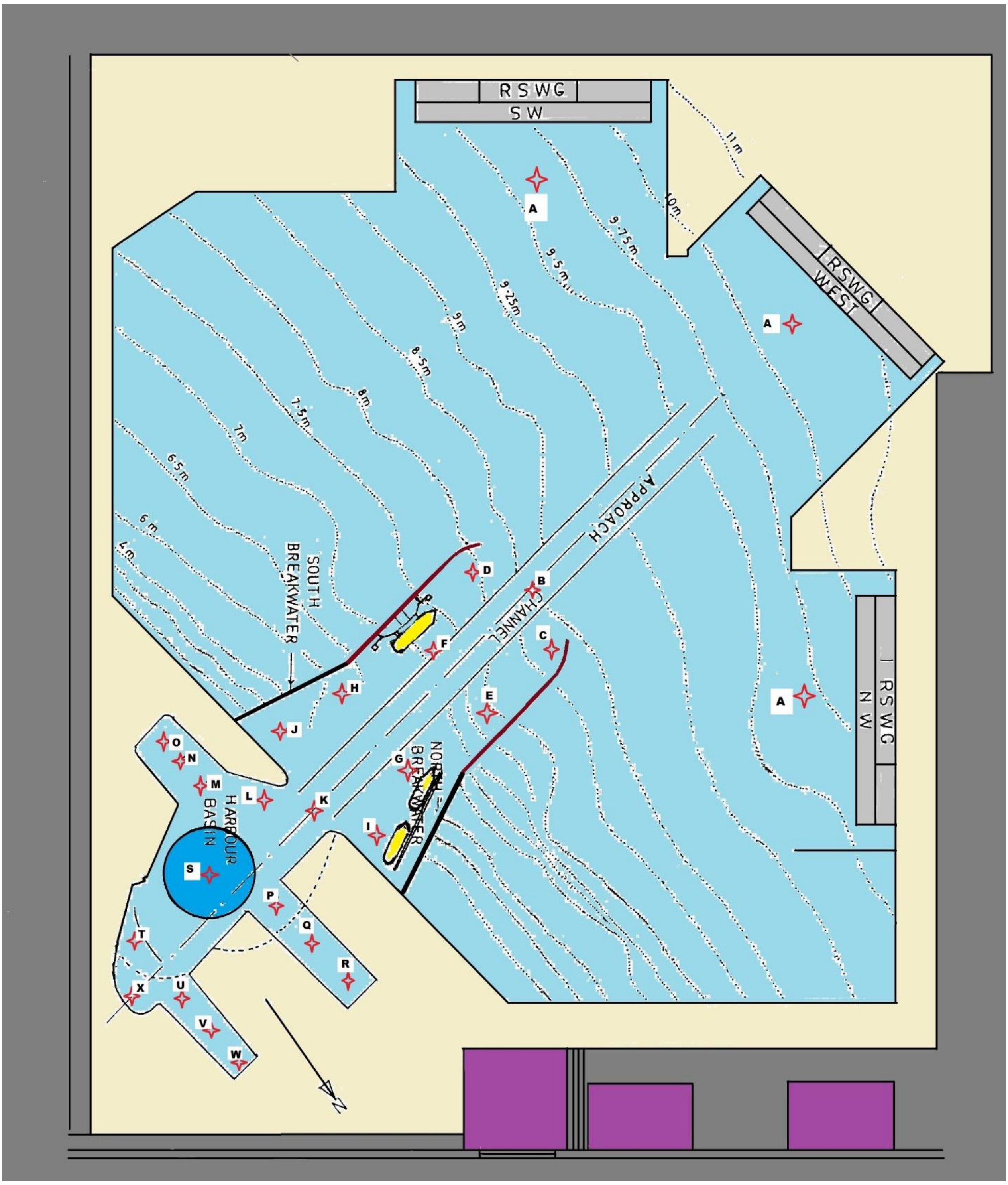


Figure 4.14 Location of Wave data acquisition points for breakwater extension with straight tips

Plate 4.7 Wave propagation with extension of breakwater with curved tips

Table 4.8 Wave tranquility in meters for waves from different directions breakwater extension with curved tips.

Sl.No	LOCATION	WAVE HEIGHT IN METERS		
		SW	W	NW
1	A	2.50	2.50	2.50
2	B	2.00	1.90	1.65
3	C	1.70	1.60	1.40
4	D	1.40	1.60	1.50
5	E	1.00	0.80	0.70
6	F	0.70	0.80	0.80
7	G	0.75	0.60	0.55
8	H	0.50	0.60	0.65
9	I	0.80	0.65	0.55
10	J	0.65	0.65	0.80
11	K	0.80	0.70	0.85
12	L	0.65	0.60	0.60
13	M	0.35	0.40	0.45
14	N	0.25	0.30	0.30
15	O	0.20	0.20	0.20
16	P	0.55	0.50	0.50
17	Q	0.40	0.30	0.35
18	R	0.30	0.20	0.20
19	S	0.70	0.65	0.60
20	T	0.50	0.55	0.45
21	U	0.30	0.20	0.25
22	V	0.20	0.15	0.15
23	W	0.15	0.10	0.10
24	X	0.45	0.40	0.45

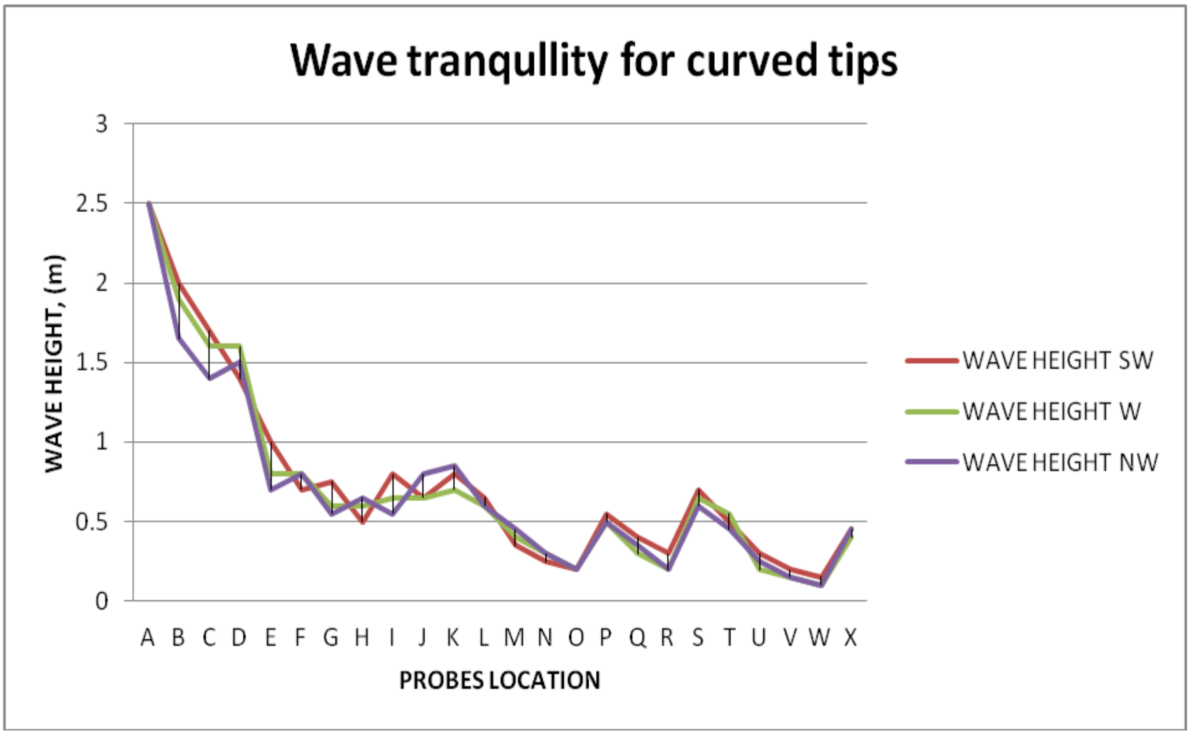


Figure 4.15 Variation of wave heights at salient locations for extension of breakwater with curved tips

4.5 STUDIES FOR FLOTILLA BERTHS

The studies conducted for flotilla berths to facilitate the berthing of vessels like tugs (draft 3.1m to 5.08m), Pilot/ multipurpose vessels/ survey launch (draft 1.0m to 2.1m) and mooring launches (draft 1.0m). In order to avoid hindrance to the main port traffic due to the small crafts and to maintain smooth operational conditions for small vessels.. A location on the eastern side of the port basin having a spending beach was identified for this development. This area is facing the harbour entrance, thus it is directly exposed to the waves entering the harbour basin. The existing spending beach in this region is functioning very well as a wave absorptive region and this is useful in maintaining the wave tranquility in the harbour basin. Considering the area available for the proposed Flotilla Berths and the size of the small crafts three finger jetties were conceptually planned. Sufficient margin of space is provided adjacent to existing berths so that the development of flotilla berthing structures should not pose any hindrance to movement/manoeuvring of the vessels. Considering the wave approach angle and to take advantage of the spending beach, three finger jetties of length 150 m each with a clear distance of 50 m face to face of each jetty structure are proposed (Fig. 4.8). To maintain the spending beach the jetties are planned on piles without disturbing the wave processes and currents along this location. The length of the proposed jetties were terminated before the Eastern face of the existing eastern dock arm since any projection beyond this limit may obstruct the ships entering or leaving the dock arm. This type of development essentially has varying draft for the proposed jetties from western to eastern end. This can be beneficially used to accommodate vessels of different drafts as per actual requirements. Small vessels are very sensitive to water movements in the harbour and particularly wave action and resonance effects; it is therefore common to undertake model studies. Comprehensive hydraulic physical model studies were conducted at CWPRS.

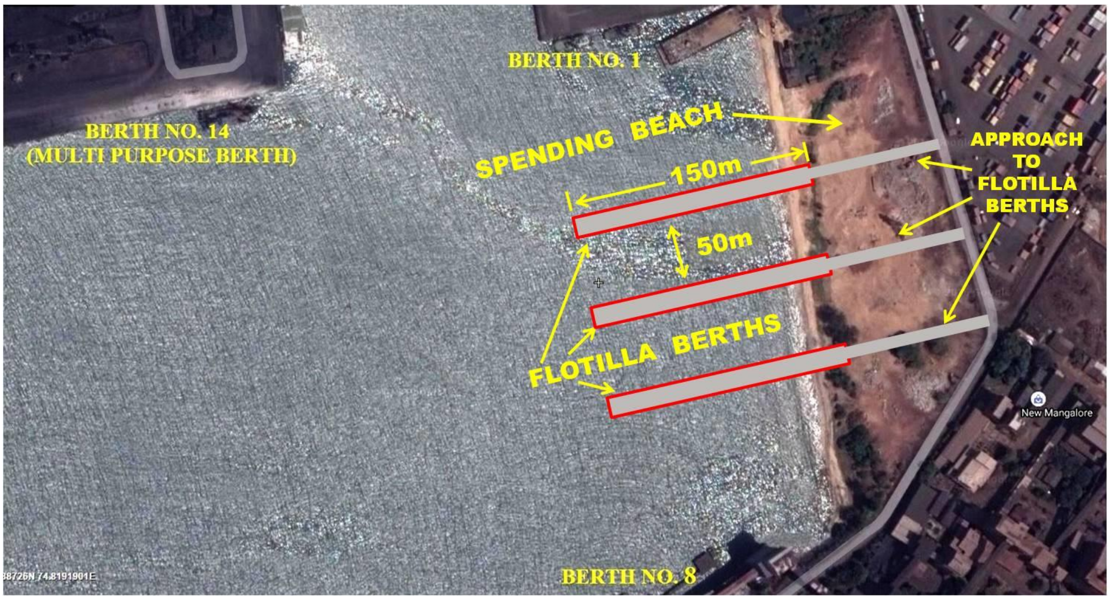


Plate 4.8 Plan showing location of Flotilla Berths and adjoining Berth Nos. 1& 8 (CWPRS Technical report no.5481, 2017)

The regular wave model studies were useful in sketching the wave crest pattern at the proposed berthing locations and random wave model studies were useful for the wave tranquility assessment at various salient locations in the model. Studies were conducted by simulating a wave absorptive surface on the eastern region of the proposed jetty by suitably placing the stone aggregates with sufficient wave run up to avoid reflection of incident waves from this region, resembling it to the beach absorptive effect at the site in the presence of spending beach.

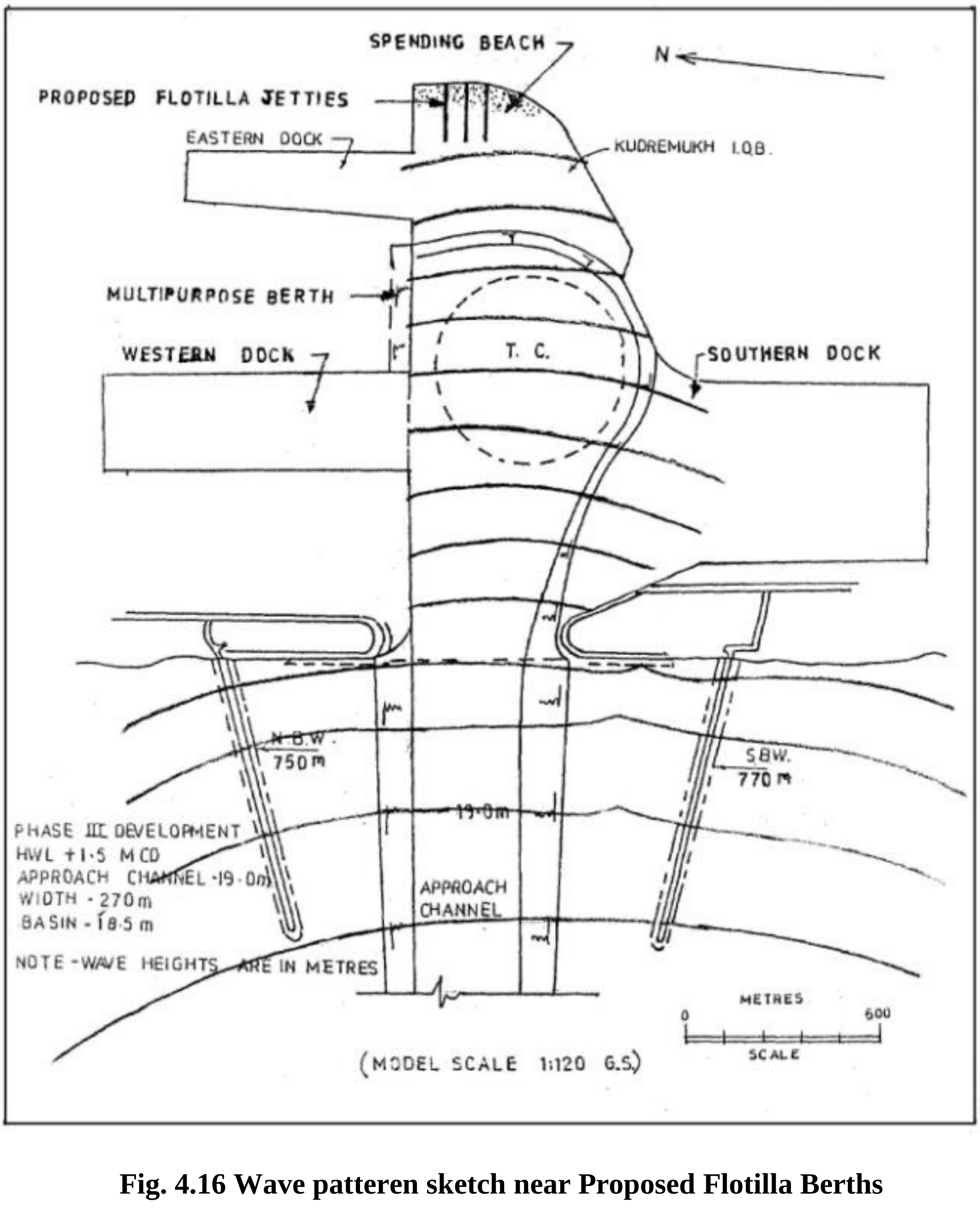


Fig. 4.16 Wave patteren sketch near Proposed Flotilla Berths

Plate 4.9 Model Photo showing wave crest near proposed flotilla berths

The results of wave tranquility for three directions are shown in Figures 4.17, 4.18, and 4.19.

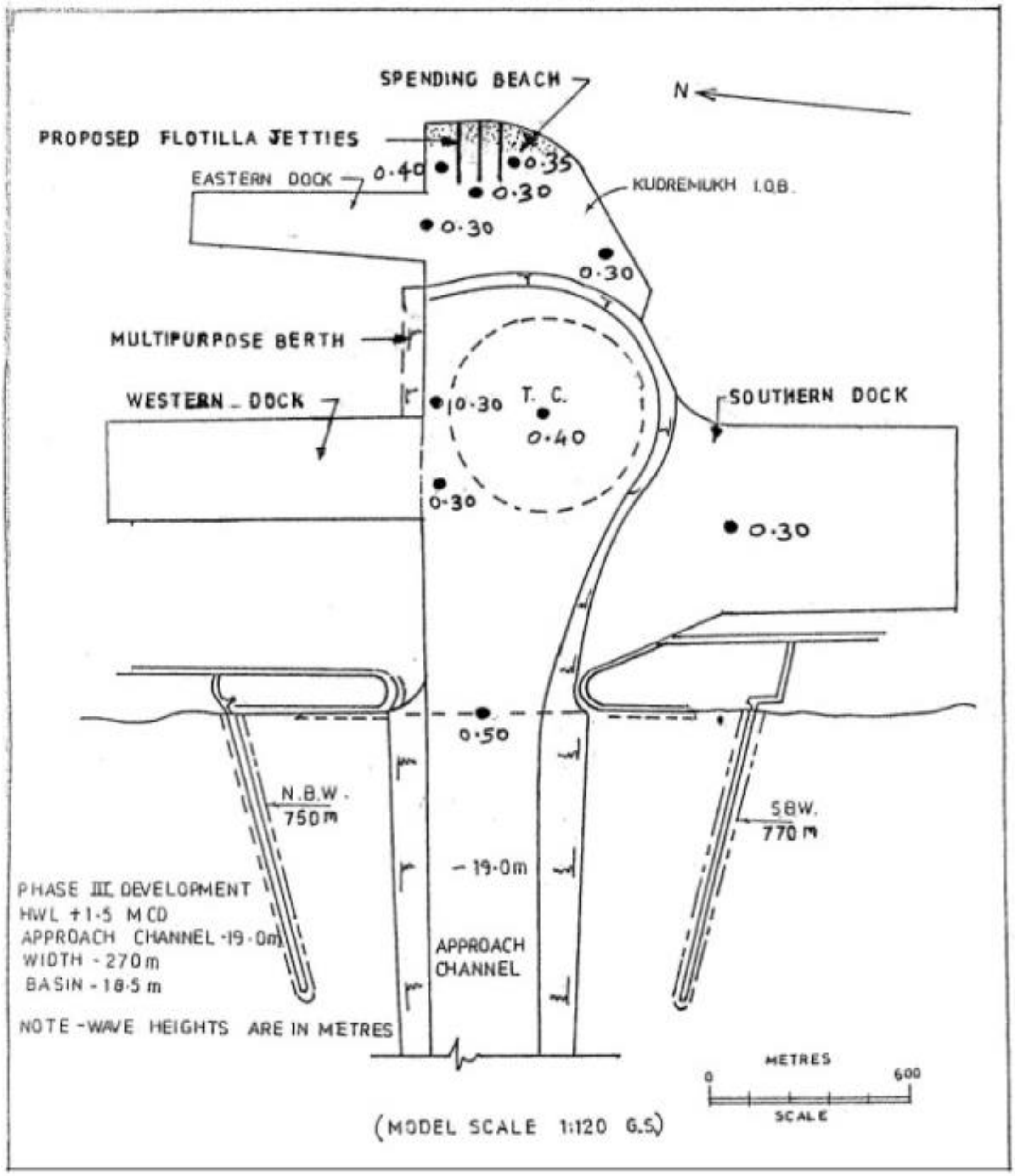


Fig.4.17 Wave tranquility for Westerly waves for the proposed Flotilla berths

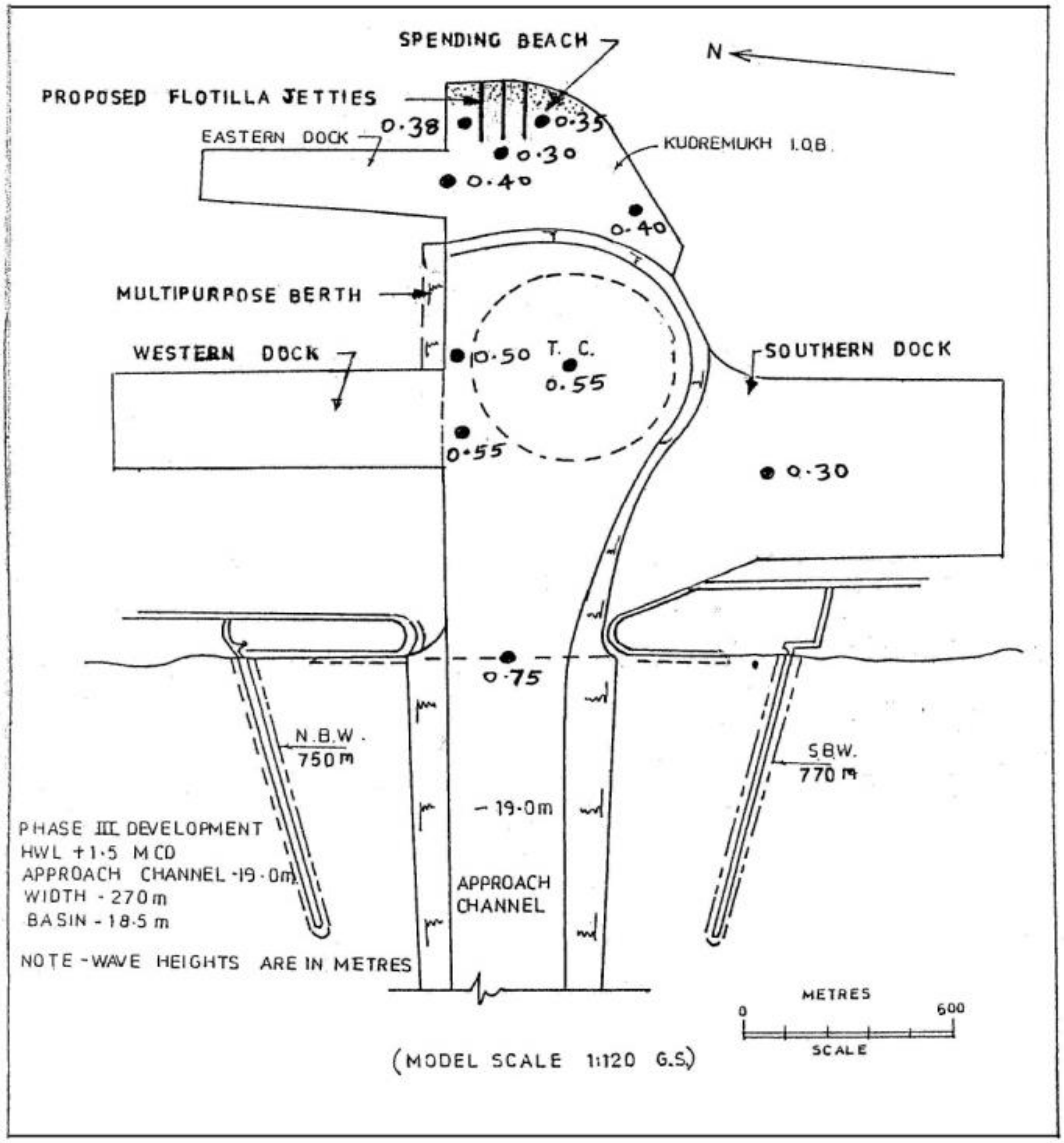


Fig.4.18 Wave tranquility for South-Westerly waves for the proposed Flotilla Berths.

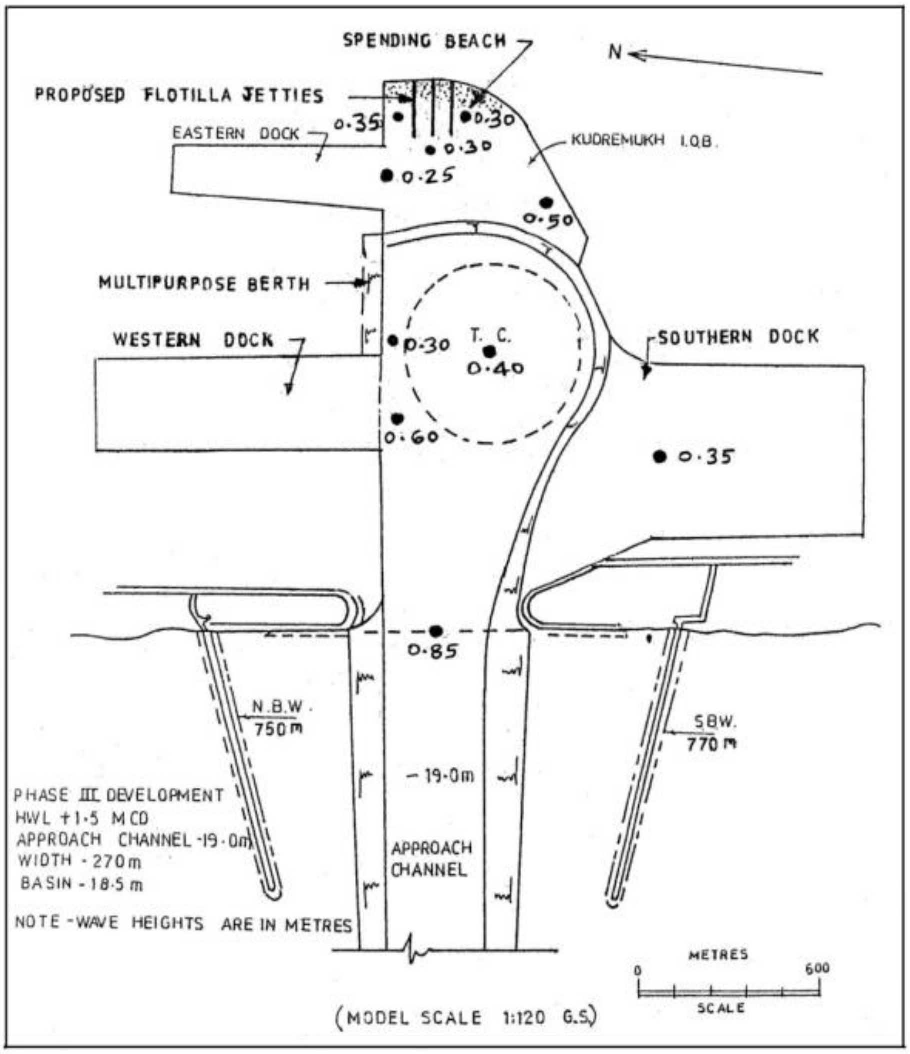


Fig.4.19 Wave tranquility for North-Westerly waves for the proposed Flotilla Berths.

The wave crest sketching and fine tuning of the flotilla berths were useful for finalization of optimum alignment of the berths with waves approaching the berths almost parallel to the berthed vessels. This helps in decreasing the vessel motion at berths thus increased comforts can be achieved during operative periods.

The studies for wave tranquility indicated the wave disturbance near proposed flotilla berths will be within 0.5m for the tested conditions from all three directions. The flotilla berthing faces facing existing Berth No.1 and Berth No.8 have relatively higher disturbance, this can be attributed to the reflection from these existing berth faces. Also the wave disturbances are little higher at the eastern side of the proposed berths as compared to western side, this is due to wave breaking and reflective effects from the spending beach adjacent to eastern side of the berths.

The measurement of wave disturbance at other locations in the port basin also indicated that there is no adverse effect on wave tranquility due to the development of flotilla berths. This is mainly due to pile jetties adopted for the proposed finger jetties which will minimize the reflective effects from the proposed development. It is also important to retain the spending beach on the eastern side of the berths to have wave absorptive effect reducing reflective and resonance phenomenon. Construction of any reflective structure in this region may lead to increased wave near the proposed berths and will increase the wave heights near the existing berth No.1 and also have an adverse effect on over all wave tranquility effect in the harbour basin.

The wave tranquility studies were conducted for Hs=2.5m at generator (which corresponds about 3.66m at deep water) is very rare from North-West direction and from South-West direction it may occur for about 10 days in a year. Considering these aspects the safe operative period of the flotilla berths can be conveniently determined. The tested wave condition prevails for the major part of the year.

4.6 RESULTS OF MODEL STUDIES FOR DIFFERENT BREAKWATER LENGTHSAND CHANNEL DIMENSIONS - EXTRACTED FROM CWPRS TECHNICAL REPORTS

The physical hydraulic model studies conducted for the New Mangalore Port development from early 1960s ever since the Mangalore harbour project proposal was made by Government of India. (CWPRS technical report no. 624, 1964) All the development woks under different stages of development were comprehensively studied by incorporating all the development proposals in the physical model maintained permanently at CWPRS, Pune.

The studies show that as and when the channel dimensions increased to facilitate higher capacity vessels into the harbour the wave tranquility inside the basin considerably improved Fig. 4.20. During first stage development the model was housed initially in open air and later in a bully shed partly covered having tripod bully supports for the model covering with A.C. sheets (Plate 4.15) and at important locations where the wave tranquilly were measured instead of a tripod support a monopod was erected. This was done to avoid wind and rain effects during the model studies. Regular wave generating facility was made use with a motor and a kop variator. The wave heights could be adjusted by varying the eccentricity in the wave generating board and wave period could be adjusted by varying frequency through kop variator. The layout of the port studied are as shown in Fig 4.21 to 4.23. (CWPRS technical report no. 640, 1964) (CWPRS technical report no. 2978, 1982) (CWPRS technical report no. 3086, 1993) Few photographs of the then existing models and the different layouts tested are as shown in Plate 4.10 to Plate 4.15.

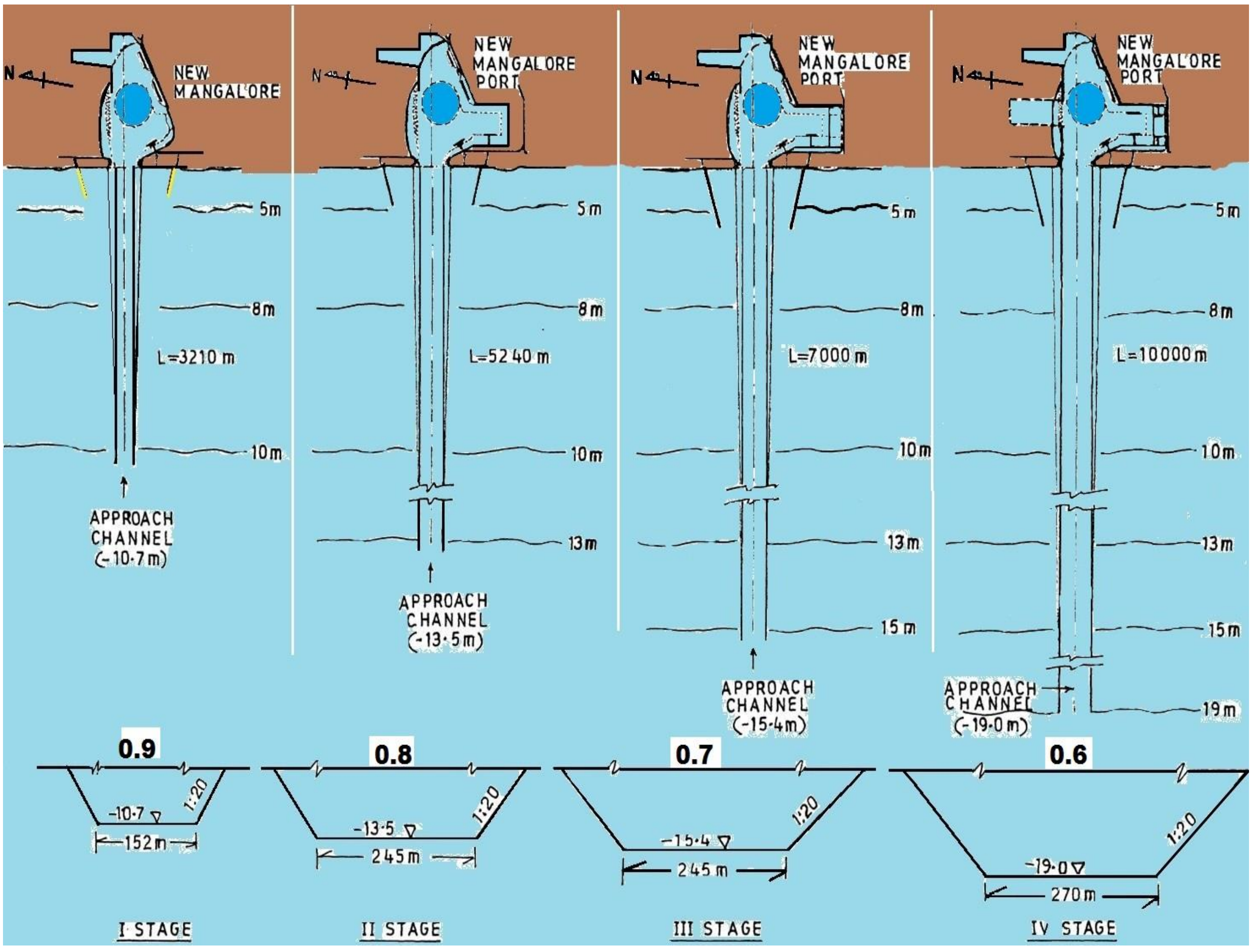


Figure 4.20 Studies for different stages of development

(WAVE HEIGHT AT THE PORT BASIN ENTRANCE IN M)

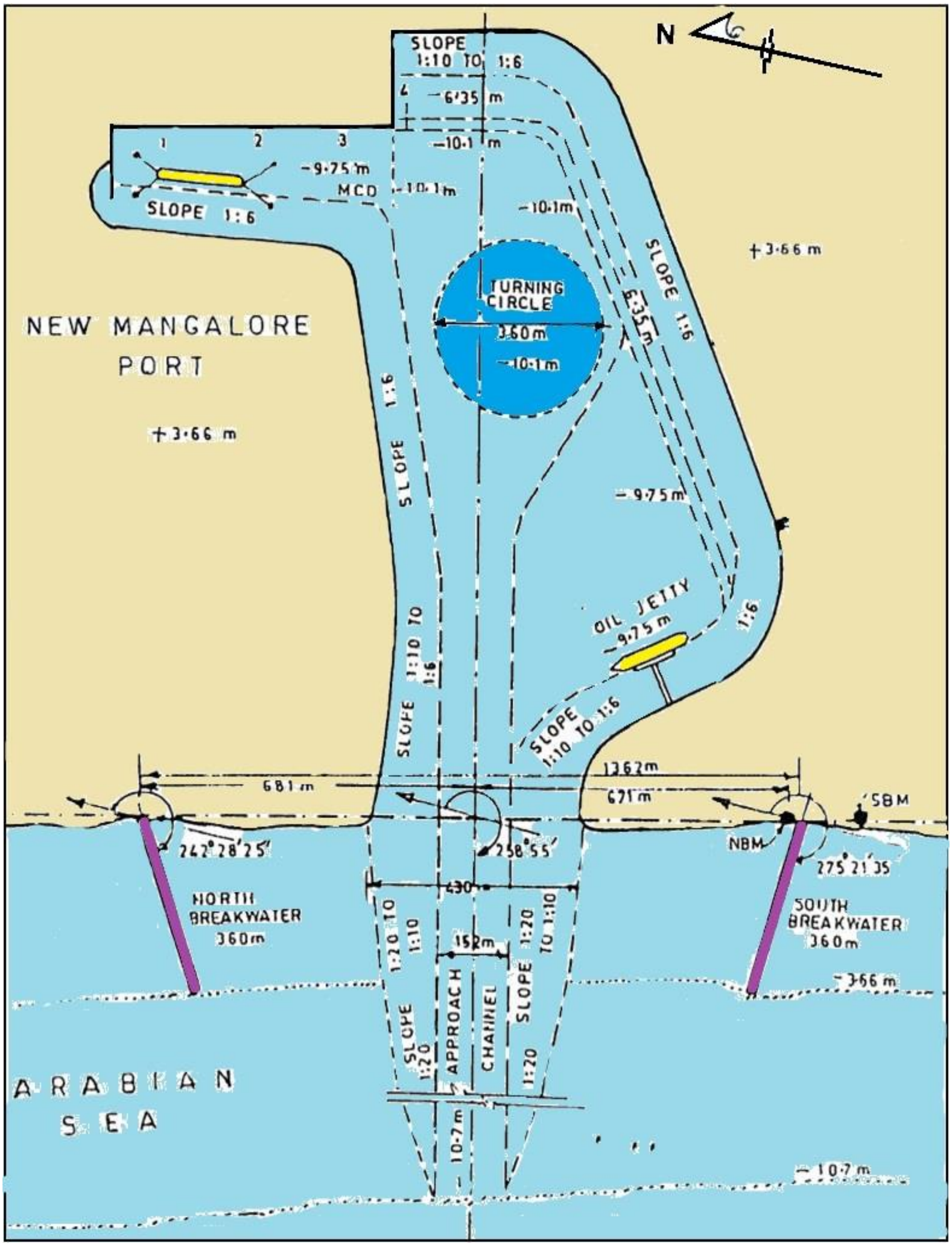


Fig. 4.21 Details of first stage development (CWPRS Technical report no. 909, 1969)

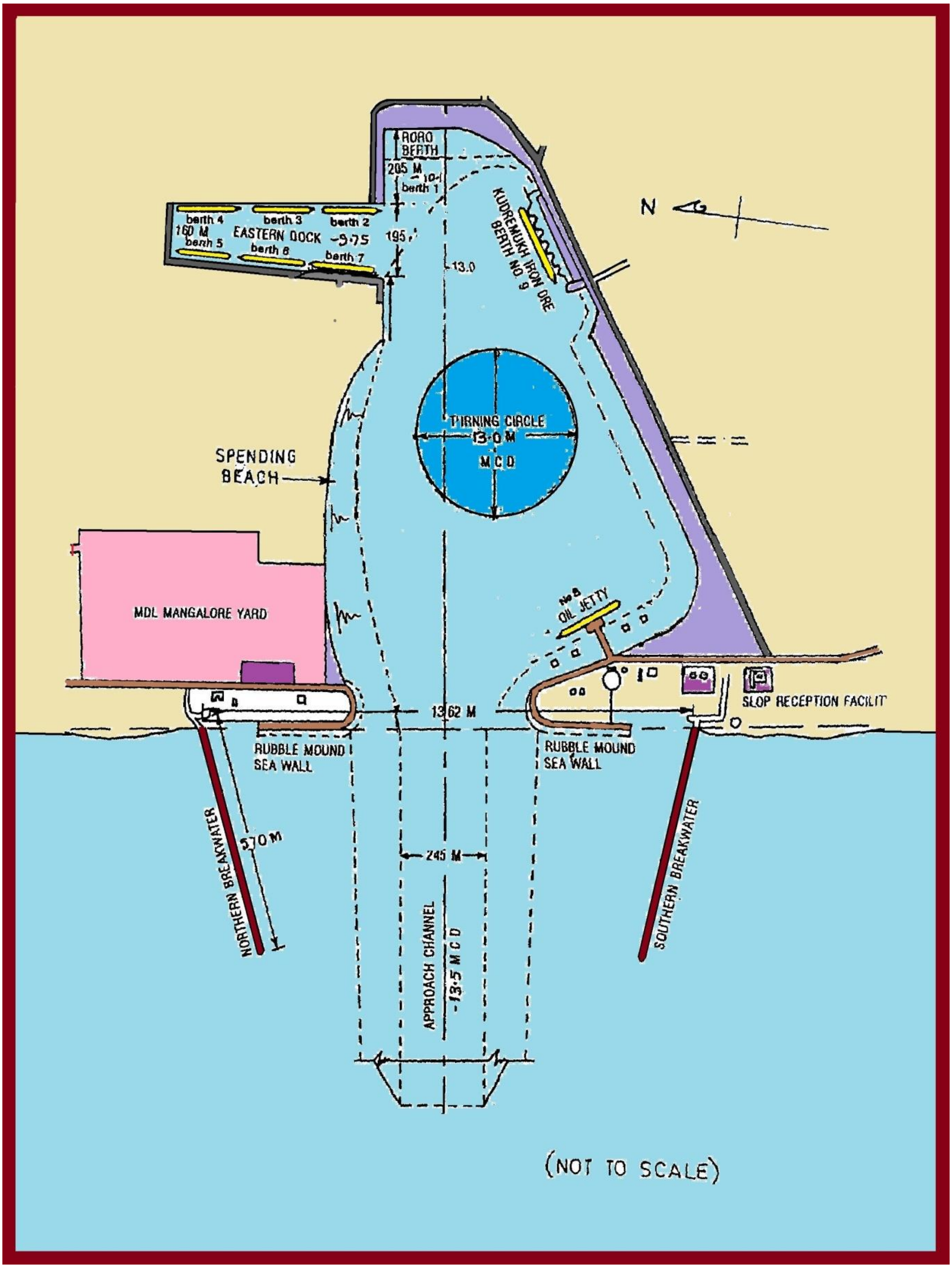


Fig. 4.22 Details of second stage development (CWPRS technical report no. 909, 1969)

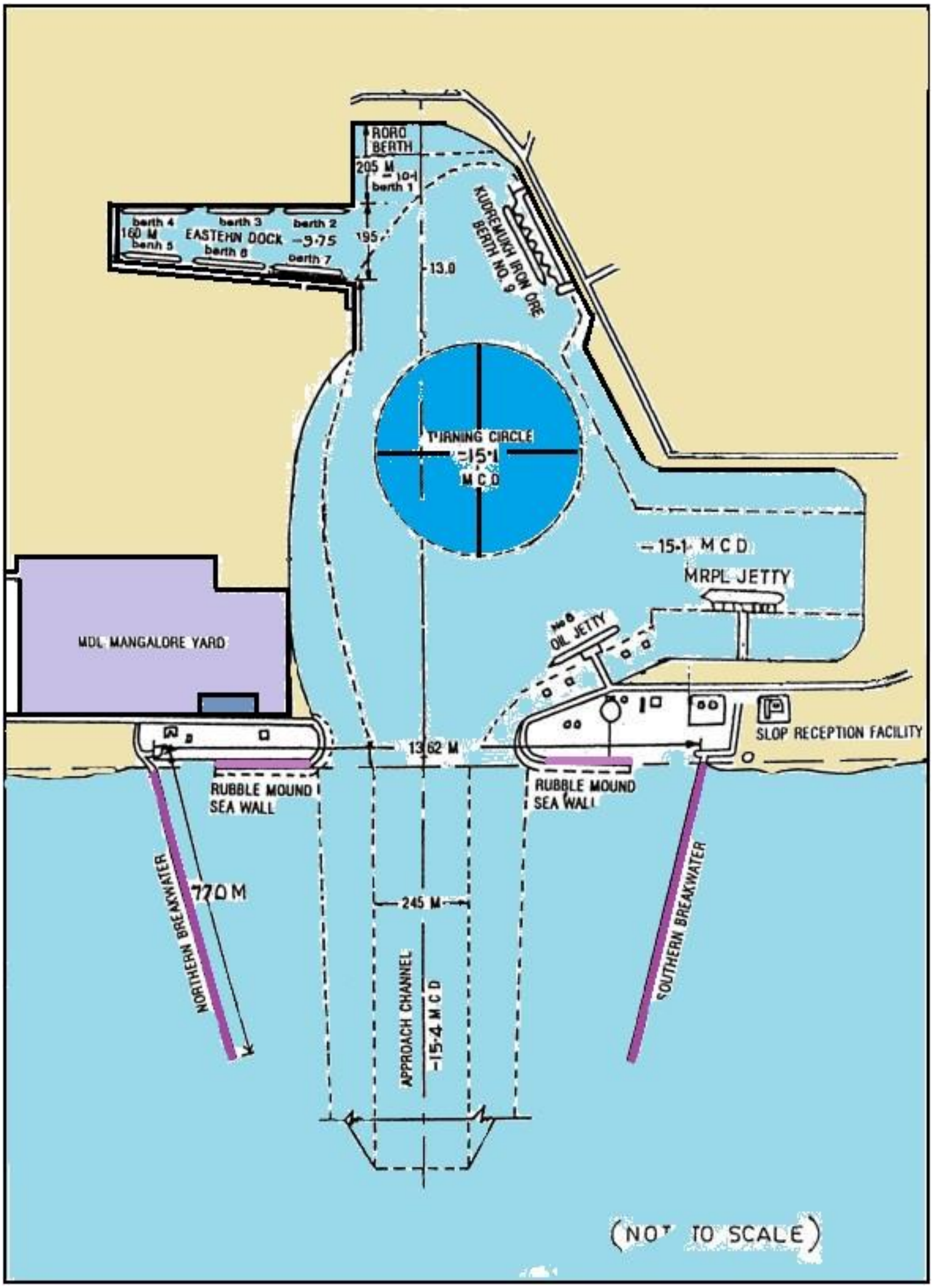


Fig. 4.23 Details of third stage development (CWPRS technical report no. 2978, 1982)

Plate 4.10 A view of model entrance

Plate 4.11 A view of dry model in open air.

Plate 4.12 A view of model with water in open air.

Plate 4.13 A view of Model data cabin.

Plate 4.14 A view of running Model from shore line.

Plate 4.15 A view of running model from sea side.

4.7 DISCUSSIONS ON MODEL RESULTS

4.7.1 Studies for wave attenuation

The results of the wave attenuation along the channel and along the adjacent sea bed for different conditions, it is observed in general that after the waves were generated, the wave height increases rapidly for about 500m length of the channel and later it reduces gradually all along the channel length. The increase in the wave heights at the generation is relatively more for vertical and 1:5 side slope condition. It was found that even with vertical edges of the channel, there was considerable wave attenuation along the length of the channel and the wave disturbance at port entrance was only 0.8 m for the waves of 2.5m generated at the boundary. For 1:5, 1:10 and 1:20 channel slopes wave disturbance of 0.7m, 0.65m and 0.6m respectively were observed at the entrance of the port basin. The wave heights along observed that Wave attenuation process is more systematic from 1250m to 2250m i.e. the last reaches of the channel. This may be attributed to partially developed sea wave state at the wave generator which gradually transforms to fully developed sea wave state as the waves travels away from generation along the sea bed bathymetry. The studies carried out with random waves also showed similar trends in respect of wave attenuation. During the experiments with random waves the difference in wave heights at different locations of wave observations are comparatively less. This may be attributed to the development fully developed sea state of waves within a short reach. This is mainly due to generation of waves of different heights and frequencies as compared to single wave height and period in regular wave generation.

Comparison of wave heights in the channel and lateral regions indicated that with the reduction in wave height in channel, there is increase in wave height along surrounding areas indicating the transfer of wave energy from the deeper channel portion to adjacent shallower regions. The wave propagation studies conducted with regular and random waves the following conclusions may be made.

The wave propagation studies have indicated that the wave attenuation is predominant for wave propagating along approach channeland this natural phenomenon may be efectively utilised for getting harbour tranquility for waves approaching along channel.This implies avoiding construction of long breakwaters which are costly particularly in weak sea bed soilconditions.This ia also an environmental friendly solution for port development.

The wave dispersion along the channel marginally increases with the increase in side slopes of the channel, thus in weak sea bed conditions due to higher side slopes better wave tranquility can be attained.

The random wave testing good developed sea wave state may be achieved within a short reach on the model. This will be useful in getting better prototype comparable results, effectively leading to achieving optimal harbour layout.

The use of random wave testing is advantageous in harbour tranquility studies, however if regular wave testing adopted for any reasons, testing with different wave heights and periods is recommended for getting optimal results.

4.7.2 Studies for directional propagation

The long crested waves generated in the model slowly disintegrate into short crested ones while travelling towards the harbour entrance. This is attributed to wave interference with the sea bed contours. A close observation into the values of rate of changes in the wave angles shows that the wave angles depends on the magnitude of difference in angle between the wave crest and the sea bed contours at the given location. Thus it can be seen that the rate of change in the crest angle is minimum for the waves generated from West direction as compared the waves from South-West and North-West direction abrupt change in the wave direction was observed while the wave front crosses the approach channel region in the model. This phenomenon can be attributed to the sudden increased depth in this region which causes sudden changes in the wave celerity in this zone. The local refraction due to the provision of side slopes of the channel, partial reflection within the channel were also contributes to these changes.

The minimum deviation of the wave crest while propagating towards harbour entrance from West direction may be attributed to refraction of the waves being lesser due to the minimum angle between the wave crest and the seabed contours along this direction of wave propagation. Thus it leads to minimum changes to wave directional propagation, for this reason the distance of the wave generator to the study area needs to be kept larger so that the long crested wave generated in the model to get modify to short crested waves as in the natural sea conditions while propagating along will take larger distance.

The waves generated from South-West and North-West directions changes in their directional propagation with in a very short distance since the angle of wave crest generated and the bottom sea bed contours are more along these directions. The wave pattern modifies to attain short crested as in the natural sea state condition with in a shorter reach as compared to the west direction waves. This ensures that the model boundary to be simulated for the wave generation can have a shorter distance whenever the wave oblicity is more with reference to bottom contours of the sea bed.

Thus it is evident that whenever a physical model is to be constructed, the angle between the sea bed contour and wave direction to be simulated needs to be examined very carefully before deciding the distance from the wave generator to the point of observation like harbour entrance, various locations in the harbour basin. More the angle lesser will be the length of wave propagation to achieve the fully developed sea state condition at the regions of interest and vice-versa This can be one of the conditions that need to be considered for selection of scale for shallow water wave models for studying wave propagation to harbour basins. Apart from these care needs be exercised to account for the sudden changes in wave pattern along the various structures in the direction of wave propagation in the model like approach channel, sudden bathymetric variations due to the presence of obstacles like island, local deep pockets etc. Sudden changes near the tip of breakwaters is in the presence of breakwater structure and the variation of the sea bed due to approach channel slopes are good examples for the abrupt changes in wave pattern within a short distance of wave travel in the model.

The assessment of directional asymmetry on shallow water wave basin can further developed by studying directional transformation for waves generated from different oblicity under standard testing conditions and nomograph can be developed. These will be useful in estimating the rate of change of wave direction and estimating the distance from where the waves need to be generated for a physical wave model for harbour planning based on wave tranquility studies. This also serves as a good guideline for deciding model boundary and scale during the initial planning of the shallow basin physical models.

4.8 Studies with different dimensions of channel, breakwater configurations, and with the effect of spending beach

The wave tranquility studies conducted for different lengths of breakwaters and channel dimensions have clearly indicated that as and when there is an increase in the channel dimension there is reduction in the wave energy entering into harbour basin. The wave disturbance measured at the harbour entrance was 0.9m for channel depth of 10.7m, it reduced to 0.8m for 13.5m channel depth,0.7m for 15.4m channel depth and 0.6m for 19m channel depth studied. It may be noticed that whenever an increase in channel depth there will be proportional increase in the length also. Thus phenomenon may be attributed to the increase in the sloping length available for the wave font to refract out from the channel reach. In addition the increase in level difference between the channel and adjacent sea bed increases the difference in the wave celerity along these two regions which finally results in more wave energy to pass out of the channel to adjacent shallower regions along the natural sea bed.

The effect of increase in the length of breakwaters is having no proportionate results on the reduction in the wave tranquility in the harbour for the waves approaching from western direction. Even the waves from Southwestern and Northwestern regions while propagating over the shallower regions in front of port approaches tends to approach from west direction due to refraction effects, thus the effects on wave tranquility by extending the breakwaters is not significant for these directions as well. The only benefit that can be expected through the extensions of breakwaters is additional stopping distance is available for the ships entering the harbour basin. The effects of retaining the spending beach while planning flotilla berths is useful in improving the wave tranquility conditions inside the harbour region.

CHAPTER 5
SUMMARY AND CONCLUSIONS

5.1 SUMMARY

The physical model studies for wave propagation along port approach channels and its effects on harbour tranquility is carried out in the coastal and off shore engineering laboratory of Central Water and Power research Station, Pune. Hydraulic physical model studies are conducted on 3-D rigid bed physical model constructed to geometrically similar scale of 1:100. The port approach channel is simulated with four different side slopes of vertical, 1:5, 1:10, 1:20 and wave tranquility tests are carried out by generating regular and random waves. During the tests water level in the model is maintained at +1.5m for all the conditions for uniform comparison conditions. Directional propagation studies are carried out to assess the directional spread of the waves generated in the model while traversing over the complex model bathymetry and port structures. The change of 2-D wave generated at wave generator to 3-D state in the harbour vicinity after undergoing the process of refraction reflection, and diffraction etc., and use of low cost 2-D wave generators instead of costlier 3-D paddle type wave generators in shallow basin modeling without compromising on the quality of model results is highlighted through model results.

The wave tranquility studies carried out for different lengths, alignment and channel dimensions is very helpful in proving the effect of breakwater and channel dimensions on harbour tranquility. The collection of model data for different lengths and cross sectional dimensions of the channel along with variation of breakwater lengths is very useful in drawing certain useful conclusions on the harbour wave tranquility and the factors governing this. The experiments on model with wave absorptive spending beaches, provision of mild slopes behind the proposed berths for wave run-up, its influence on wave tranquility in general within harbour basin are compared.

5.2 CONCLUSIONS FOR THE WAVE ATTENUATION STUDIES

Based on this present investigation the following conclusions are drawn

- Studies carried out revealed that with vertical sides of the channel, 87% of wave energy was transmitted outside the channel within a reach of 2250m and this energy transmission was 90%, 92% and 93% for channel with side slopes of 1:5, 1:10 and 1:20 respectively.
- It was found that with the vertical sides of the channel, there was wave attenuation along the length of the channel, and wave disturbance at the entrance of the port basin was only 0.8 m for 2.25m / 10 s. waves at the seaward end of channel.
- The wave attenuation could be attributed to transfer of wave energy along the edges of the channel due to interaction of waves of higher celerity in the dredged channel compared to low celerity waves outside shallow region natural sea bed adjoining the dredged channel. This interaction is in the form of internal diffraction. The wave celerity in the approach channel was estimated to be 11m/sec and the wave length of 110m in contrast to wave celerity of 8.82m/sec and wave length of 88m at 9m depth, whereas the wave celerity was 5.3m/sec with wave length of 53m at 3m depth contour, near shore.
- Studies with channel side slopes of 1:5, 1:10 and 1:20 showed that wave disturbances in the approach channel was having more attenuating effect compared to channel with the vertical sides The wave energy dispersed from channel due to the wave attenuation process was 87% in case of channel with vertical sides, 90%, 92% and 93% for channel side slopes of 1:5, 1:10 and 1:20 respectively. This could be attributed to transfer of wave energy due to wave refraction effects on the side slopes in addition to lateral internal diffraction of wave energy due to difference in wave celerity in the channel and surrounding region. The study revealed that the approach channel has some natural wave attenuating mechanism due to diffraction effects caused by difference in wave celerity in the channel and outer region, wave refraction and combined diffraction effects on the side slopes as well.

- Wave attenuation is an important phenomenon in wave propagation studies along complex bathymetric conditions for a port layout design. Wave attenuation in long approach channel to ports occurs due to difference in wave celerity in dredged approach channel and surrounding natural shallow regions. Internal wave diffraction effects, refraction and diffraction on the channel side slopes are the other major concerns for the wave attenuation process.
- This phenomenon of wave attenuation in channel can be used beneficially in port layout designs when the harbour entrance is facing the critical wave approach direction. In contrast to earlier studies the wave attenuation not solely dependent on the side slopes but it depends on the wave celerity variation in the channel and the adjacent sea bed bathymetric conditions.
- The contribution of bed friction and sea bed percolation to the wave decay is assumed to be negligible compared with the effect of wave celerity and refraction due to side slopes. There is scope for literature review in this direction to find the latest development both theoretical and laboratory investigations.

5.3 CONCLUSIONS FOR THE DIRECTIONAL PROPAGATION STUDIES

Based on the experimental results of model study on conventional breakwater model constructed with concrete cube armor, the following conclusions are drawn.

- Long crested waves generated from the critical directions in model disintegrate into short crested ones resembling natural sea conditions upon propagation towards the model testing area. Thus the long crested waves generated in a 2-D form in model can be used for wave tranquility studies in harbour planning using shallow basin physical models.
- The change in the wave crest pattern depends mainly on the obliquity of wave approach and other coastal structures that intervenes the wave front along the direction of propagation. Hence sufficient care needs to be exercised to determine the model boundary of a shallow basin wave model. In order to

have fully developed 3-D state the model boundary will be comparatively at farther distance when the wave obliquity is less

- A shallow basin physical wave models are economical tool for the studies for wave propagation in the near shore region to study the complex wave conditions occurring due to various phenomena the waves undergoes in the near shore regions. These types of models have been used for the development of various coastal development works in the near shore regions successfully.
- The assessment of directional asymmetry on shallow water wave basin can further be developed by studying directional transformation for waves generated from different obliquity and standard conditions for simulation of boundary conditions in physical models can be established. Further by incorporating suitable structures in front of long wave boards it is possible to modify the long crested waves into short crested waves resembling the real sea state. This process will reduce the model cost since the generation of short crested waves using multi-peddle type of wave generation are more complex and costlier. This will also help in reducing the size and the cost of the physical models.

5.4 CONCLUSIONS FOR WAVE TRANQUILITY STUDIES WITH BREAKWATER EXTENTION

- The wave tranquility studies conducted for different lengths of breakwaters and channel dimensions have clearly indicated that as and when there is an increase in the channel dimension there is reduction in the wave energy entering into harbour basin. This may be attributed to the increase in the sloping length available for the wave font to refract out from the channel reach. In addition the increase in level difference between the channel and adjacent sea bed increases the difference in the wave celerity along these two regions which finally results in more wave energy to pass out of the channel to adjacent shallower regions.

- The effect of increase in the length of breakwaters is having no proportionate results on the reduction in the wave tranquility in the harbour for the waves approaching from western direction.
- Even the waves from Southwestern and Northwestern regions while propagating over the shallower regions in front of port approaches tends to approach from west direction due to refraction effects, thus the effects on wave tranquility by extending the breakwaters is not significant for these directions as well.

5.5 RECOMMENDATIONS

While developing harbours on open coasts the natural advantage of wave attenuation along long port approach channels can be effectively utilised by proper alignment of approach channel. For this reason the approach channel can be aligned and entrance can face critical wave direction also.

The alignment of channel perpendicular to the sea bed contour will effectively reduce the channel length, thus savings can be achieved in capital and maintenance cost of dredging.

While planning the berths it is recommended to provide sloping surface behind the berths facilitating wave run up and dissipation of wave energy. This effectively improves the wave tranquility conditions in the harbour basin.

The naturally formed spending beaches in the harbour basins are very effective in absorbing the wave energy penetrating through the harbour entrance it is recommended to preserve these spending beaches by resorting to piled jetties along at these locations.

For wave tranquility studies in shallow basin wave models generation of long crested random waves will provide sufficiently good results and use of 3-D peddle type wave generators can be restricted to off shore structures. This reduces the cost of model studies for wave tranquility.

5.6 SCOPE FOR FUTURE WORK

1. During the course of studies on wave attenuation, it is noticed that there is a sudden increase in wave height in the initial reaches of the channel. The exact reason for this phenomenon can be explored. It's possible to study with some enlarged model scale the reflection effects for possible effect on sudden increase in the wave heights in front of wave generator can be studied with different wave frequencies.
2. Sudden increase in wave heights in the channel is noticed at certain reaches, this phenomenon also need to be established with some basic research works.
3. The assessment of directional asymmetry on shallow water wave basin can further be developed by studying directional transformation for waves generated from different obliquity with the bottom sea bed contours. These will be useful in estimating the rate of change of wave direction and estimating the distance of wave board from harbour entrance in the model.
4. By incorporating suitable structures in front of long wave boards it is possible to modify the long crested waves into short crested resembling the natural sea state near the wave generator itself. The type of structure to be incorporated needs to be determined with some basic studies. This will reduce the model cost since the generation of short crested waves using multi-peddle type is more complex and costlier as well. This will also help in reducing the size of the physical model which in turn reduces the cost of the model studies.

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