15 -ാം കേരള നിയമസഭ

11 -ാം സമ്മേളനം

നക്ഷത്ര ചിഹ്നം ഇല്ലാത്ത ചോദ്യം നം. 1199 **12-06-2024 - ൽ മറുപടിയ്ക്ക്**

മുതലപ്പൊഴി ഹാർബർ നിർമ്മാണത്തിലെ അശാസ്ത്രീയത

ചോദ്യം		ഉത്തരം	
ശ്രീ. എം. വിൻസെന്റ്		ശ്രീ സജി ചെറിയാൻ (മത്സ്യബന്ധനം, സാംസ്കാരികം, യുവജനകാര്യ വകുപ്പ് മന്ത്രി)	
(എ)	മുതലപ്പൊഴി ഹാർബർ നിർമ്മാണത്തിലെ അശാസ്ത്രീയത സംബന്ധിച്ച് സി.ഡബ്ല്യൂ.പി.ആർ.എസ് (സെൻട്രൽ വാട്ടർ ആൻഡ് പവർ റിസർച്ച് സ്റ്റേഷൻ) പഠനം നടത്തി അന്തിമ റിപ്പോർട്ട് സമർപ്പിച്ചിട്ടുണ്ടോയെന്ന് വ്യക്തമാക്കുമോ; ഉണ്ടെങ്കിൽ പ്രസ്തുത റിപ്പോർട്ടിന്റെ പകർപ്പ് ലഭ്യമാക്കാമോ?	(എ)	മുതലപ്പൊഴി മത്സ്യബന്ധന തുറമുഖം അപകടരഹിതമാക്കുന്നതിനെക്കുറിച്ച് പഠിച്ച് റിപ്പോർട്ട് സമർപ്പിക്കുവാൻ 22/08/22 തീയതിയിലെ സർക്കാർ ഉത്തരവ് (സാധാ) നം. 660/2022/മതുവ പ്രകാരം പൂനെ ആസ്ഥാനമായുള്ള സെൻട്രൽ വാട്ടർ ആൻഡ് പവർ റിസർച്ച് സ്റ്റേഷനെ ചുമതലപ്പെടുത്തുകയും തുടർന്ന് സെൻട്രൽ വാട്ടർ ആൻഡ് പവർ റിസർച്ച് സ്റ്റേഷൻ ഉദ്യോഗസ്ഥർ നേരിട്ട് ഡേറ്റാ കളക്ഷൻ നടത്തി മത്സ്യത്തൊഴിലാളികൾ, ജനപ്രതിനിധികൾ ഉൾപ്പെടെയുള്ള പ്രദേശവാസികളുമായി ചർച്ച നടത്തി. ആയത് പ്രകാരം മാതൃകാപഠനം പൂർത്തിയാക്കി റിപ്പോർട്ട് സമർപ്പിക്കുകയുണ്ടായി. ടി റിപ്പോർട്ട് പ്രകാരം തെക്ക് ഭാഗത്ത് നിന്നും അഴിമുഖത്തേക്ക് മണ്ണ് അടിഞ്ഞ് കൂടുന്നത് തടയുന്നതിന് വേണ്ടി തെക്കേ പുലിമുട്ടിന്റെ നീളം വർദ്ധിപ്പിച്ച് അഴിമുഖം വടക്ക് ഭാഗത്തേക്ക് മാറ്റുന്ന വിധത്തിലാണ് പുലിമുട്ടിന്റെ രൂപകൽപ്പന നൽകിയിട്ടുള്ളത്. പ്രസ്തുത റിപ്പോർട്ട് അനുബന്ധമായി ചേർക്കുന്നു. സെൻട്രൽ വാട്ടർ ആന്റ് പവർ റിസർച്ച് സ്റ്റേഷൻ (CWPRS) ലഭ്യമാക്കിയ റിപ്പോർട്ടിന്റെ അടിസ്ഥാനത്തിൽ 164 കോടി രൂപയുടെ വിശദമായ പദ്ധതി രൂപരേഖ തയ്യാറാക്കി കേന്ദ്ര സർക്കാരിന് സമർപ്പിച്ചിട്ടുണ്ട്.

സെക്ഷൻ ഓഫീസർ

Government of India
Ministry of Jal shakti
Department of Water Resources,
River Development and
Ganga Rejuvenation

सत्यमेव जयते

भारत सरकार
जल शक्ति मंत्रालय
जल संसाधन, नदी विकास
और गंगा संरक्षण विभाग

Technical Report No. 6215
March 2024

MATHEMATICAL MODEL STUDIES FOR HYDRODYNAMICS, SILTATION, WAVE PROPAGATION AND SHORELINE CHANGES FOR THE PROPOSED DEVELOPMENT OF MUTHALAPOZHY FISHERIES HARBOUR, KERALA

केन्द्रीय जल और विद्युत अनुसंधान शाला, पुणे
CENTRAL WATER AND POWER RESEARCH STATION, PUNE

Dr. R.S. Kankara
Director

सत्यमेव जयते
भारत सरकार / Government of India
जल शक्ति मंत्रालय / Ministry of Jal Shakti
जल संसाधन, नदी विकास और गंगा संरक्षण विभाग
Department of Water Resources, River Development and Ganga Rejuvenation
केन्द्रीय जल और विद्युत अनुसंधानशाला
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दिनांक: 06 MAR 2024

To,

कैप्टन प्रदीप मोहंती / The Chief Engineer
उप संरक्षक / Harbour Engineering Department
दीनदयाल बंदरगाह प्राधिकरण / Kamaleswaram
गांधीधाम (कच्छ)/Manacaud
गुजरात - 370201 / Thiruvananthapuram -695 009

विषय : "Mathematical Model Studies for Hydrodynamics, Siltation, Wave Propagation and Shoreline Changes for the Proposed Development of Muthalapozhy Fisheries Harbour, Kerala." से सम्बंधित तकनीकी रिपोर्ट।

महोदय,

Please find enclosed herewith two copies of the technical report on "Mathematical Model Studies for Hydrodynamics, Siltation, Wave Propagation and Shoreline Changes for the Proposed Development of Muthalapozhy Fisheries Harbour, Kerala" bearing No. 6215 of March 2024. For any clarification in the studies, you may contact Dr. Jiweshwar Sinha, Scientist-E (Tel. 020-2410 3293).

Receipt of the same may please be acknowledged.

निदेशक, के. ज. और वि. अ. शाला के अनुमोदन से यह जारी किया जाता है।
धन्यवाद,

भवदीय,

संलग्न : तकनीकी रिपोर्ट की दो प्रतियाँ।

6/3/2024

(एस. जी. मंजुनाथ / S. G. Manjunatha)
वैज्ञानिक 'ई' (तकनीकी समन्वय)
Scientist 'E' (Technical Co-ordination)
For Director

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सत्यमेव जयते

भारत सरकार / Government of India

जल शक्ति मंत्रालय / Ministry of Jal Shakti

जल संसाधन, नदी विकास और गंगा संरक्षण विभाग

Department of Water Resources, River Development and Ganga Rejuvenation

केन्द्रीय जल और विद्युत अनुसंधान शाला

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CENTRAL WATER & POWER RESEARCH STATION

Khadakwasla, Pune – 411024

MATHEMATICAL MODELING IN COASTAL ENGINEERING DIVISION

Technical Report No. 6215

March, 2024

MATHEMATICAL MODEL STUDIES FOR HYDRODYNAMICS, SILTATION, WAVE PROPAGATION AND SHORELINE CHANGES FOR THE PROPOSED DEVELOPMENT OF MUTHALAPOZHY FISHERIES HARBOUR, KERALA

Dr. R.S. KANKARA

Director

REPORT DOCUMENTATION SHEET

Technical Report No.6215 **March, 2024**

Title: Mathematical Model Studies for Hydrodynamics, Siltation, Wave Propagation and Shoreline Changes for the Proposed Development of Muthalapozhy Fisheries Harbour, Kerala.

Officers Responsible for Conducting the Studies :

These studies were conducted by Shri B.L. Meena Scientist 'C', Dr. A.K. Singh, Scientist 'C', Dr. Anil Kumar Bagwan Scientist 'B', Mrs. Vaibhawi Roy Research Assistant, under the supervision of Dr. Jiweshwar Sinha, Scientist-E and Dr. Prabhat Chandra, Additional Director under overall guidance of Dr. R.S. Kankara, Director, CWPRS, Pune.

Name and Address of Organization Conducting the Studies :

Mathematical Modelling for Coastal Engineering Division
Coastal and Offshore Engineering Laboratory,
Central Water and Power Research Station, Khadakwasala Pune -411024, India

Name and Address of Authority Sponsoring the Studies:

The Chief Engineer, Harbour Engineering department, Kamaleswaran,Manacaud – P.O. Thiruvananthapuram-695 009

Synopsis

Muthalapozhy fishing harbor is located inside a coastal inlet on the West Coast of India between Vizhinjam and Thangassery in Kerala. The Vamanpuram River falls into Arabian Sea through this inlet. The entrance of the inlet is silted up mainly due to net littoral drift being transported from south to north causing the operation of fishing vessels extremely difficult through the inlet during rough weather season. Considering the difficulties being faced by the fishermen, Harbour Engineering Department (HED), Kerala, the Project Authority, prepared a proposal for rectifying the problem at Muthalapozhy inlet. In order to address the siltation issue at the entrance of the inlet, it was proposed to train the inlet by providing extension to the existing breakwaters. The existing northern breakwater and southern breakwater lengths are 410m and 430m respectively. Recently several accidents of fishing boats are reported. In order to have a safe entrance at the inlet, it is proposed to train the inlet by providing extension to the existing breakwaters. Different layouts were simulated and optimized breakwater extension of southern breakwater of length 420m was found suitable. With optimized layout littoral drift studies indicates that there is no need of sand bypassing for the next 25 years. Wave tranquility studies reveal that significant wave heights will be of order 0.6m at the entrance. From hydrodynamics study it was observed that currents in existing condition near bridge will be of the order of 0.3m/s which reduced to 0.2m/s with proposed development. Sedimentation studies reveal that there is negligible siltation in existing condition where as 10cm siltation was observed in the dredged approach channel (-6m below C.D) annually. With the optimized layout there is no adverse impact from wave tranquility as well as hydrodynamics point of view, hence the layout is recommended.

Keywords : Mathematical model, hydrodynamics, sedimentation, tide and current.

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MATHEMATICALMODEL STUDIES FOR HYDRODYNAMICS, SILTATION, WAVE PROPAGATION AND SHORELINE CHANGES FOR THE PROPOSED DEVELOPMENT OF MUTHALAPOZHY FISHERIES HARBOUR, KERALA

Technical Report No. 6215 **Date: March 2024**

1. INTRODUCTION

Muthalapozhy fishing harbor is located inside a coastal inlet on the West Coast of India at 8^0 38^0 N latitude and 76^0 50^0 E longitude between Vizhinjam and Thangassery in Kerala State. Location of the coastal inlet is shown in Fig.1. The Vamanpuram River falls into Arabian Sea through this inlet. The entrance of the inlet is silted up mainly due to net littoral drift generally transported from south to north causing the operation of fishing vessels extremely difficult through the inlet during rough weather season.

Fig.1 Location of coastal inlet at Muthalapozhy

Considering the difficulties being faced by the fishermen, Harbour Engineering Department (HED), Kerala State, the Project Authority, prepared a proposal for rectifying the problem at Muthalapozhy inlet. In order to address the siltation issue at the entrance of the inlet, it was proposed to train the inlet by providing extension to the existing breakwaters. Recently, several accidents of fishing boats were reported near the entrance of inlet channel. In order to have a safe entrance at the inlet, it was proposed to train the inlet by providing extension to the existing breakwaters. HED officials visit CWPRS in January and February 2024 to discuss about optimum breakwater layout evolved.

The mathematical model studies were undertaken at CWPRS for evolution of the optimum layout and the hydraulic aspects were studied in respect, wave tranquility, littoral drift and shore line evaluation, flow condition and sedimentation under existing condition and with suggested optimum layout. This report describes mathematical model studies carried out for estimation of littoral drift distribution and shoreline changes, and for examining the wave tranquility due to the proposed extension of breakwaters at the inlet. To examine the flow conditions hydrodynamic studies and to assess the siltation pattern in the channel and near the entrance sediment studies were conducted.

2. SCOPE OF STUDIES

Mathematical model studies were undertaken to examine shoreline changes (1-D model), for examining wave tranquility (2-D model) and for determination of siltation pattern (2-D) due to tidal effect in the channel. The studies were carried out in the following stages:

(i) Littoral drift distribution and shoreline evolution studies using LITPACK model.
(ii) Transformation of wave height and wave direction while propagating from deep water to -20 m depth contour (w.r.t.CD) using MIKE21 SW model to obtain nearshore wave conditions.
(iii) Wave tranquility studies for accessing the wave disturbance at the entrance of the inlet using MIKE21 BW model.
(iv) To assess the hydrodynamics condition and siltation pattern inside the navigational channel and in harbour area.

3. SITE CONDITIONS

3.1 Bathymetry

Bathymetry of the Muthalapozhy inlet site was provided by the Project Authority which indicated mostly uniform contours parallel to the coastline. Part bathymetry of seaside was taken from C-map.

3.2 Tidal Levels and Currents

Tidal level at one location and tidal currents at three locations were observed by CWPRS for a period of ten days in the month of October 2023. The tidal range during spring tide was found to be 1.0m while during neap tide it was only 0.2m. The maximum tidal currents at the site were observed to be of the order of 0.6m/s.

3.3 Sediment Characteristics

Sediment samples were also collected and analyzed by CWPRS indicating fine sand to silt having sizes ranging from 0.18mm to 0.35mm at different location of the channel exists at the site.

3.4 Offshore Wave Climate

For determination of propagation of waves and, computation of littoral drift and shoreline changes, nearshore wave data is essential. Instrumentally observed wave data at the site over a period of several years, if available, is the best suited for this purpose. However, due to non-availability of such data, the nearshore wave climate can be obtained by transforming the offshore wave data to a nearshore location by using mathematical modeling technique. For this purpose, the offshore wave data extracted from ERA5 has been used which is the fifth generation ECMWF re-analysis for the global climate and weather for the past 8 decades. Data is available from 1940 onwards. ERA5 replaces the ERA-Interim re-analysis.

Re-analysis combines model data with observation from across the world into a globally complete and consistent dataset using the laws of physics. This principle, called data assimilation, is based on the method used by numerical weather prediction centre, where every so many hours (12 hours at ECMWF) a previous forecast is combined with newly available observations in an optimal way to produce a new best estimate of the state of the atmosphere, called analysis, from which an updated, improved forecast is issued. Re-analysis works in the same way, but at reduced resolution to allow for the provision of a dataset spanning back several decades. Re-analysis does not have the constraint of issuing timely forecasts, so there is more time to collect observations, and when going further back in time, to allow for the ingestion of improved versions of the original observations, which all benefits the quality of the re-analysis product. The data set presented here is a re-gridded subset of the full ERA5 data set on native resolution. It is online on spinning disk, which should ensure fast and easy access. It should satisfy the requirements for most common applications.

In the present study, the wave data during the past 33 years from 01.01.1990 to 16.11.2023 were considered in offshore region of Muthalapozhy. Wave data used were extracted along the model domain boundaries in the dfs1 format. The deep water wave data were transformed to get nearshore wave climate near the site of development using MIKE21 SW.

4. MODELLING TECHNIQUES

Mathematical model; MIKE21 SW, LITPACK, MIKE21 BW and MIKE-HD/MT were used for simulation of nearshore wave field, littoral drift/shoreline changes, wave disturbance, hydrodynamics and siltation pattern at the site and in the channel, respectively. Brief description of these model is given in the succeeding paragraphs.

4.1 MIKE21 SW model

As waves travel from deep sea to shallow coastal waters, they undergo changes in direction and height due to the processes of refraction and shoaling. The computation of wave transformation from deep to shallow coastal waters was carried out using MIKE21-SW model. MIKE21 Spectral Wave (SW) model is one of the state-of-the art third generation spectral wind wave models. The MIKE21 SW model simulates wave growth due to wind action, transformation due to refraction and shoaling resulting from depth variations, and decay due to white capping, bottom friction and wave breaking. The effects of wave-current interaction, non-linear wave-wave interaction, time-varying water depth and diffraction are also included within the model. The model is based on flexible mesh which allows for coarse spatial resolution for offshore area and high-resolution mesh in shallow water and at the coastline.

4.2 MIKE21 BW model

Mathematical model MIKE21 BW was used for studying the wave disturbance in the inlet region. The model is based on time dependant Boussinesq equations of conservation of mass and momentum obtained by integrating the three-dimensional flow equations without neglecting vertical acceleration. They operate in time domain, so that irregular waves can be simulated. These equations include nonlinearity as well as frequency dispersion. The frequency dispersion is included in flow equations by taking into account the effect of vertical acceleration or the curvature of stream lines on pressure distribution. The model simulates the processes of shoaling, refraction, diffraction from breakwater tips and bed friction. It also takes into account partial reflections from the boundaries, piers and breakwaters. This is done by including porosity terms in the governing equations. The governing equations are

Continuity Equation

$$n\frac{\partial \varsigma}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=0 \tag{1}$$

X Momentum Equation

$$n\frac{\partial p}{\partial t}+\frac{\partial}{\partial x}\left(\frac{p^2}{h}\right)+\frac{\partial}{\partial y}\left(\frac{pq}{h}\right)+n^2gh\frac{\partial \varsigma}{\partial x}+n^2p\left(\alpha+\beta\sqrt{\frac{p^2+q^2}{h^2}}\right)-\frac{p^2}{nh}\frac{\partial n}{\partial x}-\frac{pq}{nh}\frac{\partial n}{\partial y}$$

$$= n\frac{D^2}{3}\left(\frac{\partial^3 p}{\partial x^2\partial t}+\frac{\partial^3 q}{\partial x\partial y\partial t}\right) \tag{2}$$

Y Momentum Equation

$$n\frac{\partial q}{\partial t}+\frac{\partial}{\partial y}\left(\frac{q^2}{h}\right)+\frac{\partial}{\partial x}\left(\frac{pq}{h}\right)+n^2gh\frac{\partial \varsigma}{\partial y}+n^2q\left(\alpha+\beta\sqrt{\frac{p^2+q^2}{h^2}}\right)-\frac{q^2}{nh}\frac{\partial n}{\partial y}-\frac{pq}{nh}\frac{\partial n}{\partial x}$$

$$= n\frac{D^2}{3}\left(\frac{\partial^3 q}{\partial y^2\partial t}+\frac{\partial^3 p}{\partial x\partial y\partial t}\right) \quad (3)$$

Where,

ς (x,y,t) = water surface elevation above datum

p(x,y,t) = flux density in x direction

q(x,y,t) = flux density in y direction

D(x,y) = still water depth h(x,y,t) = water depth

n(x,y) = porosity g = gravity

x, y = space coordinates t = time

These differential equations are solved using a time-centered implicit finite difference scheme with variables defined on a space-staggered rectangular grid. Output of MIKE21-BW model is in the form of wave height distribution plot in the area of study. This plot gives the contours of wave heights in the study area.

4.3 LITPACK Model

LITPACK software was used for computation of littoral drift and simulation of shoreline changes due to construction of the breakwater. LITPACK is a professional engineering software package for modelling of non-cohesive sediment transport in waves and currents, littoral drift, coastline evolution and profile development along quasi-uniform beach. The LITDRIFT module simulates cross-shore distribution of wave height, setup and longshore current for an arbitrary coastal profile. It provides a detailed deterministic description of cross-shore distribution of longshore sediment transport for an arbitrary bathymetry for both regular and irregular sea conditions. The longshore and the cross-shore momentum balance equations are solved to give the cross-shore distribution of longshore current and setup. Wave decay due to breaking is modelled either by an empirical wave decay formula or by a model of Battjes and Janssen. LITDRIFT calculates net/gross littoral transport over a specific design period. Important factors, such as linking of the water level and the beach profile to the incident sea state, are included. Based upon the results from LITDRIFT, LITLINE simulates the coastal response to gradients in longshore sediment transport capacity resulting from natural features and a wide variety of coastal structures. LITLINE predicts the coastline evolution by solving a continuity equation for sediment in the littoral zone. The influence of structures, sources and sinks is included. With jetties and breakwaters, the influence of diffraction on wave climate is also included. LITPACK is a 1-D model. The flow around a breakwater comprises complicated 2-dimensional circulations, which cannot be fully assessed by any 1-D model. With respect to coastline evolution modelling, the prime effect of an offshore breakwater is its sheltering effect. In the shadow region between the offshore breakwater and the coastline, the wave disturbance and the driving forces for a long shore current are reduced (and may even reverse, depending on the relative size and proportions of the breakwater). This leads to a decrease in the longshore sediment transport. It is this effect that is modelled in LITLINE, and it is solely used to assess stability of local coastline through the changes to longshore transport rates. The annual sediment budget is found by the contribution of transport from each of wave incidents occurring during the year. Thus, the total annual drift is the sum of contributions from all incident waves.

4.4 MIKE 21HD Model

In order to simulate dynamics of cohesive sediment, it is necessary to initially compute hydrodynamics of water body in terms of velocity and water level fluctuations. Appropriate governing equations for hydrodynamics in tidal areas are given by shallow water wave equations. These two dimensional shallow water equations are derived from Navier Stokes equations of motion with the following simplified assumptions:

(i) The flow is incompressible

(ii) The flow is well mixed

(iii) Vertical accelerations are negligible

(iv) Bed stress can be modeled using a quadratic friction law.

The conservation of mass and momentum integrated over the vertical are used in hydrodynamic model to describe the flow and water level variations.

$$\frac{\partial H}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=\frac{\partial d}{\partial t} \tag{4}$$

$$\frac{\partial P}{\partial t}+\frac{\partial}{\partial x}\left(\frac{p^2}{h}\right)+\frac{\partial}{\partial y}\left(\frac{pq}{h}\right)+gh\frac{\partial H}{\partial x}+\frac{gp\sqrt{p^2+q^2}}{c^2h^2}$$

$$-\frac{1}{\rho_w}\left[\frac{\partial}{\partial x}(h\tau_{xx})+\frac{\partial}{\partial y}(h\tau_{xy})\right]-\Omega q-fVV_x+\frac{h}{\rho_w}\frac{\partial}{\partial x}(P_a)=0 \tag{5}$$

$$\frac{\partial q}{\partial t}+\frac{\partial}{\partial y}\left(\frac{q^2}{h}\right)+\frac{\partial}{\partial x}\left(\frac{pq}{h}\right)+gh\frac{\partial H}{\partial y}+\frac{gq\sqrt{p^2+q^2}}{c^2h^2}-\frac{1}{\rho_w}\left[\frac{\partial}{\partial y}(h\tau_{yy})+\frac{\partial}{\partial x}(h\tau_{xy})\right]+\Omega p-fVV_y$$

$$+\frac{h}{\rho_w}\frac{\partial}{\partial y}(P_a)=0 \tag{6}$$

The following symbols are used in the equations:

h(x, y, t)	:	water depth(m)
H(x, y, t)	:	surface elevation (m)

d(x, y, t)	:	time varying water depth (m)
p,q(x,y,t)	:	flux densities in x and y directions [$m^3/(s.m)$]
c(x,y)	:	Chezy resistance ($m^{1/2}/s$)
g	:	acceleration due to gravity (m/s^2)
f(v)	:	wind friction factor
v, v_x, $v_y(x,y,t)$	:	wind speed and components in x and y direction (m/s)
$\Omega(x,y)$	:	Coriolis parameter (s^{-1})
P_a	:	atmospheric pressure (kg/ms^2)
ρ_w	:	density of water (kg/m^3)
x, y	:	space coordinates (m)
t	:	time(s)
τ_{yy}, τ_{xy}, τ_{xx}	:	components of effective shear stress (N/m^2)

MIKE 21 Flow Model FM is based on a flexible mesh approach and it has been developed for applications within oceanographic, coastal and estuarine environments. The modelling system may also be applied for studies of overland flooding. The system is based on numerical solution of the two-dimensional incompressible Reynolds averaged Navier-Stokes equations invoking the assumptions of Boussinesq and of hydrostatic pressure. The spatial discretization of primitive equations is performed using a cell-centred finite volume method. The spatial domain is discretized by subdivision of the continuum into non-overlapping elements/cells. In horizontal plane, an unstructured grid is used while in vertical domain in 3D mode, a structured mesh is used. In 2D model, the elements can be triangles or quadrilateral elements. The spatial discretization of the primitive equations is performed using a cell-centred finite volume method. The typical mesh for 3-Dimesional model is given below.

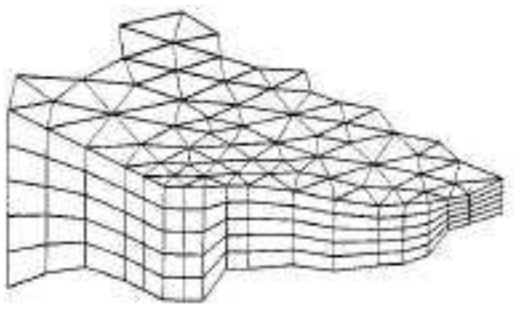

Fig.1 (A) 3-Dimensional model mesh

4.5 MIKE 21 MT Model

The sediment transport studies were carried out using MIKE 21 MT model. This model simultaneously solves hydrodynamic and sediment transport equations. The calibration of sediment transport model is difficult because morphological changes are too slow and temporal bed changes are too variable to measure anything significant for comparison. The sediment fluxes at various locations may differ and the following factors contribute for these variations:

- Unsteadiness of flow,
- Mixtures of sediment in suspension,
- Variability of supply of mobile sediment on the bed,
- Presence of sandy (non-cohesive) sediment,
- Omission of depth variation,
- Effect of wave stirring.

The erosion, transport and deposition of silt, mud and clay particles under the action of currents and waves can be best described by a multi-layer mode of mud transport module of MIKE 21. The sediment transport module is dynamically coupled with 2-dimensional hydrodynamic module, MIKE21 HD. The module solves the primitive equations in two dimensions using finite difference methods by Alternating Direction Implicit technique and the Double Sweep algorithm. The sediment transport formulations are built into the advection-dispersion module, MIKE 21 AD, which solves advection-dispersion equation:

$$\frac{\partial c}{\partial t}+u\frac{\partial c}{\partial x}+v\frac{\partial c}{\partial y}=\frac{1}{h}\frac{\partial}{\partial x}\left(hD_x\frac{\partial c}{\partial x}\right)+\frac{1}{h}\frac{\partial}{\partial y}\left(hD_y\frac{\partial c}{\partial y}\right)+Q_L C_L\frac{1}{h}-S \quad (7)$$

The following symbols are used in the equation:

c	:	compound concentration (arbitrary units)
u,v	:	horizontal velocity components in x, y directions (m/s)
h	:	water depth (m)
D_x,D_y	:	dispersion coefficients in the x,y directions (m^2/s)
S	:	accretion/erosion term (kg/m^3/s)
Q_L	:	source discharge per unit horizontal area (m^2/s/m^2)
C_L	:	concentration of source discharge (kg/m^3)

The advection-dispersion equation is solved using an explicit, third-order finite difference scheme known as the ULTIMATE scheme.

5. COMPUTED WAVE DATA

5.1 Offshore wave climate

The offshore wave data is given in Table 1 and its rose diagram is shown in Fig.2. It indicates that mean waves ranging from 1.0 m to 3.0 m with mean angles west to south directions mostly present at offshore of the site. It is seen that the percentage variation of most of these waves is between 1.4% and 42.3%.

Table 1. Percentage occurrence of wave height & direction at Muthalapozhy in offshore region for entire period (Jan-Dec)

Significant Wave Height (m)	**Mean Wave Direction**								
	SE	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**TOTAL**
0.0-0.5	<0.1%	<0.1%	<0.1%	<0.1%	<0.1%				0.2%
0.5-1.0	<0.7%	10.8%	22.1%	5.7%	1.6%	0.7%	0.4%	<0.1%	42.3%
1.0-1.5	<0.1%	1.2%	8.7%	8.1%	5.1%	2.9%	0.9%	<0.1%	27.0%
1.5-2.0	<0.1%	<0.1%	0.7%	3.1%	5.9%	7.7%	3.2%	<0.1%	20.7%
0.2-2.5	<0.1%	<0.1%	<0.1%	0.4%	1.0%	3.8%	2.6%	<0.1%	8.5%
2.5-3.0	<0.1%	<0.1%	<0.1%	<0.1%	10.0%	0.6%	0.6%	<0.1%	1.4%
3.0-3.5	<0.1%	<0.1%	<0.1%	<0.1%	<0.1%	<0.1%	<0.1%	<0.1%	0.1%
Total	**0.7%**	**12.0%**	**31.7%**	**17.3%**	**14.3%**	**15.7%**	**7.8%**	**0.3%**	**100%**

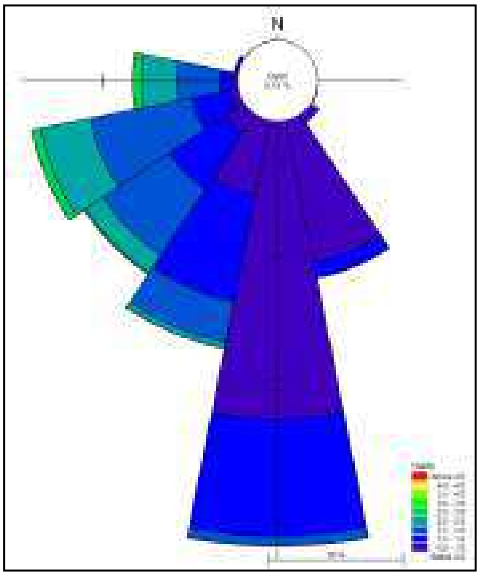

Fig.2 Offshore wave rose diagram

5.2 Nearshore wave climate for Littoral drift studies

For littoral drift distribution and shoreline evolution studies, the offshore wave climate was transformed to a nearshore location at -15m depth contour near the site. The nearshore wave data distribution is given in Table 2 and its wave rose diagram is shown in Fig.3. The nearshore wave data indicated that waves having height 1.5m to 2.5m with predominant directions from SSW, SW and WSW were prevailing at the site. It is seen that the percentage variation of these waves is between 2.9% and 21.8%. The littoral drift model under the existing and the proposed conditions were simulated for the nearshore wave climate enumerated in Table 3.

Table 2 Percentage occurrence of wave height & direction at Muthalapozhy in (-)15m depth for entire period (Jan-Dec)

	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	
0.00 - 0.50 m									0.19%	29.92%	0.25%	< 0.01%	0.04%	< 0.01%	< 0.01%			30.42%
0.50 - 1.00 m										19.02%	7.17%	1.27%	0.51%	0.03%				28.00%
1.00 - 1.50 m										2.74%	11.61%	6.83%	0.62%	< 0.01%				21.81%
1.50 - 2.00 m										0.24%	5.45%	10.57%	0.45%					16.70%
2.00 - 2.50 m										< 0.01%	0.69%	2.19%	0.04%					2.92%
2.50 - 3.00 m											0.04%	0.10%	< 0.01%					0.14%
3.00 - 3.50 m												< 0.01%						0.00%
3.50 - 4.00 m																		0.00%
> 4.00 m																		0.00%
Total	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.19%	51.92%	25.21%	20.97%	1.66%	0.03%	0.00%	0.00%	0.00%	100.00%

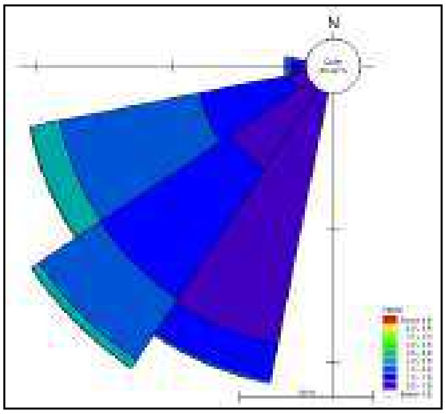

Fig.3 Nearshore wave rose diagram for Littoral Drift studies

Table 3 Significant nearshore wave climate for Littoral drift studies

Wave direction (^{0}N)		Peak Wave period Tp (sec.)	Significant Wave height (m)
202.5	SSW	11	1.5
225	SW	11	2.0
247.5	WSW	11	2.5

5.3 Nearshore wave climate for wave propagation studies

For wave propagation studies, the offshore wave climate was transformed to a nearshore location at -20m depth contour near the site. The wave climate distribution is given in Table 4 and its rose diagram is shown in Fig.4. The nearshore wave data indicated that incident waves having significant heights 1.5m to 2.5m and predominant directions from SSW, SW, WSW and West were present. It is seen that the percentage variation of most of these waves is between 17.3% and 29.2%. The peak wave period for all the incident waves was calculated as 11s. The nearshore wave climates at -20m depth contour are enumerated in Table 5. The wave propagation model under the existing and the proposed conditions were simulated for the mentioned above nearshore wave climate.

Table 4 Percentage occurrence of nearshore wave height & direction at -20m depth at Muthalapozhy for entire period (Jan-Dec)

Significant Wave Height(m)	Mean Wave Direction						
	S	**SSW**	**SW**	**WSW**	**W**	**WNW**	**TOTAL**
0.0-0.5	0.5%	28.5%	0.1%	<0.1%	<0.1%	<0.1%	29.2%
0.5-1.0	<0.1%	21.9%	5.4%	1.1%	0.6%	<0.1%	29.1%
1.0-1.5	<0.1%	3.9%	10.5%	6.1%	0.9%	<0.1%	21.3%
1.5-2.0	<0.1%	0.4%	5.3%	10.4%	1.1%	<0.1%	17.3%
0.2-2.5	<0.1%	<0.1%	0.6%	2.3%	0.1%	<0.1%	3.0%
2.5-3.0	<0.1%	<0.1%	<0.1%	0.1%	<0.1%	<0.1%	0.2%
Total	**0.5%**	**54.7%**	**21.9%**	**20.1%**	**2.7%**	**0.1%**	**100%**

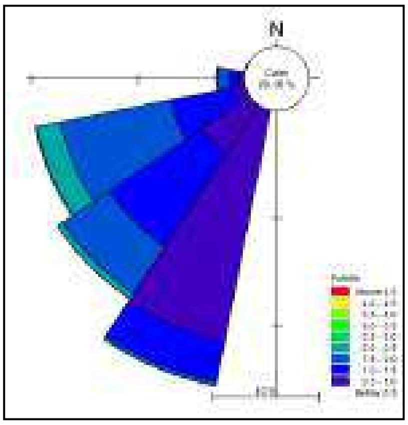

Fig.4 Nearshore wave rose diagram for Wave propagation studies

Table 5. Significant wave climates for Wave tranquility studies

Wave direction (^{0}N)		Peak Wave period Tp (sec)	Significant Wave height (m)
202.5	SSW	11	1.5
225	SW	11	2.0
247.5	WSW	11	2.5
270	West	11	2.0

6. EXISTING CONDITION

6.1 Littoral drift computation & Shoreline changes under Existing Condition

In the absence of a known magnitude of littoral transport prevailing at the site it's quantity is computed by using back calculation from the shift of shoreline occurred in the past in certain period of time. For this method generally satellite imageries or images from Google Earth is used. In this study Google Earth images are used. As per Google Earth image, it was observed that in the last ten years the shoreline near the inlet of Muthalapozhy was shifted to about 300m as indicated in Fig. 5. The LITLINE model was simulated for using the trial and error method to get the shoreline shift of about 300m during the 10 years period under various littoral drift quantities. After a few iterations it was observed that with the net littoral drift of the order of 0.24MCM in northward direction satisfied the criteria of shoreline advancement. The gross littoral drift is about 0.9MCM. The shift of shoreline computed for 10 years period is shown in Fig. 6(A-B).

Fig.5 Present position of shoreline with shifting in 10 years
(Courtesy: Google Earth)

Fig.6(A) Model output in terms of shoreline after 10 years of simulation

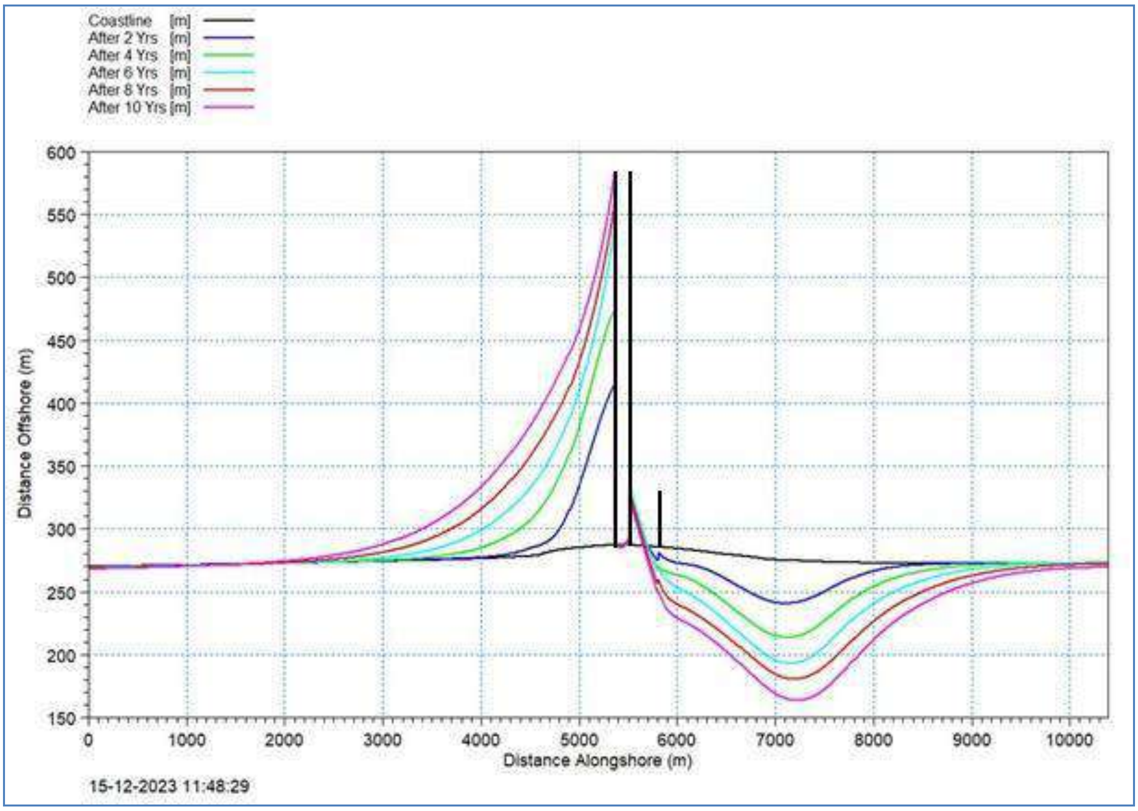

Fig.6(B) Model output showing change in shoreline after 10 years of simulation

The littoral drift distribution is shown in Fig.7. From the figure it is seen that littoral drift in both the directions prevailing at the site. As the northward littoral drift is more than the southward drift causing net drift towards north direction.

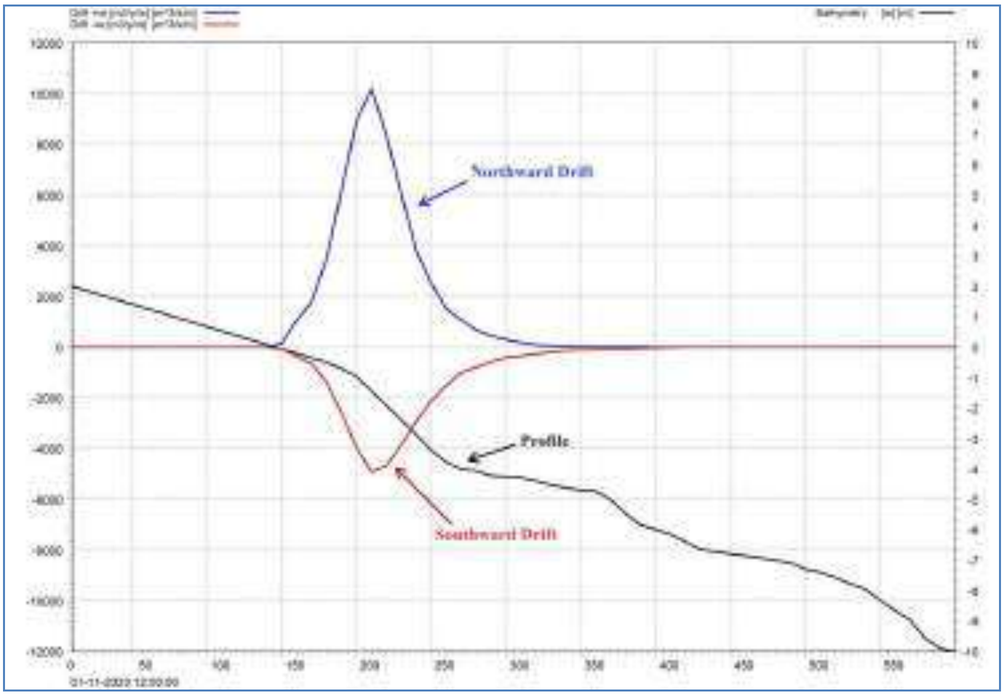

Fig.7 Littoral drift distribution at Muthalapozhy inlet site

From the figure, it is further seen that maximum drift occurs at -2m depth contour. However, 75% of drift could be traced upto -3.5m depth contour. Further, it can also be observed that littoral zone at the site is upto -5m depth contour and it exist upto about 350m distance from the seashore. The net and gross littoral transport estimated for at Muthalapozhy site is depicted in Table 6.

Table 6. Littoral transport at Muthalapozhy site (in MCM)

Northward	Southward	Net (Northward)	Gross
0.5707	0.3310	0.2397	0.9017

6.2 Wave propagation and tranquility under Existing Condition

The model simulations were carried out initially for the existing condition. In model bathymetry, the position and the length of breakwaters, northern groyne and the inlet mouth existing at the site were incorporated. The opening or cut made in the southern breakwater for approaching to Adani jetty was also incorporated in the model bathymetry. The bathymetry for wave tranquility model under the existing condition is shown in Fig.8.

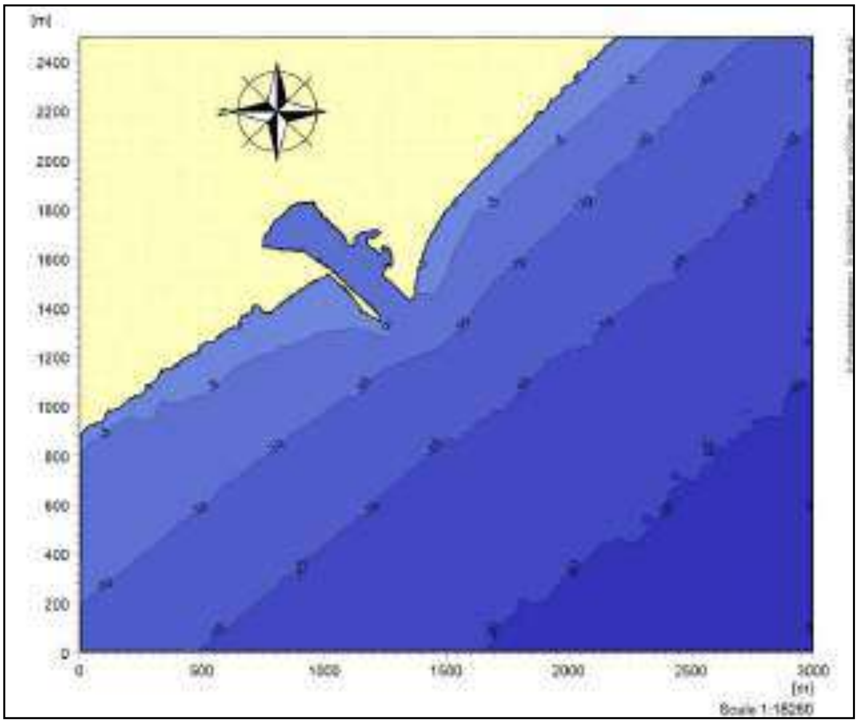

Fig.8 Bathymetry for wave tranquility model under existing condition

Wave tranquility model under the existing condition was simulated for all four predominant wave heights and directions (see Fig.4 & Table 5.) transformed from offshore locations to a nearshore location at -20m depth contours near the site. The simulated results were obtained in the form of wave propagation and its height distribution for the predominant wave climates. The model plots for each predominant wave climate are shown in Fig.9 (A-D).

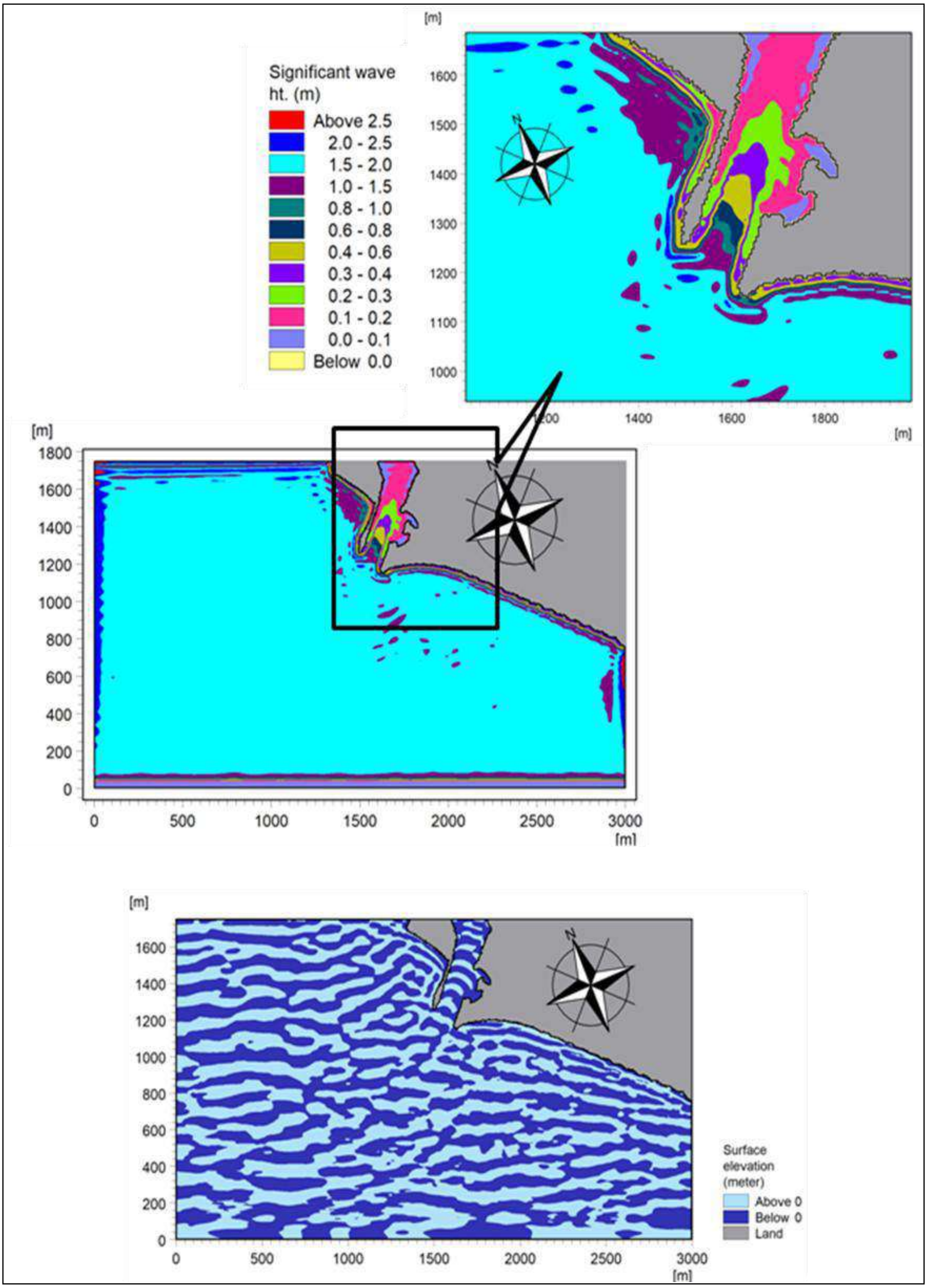

Fig.9(A) Wave height distribution and wave propagation plots from SSW (202.5^{0}N) direction and incident wave height 1.5 m

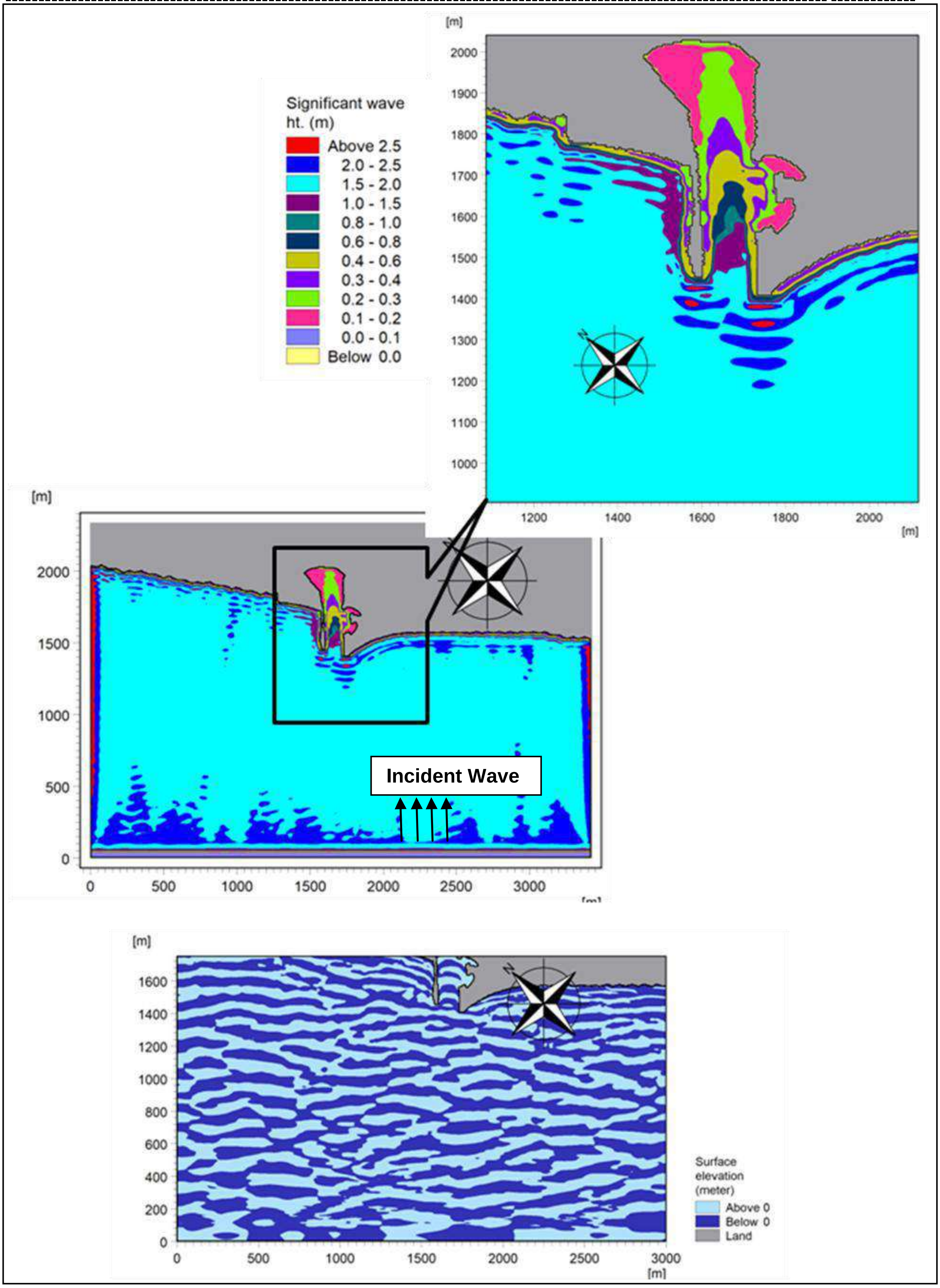

Fig.9(B) Wave height distribution and wave propagation plots from SW (225^{0}N) direction and Incident wave height 2m

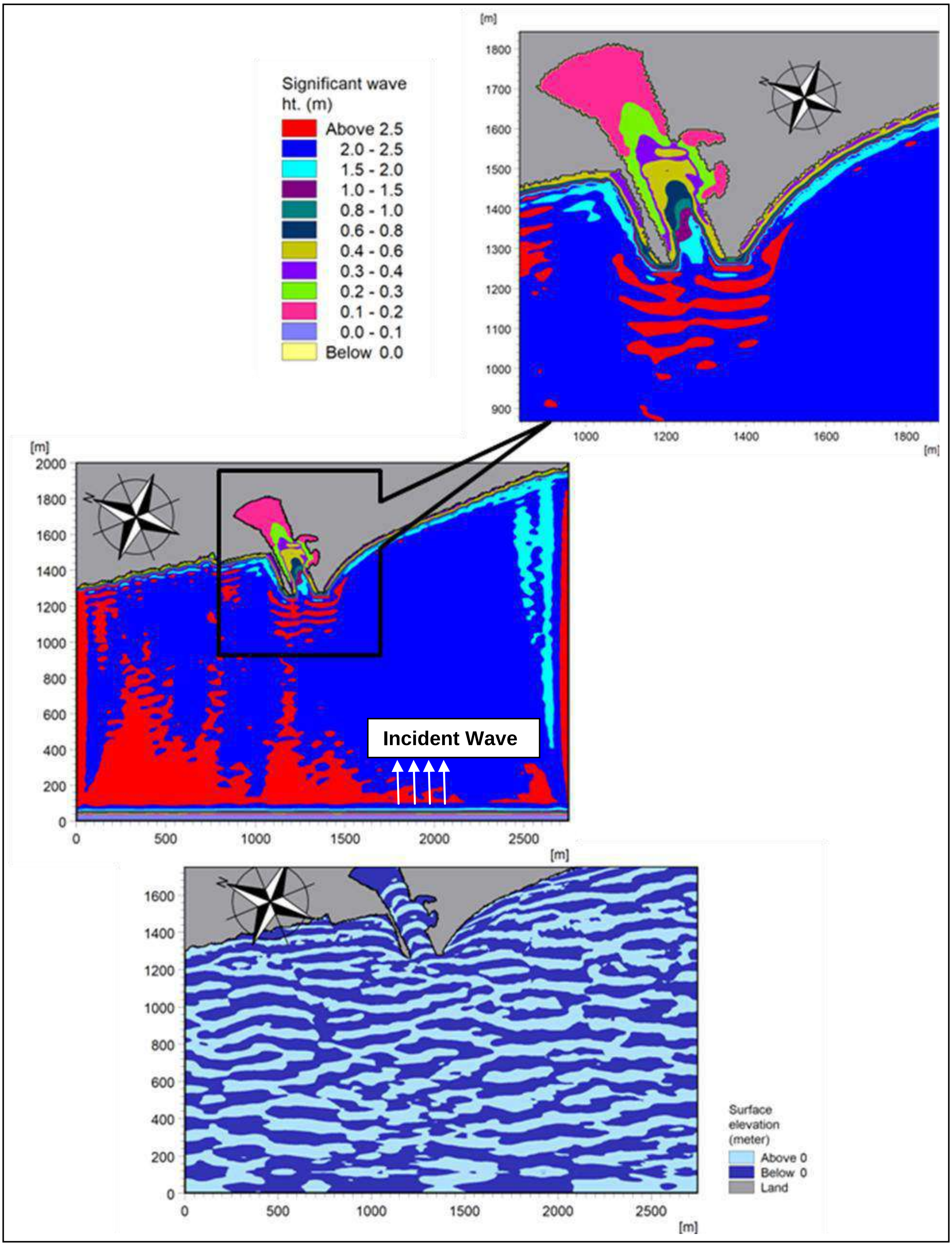

Fig.9(C) Wave height distribution and wave propagation plots from WSW (247.5^{0}N) direction and incident wave height 2.5m

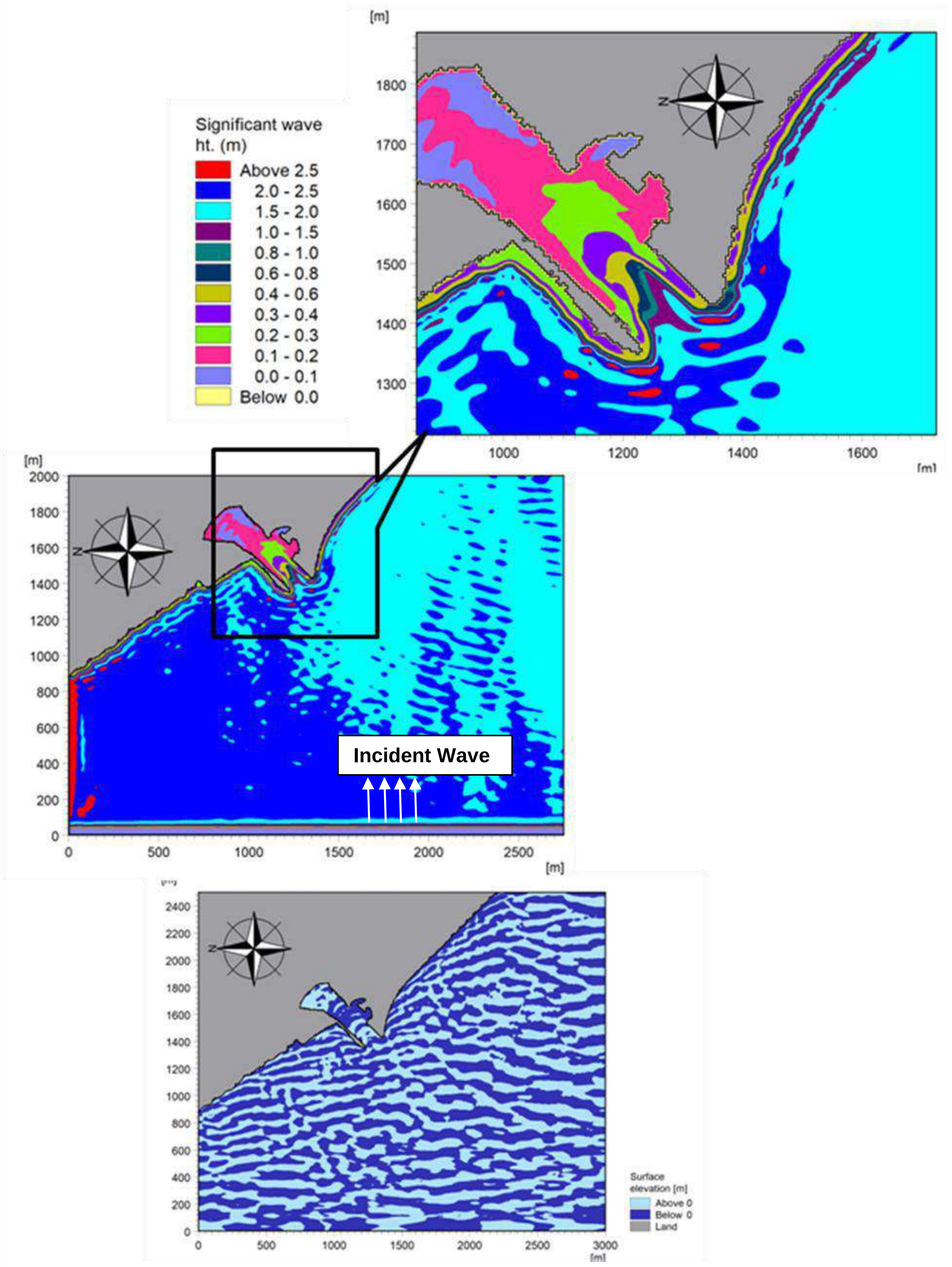

Fig.9(D) Wave height distribution and wave propagation plots from West (270^{0}N) direction and Incident wave height 2m

In the model bathymetry it is seen that the orientation of the inlet mouth in the existing condition is in the south-west direction. Therefore the incident waves from SSW, SW, WSW and West could able to enter directly into the mouth of the channel and waves having height more than 1.5m were observed in the simulation results as shown in Figs.(9A-9D) making it difficult for manoeuvring of fisheries boats.

7. MODEL SIMULATIONS FOR PROPOSED CONDITION

7.1 Prediction of shoreline evolution for proposed condition

Based on the calibrated littoral drift quantity, the change in shoreline was computed for the next 30years period. The computation was made under the estimated littoral transport and the nearshore annual wave climate (pl. see Fig.3 & Table 3) prevailing at the site. The model output of probable changes in shoreline is shown in Fig.10.

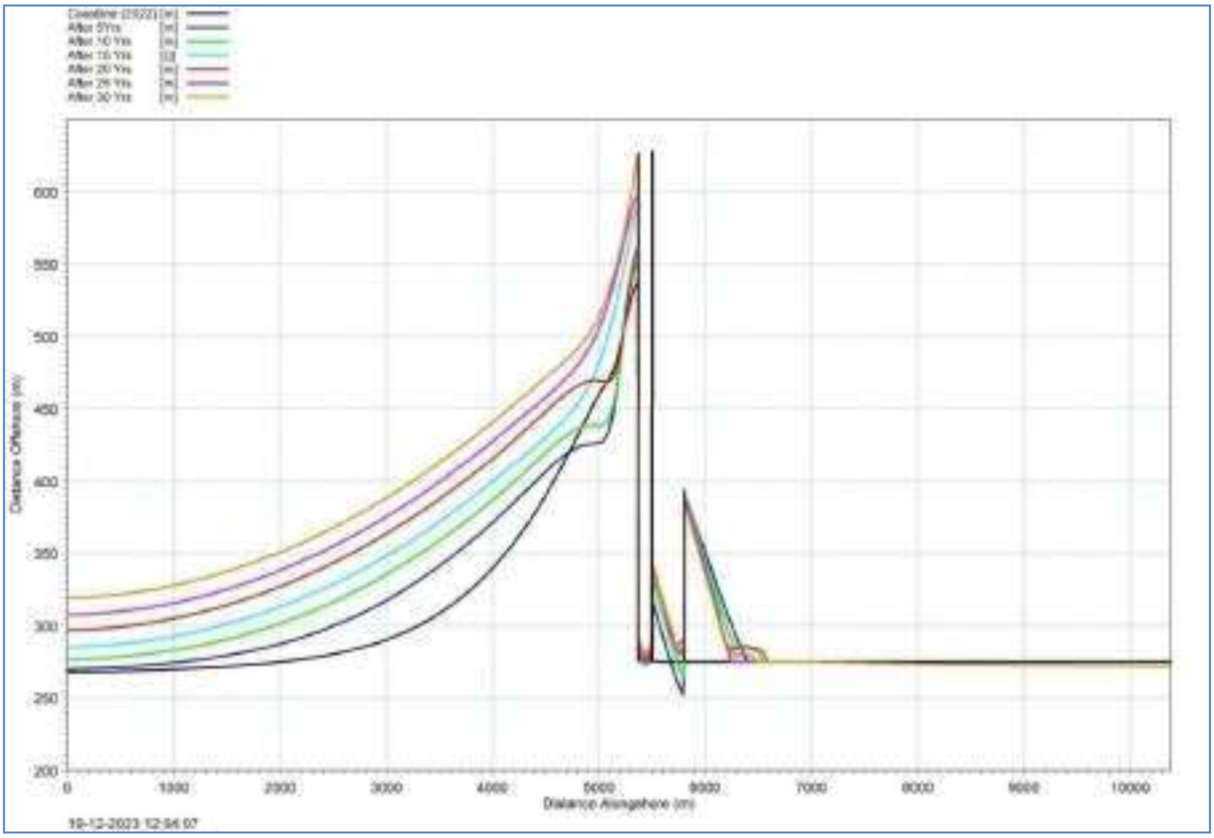

Fig.10 Model output showing changes in shoreline in next 30 years

From the figure, it is observed that in the next 30 years period the shoreline near the inlet mouth would be shifting to a distance of about 85m towards sea from its present position under the estimated littoral drift prevailing at the site. Based on the model results, it may be inferred that sand bypassing from south to north would not be required in next 30 years period from the present position. It is also seen that erosion would take place in northern side of the inlet mouth due to net northward littoral transport. Further, the incident wave falling normal to the coastline would cause erosion to the shoreline. In order to mitigate the probable effect of erosion suitable shore protection measures are required to be taken along the northern side of the inlet mouth.

7.2 Initial Proposed Layouts (IPLs)

The model simulation results for littoral transport and wave propagation indicated that the length of the existing southern breakwater should be increased to control the transfer of littoral drift and the orientation of inlet mouth should be towards north-east to restrict the propagation of higher waves inside the channel. Keeping these aspects in view, four preliminary layouts were proposed by the Project Authority. The initial proposed layouts (IPL-1 to IPL-4) incorporated in model bathymetry are shown in Fig. 11(A-D).

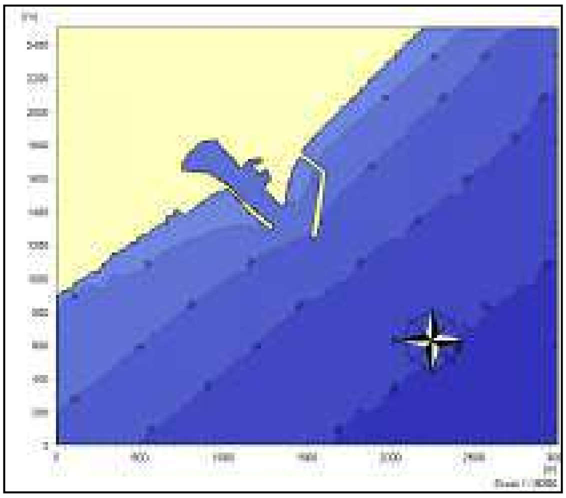

Fig.11(A) Initial Proposed Layout-1 (IPL-1)

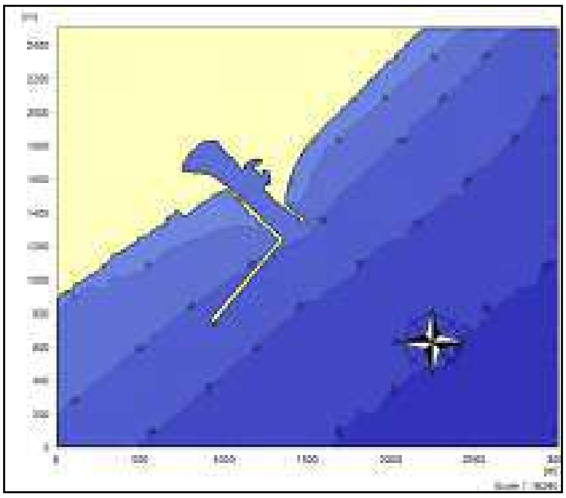

Fig.11(B) Initial Proposed Layout-2 (IPL-2)

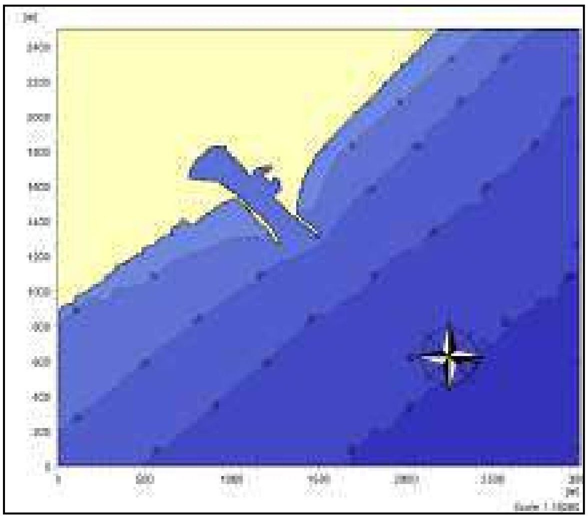

Fig.11(C) Initial Proposed Layout-3 (IPL-3)

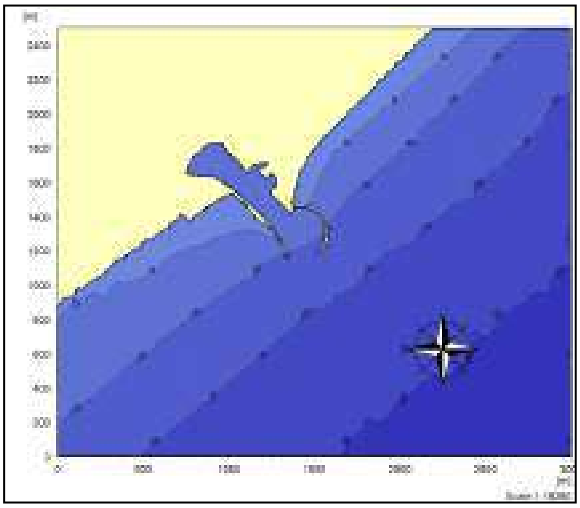

Fig.11(D) Initial Proposed Layout-4 (IPL-4)

All the initial proposed layouts (i.e. IPL-1, IPL-2, IPL-3 & IPL-4) were simulated for wave propagation studies. Each layout model incorporated in the bathymetry was simulated under the nearshore wave climate (pl. see Fig.4 & Table 5). The simulated results for each initial proposed layout under all the four incident prominent wave conditions were obtained as depicted in Figs.12(A-D).

(A) Incident wave height 1.5m with SSW (angle 202.5^0)

In Fig.12(A), the model results for IPL-1 under the influence of incident wave condition (1.5m height with SSW direction) is shown. From the figure it is seen that the wave having height of the order of 1.0-1.5m was present at the entrance of the channel creating unfavorable conditions for maneuvering of the fishing boats.

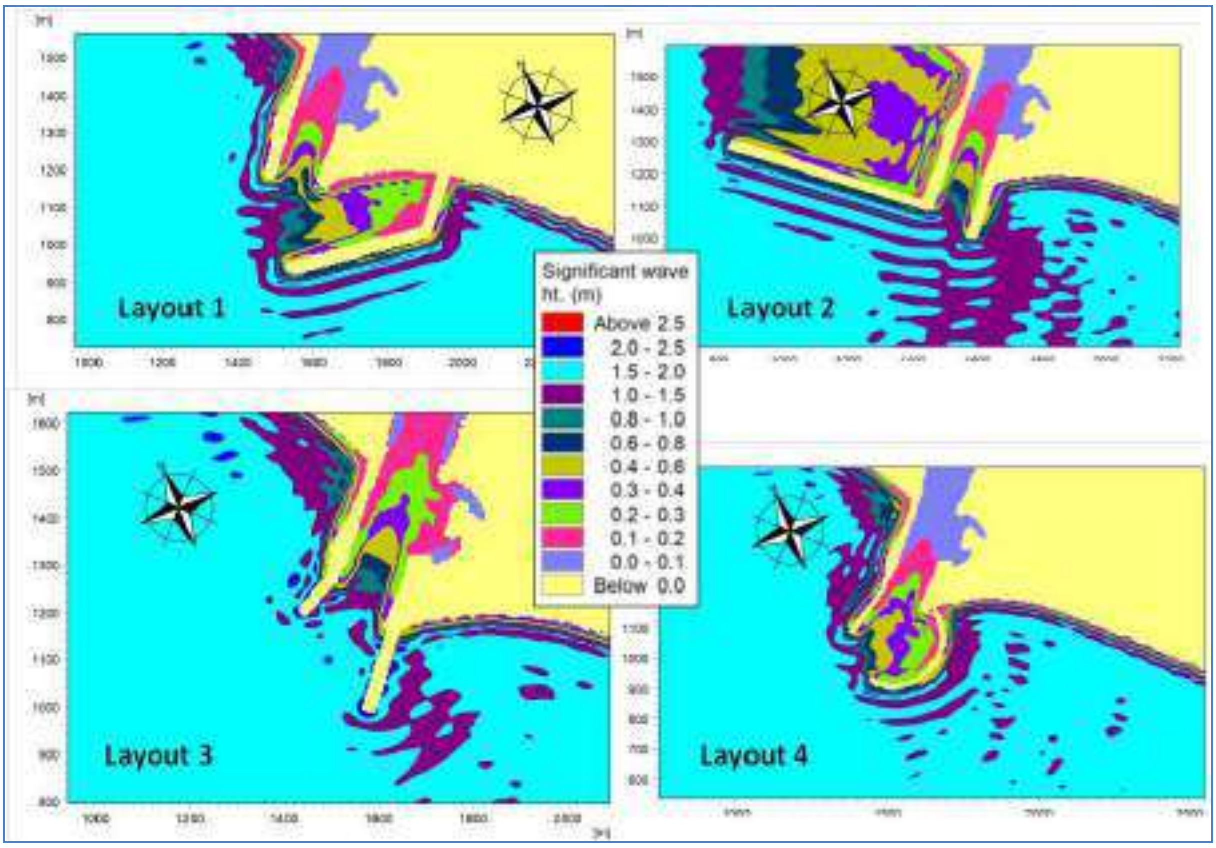

Fig.12 (A). Wave height distribution plot for waves incident from SSW (202.5^0) direction (Incident wave height: 1.5m)

(B) Incident wave height 2m with SW (angle 225^0)

In Fig.12(B), the model results for IPL-1 under the influence of incident wave condition (2.0m height with SW direction) is shown. From the figure it is seen that the wave having height of the order of 1.0-1.5m was present at the entrance of the channel creating unfavorable conditions for manoeuvring of the fishing boats.

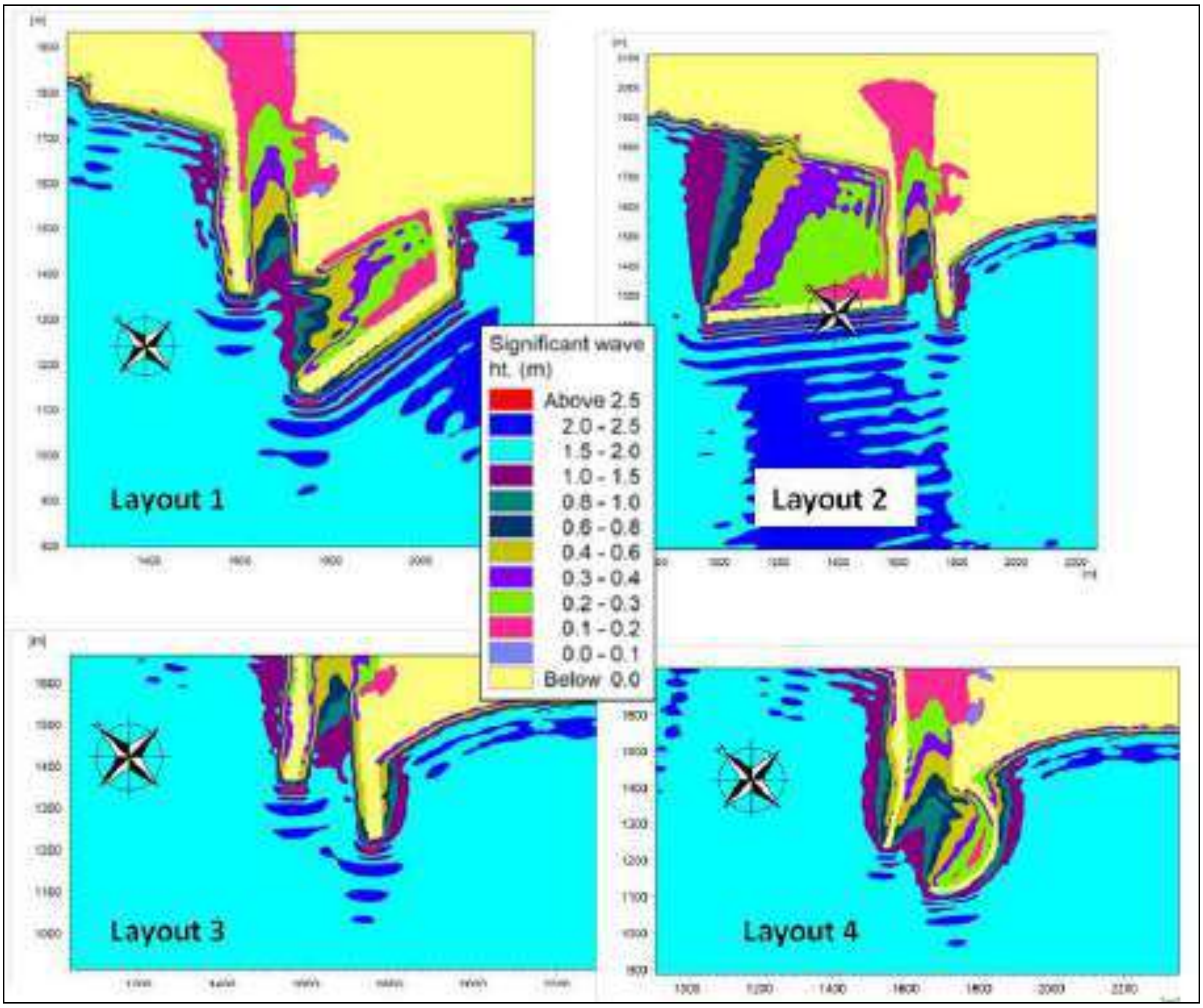

Fig. 12 (B). Wave height distribution plot for waves incident from SW (225^0) direction (Incident wave height: 2m)

(C) Incident wave height 2.5m with WSW (angle 247.5^0)

In Fig.12(C), the model results for IPL-1 under the influence of incident wave condition (2.5m height with WSW direction) is shown. From the figure it is seen that the wave having height of the order of 1.5-2.0m was present at the entrance of the channel creating unfavorable conditions for maneuvering of the fishing boats.

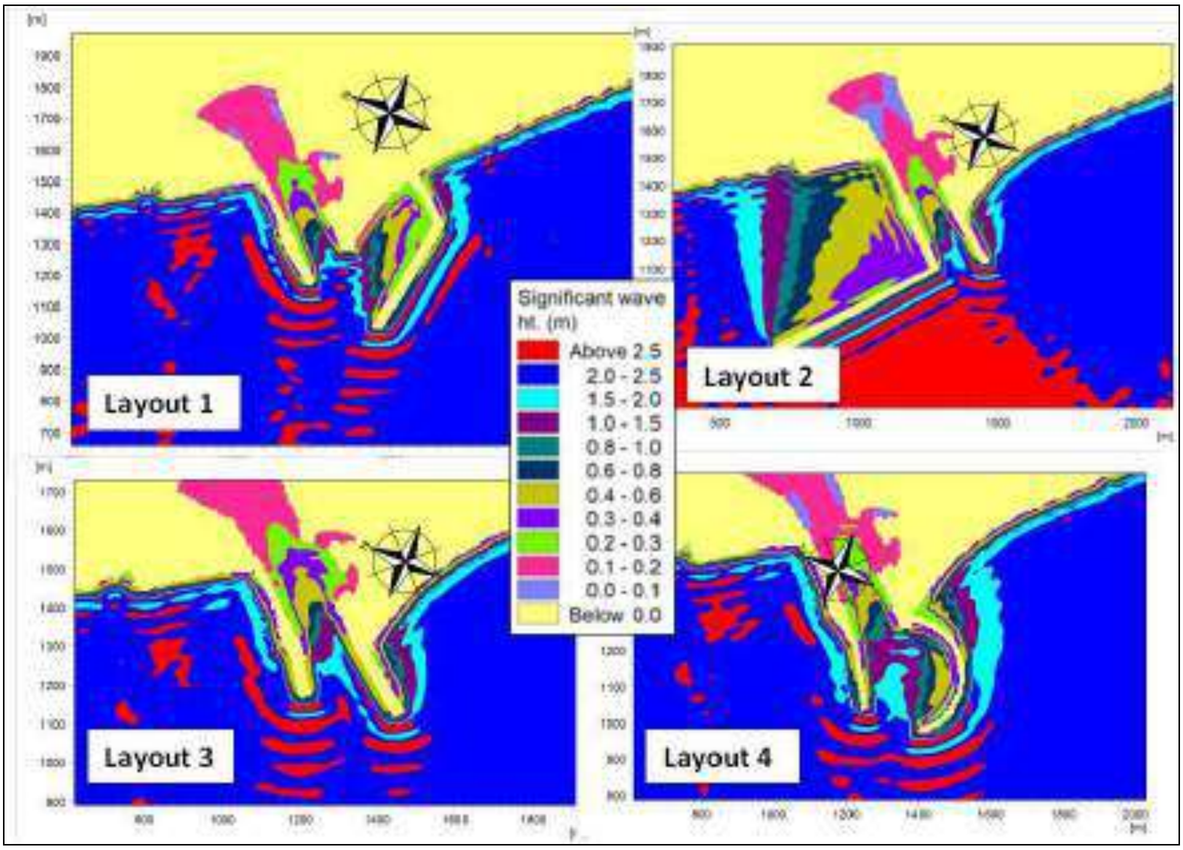

Fig.12 (C). Wave height distribution plot for waves incident from WSW (247.5^0) direction (Incident wave height: 2.5m)

(D) Incident wave height 2m with W (angle 270^0)

In Fig.12(D), the model results for IPL-1 under the influence of incident wave condition (2.0m height with W direction) is shown. From the figure it is seen that the wave having height of the order of 1.75m was present at the entrance of the channel creating unfavorable conditions for maneuvering of the fishing boats.

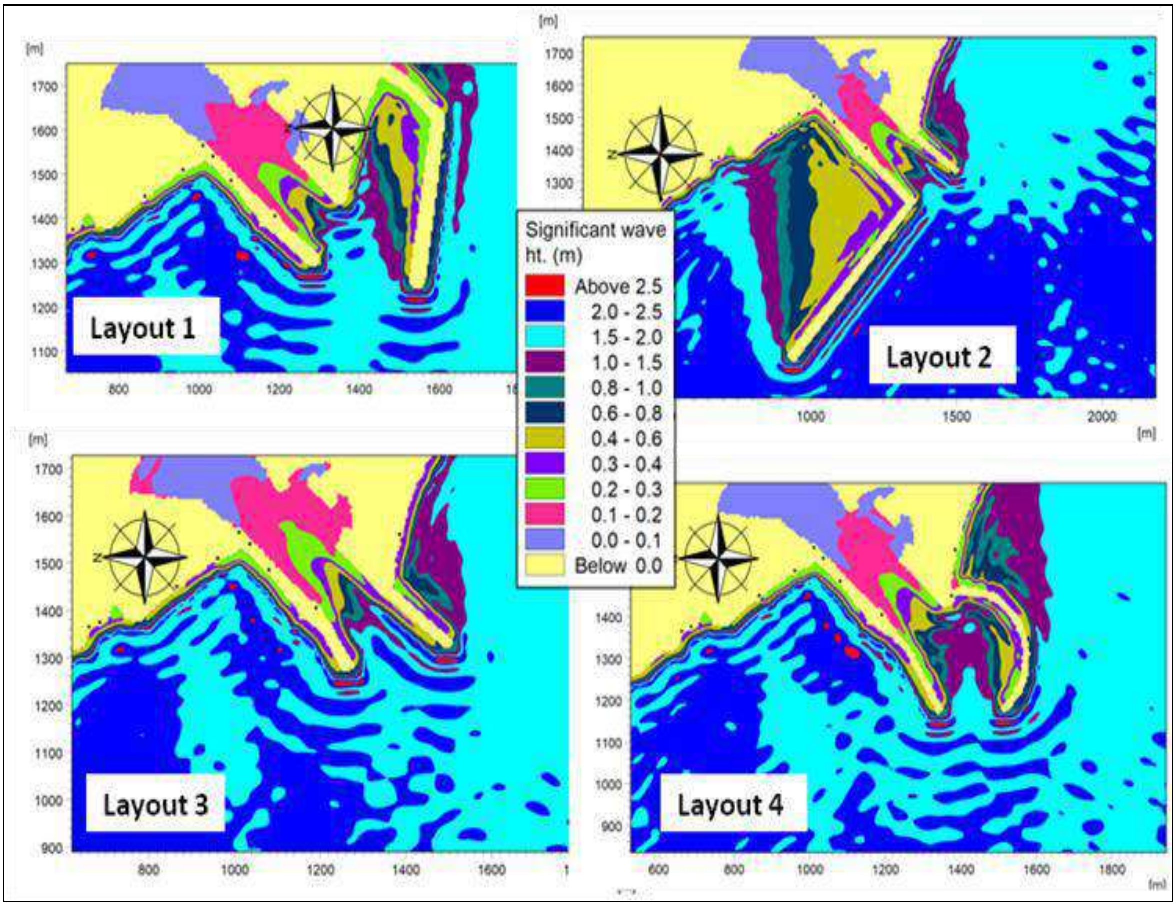

Fig.12 (D) Wave height distribution plot for waves incident from West (270^0) direction (Incident wave height: 2m)

Based on the model results obtained from wave propagation studies, it is observed that in all initial proposed layouts (IPL-1 to IPL-4), the wave height at the entrance was found to be more than 1.0m indicating the required tranquil condition at the entrance of the inlet channel could not be achieved for the nearshore wave climate prevailing at the site. Based on the model results, it is observed that the required tranquil condition at the entrance could be achieved with the overlapping of breakwaters length. In order to fulfill such conditions, the proposed layout was modified with curved southern breakwater and overlapped it with the existing northern breakwater. The modified layout under this consideration was called hereafter as Modified Proposed Layout (MPL)

7.3 Modified Proposed Layout (MPL)

The Southern Breakwater of the Modified Proposed Layout (MPL) was proposed to be extended from a distance of about 122m from south of the existing Southern Breakwater. The total length was proposed to be about 580m. The layout MPL is shown in Fig.13. From the figure it is seen that the tip of the extended proposed Southern Breakwater would be at -10 m depth contour.

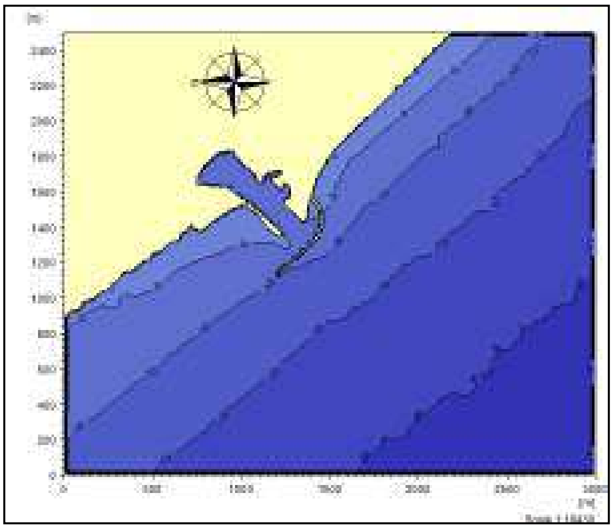

Fig.13 Model Bathymetry for Modified Proposed Layout (MPL)

The model for MPL was simulated under all the nearshore wave climate already enumerated in Table 5. The simulated results are shown in Figs.14(A-D) and discussions are made as under.

(i) Incident wave 1.5m in SSW (202.5^0) direction

In Fig.14(A), the model results for MPL under the influence of incident wave condition (1.5m height with SSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.8m exists which does not satisfy the required criteria of maneuvering of fishing boats. Also, at the outside of the entrance waves having height of the order of 0.8-1.0m are present.

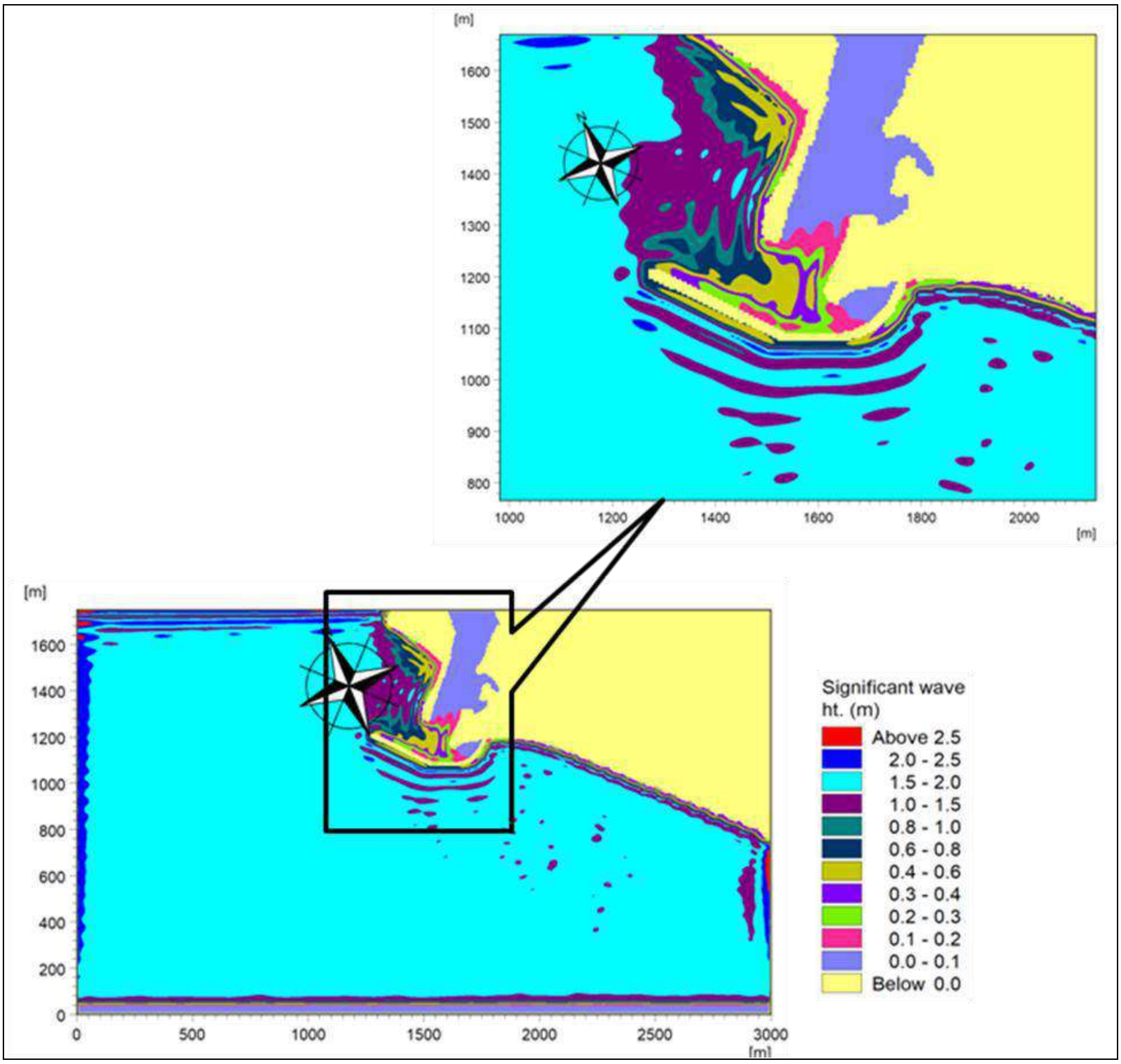

Fig. 14(A). Wave height distribution plot for waves incident from SSW (202.5^0) direction (Incident wave height: 1.5m)

(ii) Incident wave 2.0m in SW (225^0) direction

In Fig.14(B), the model results for MPL under the influence of incident wave condition (2.0m height with SW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.6m exist which satisfies the required criteria. However, at the outside of the entrance waves having height of the order of 0.8m are present.

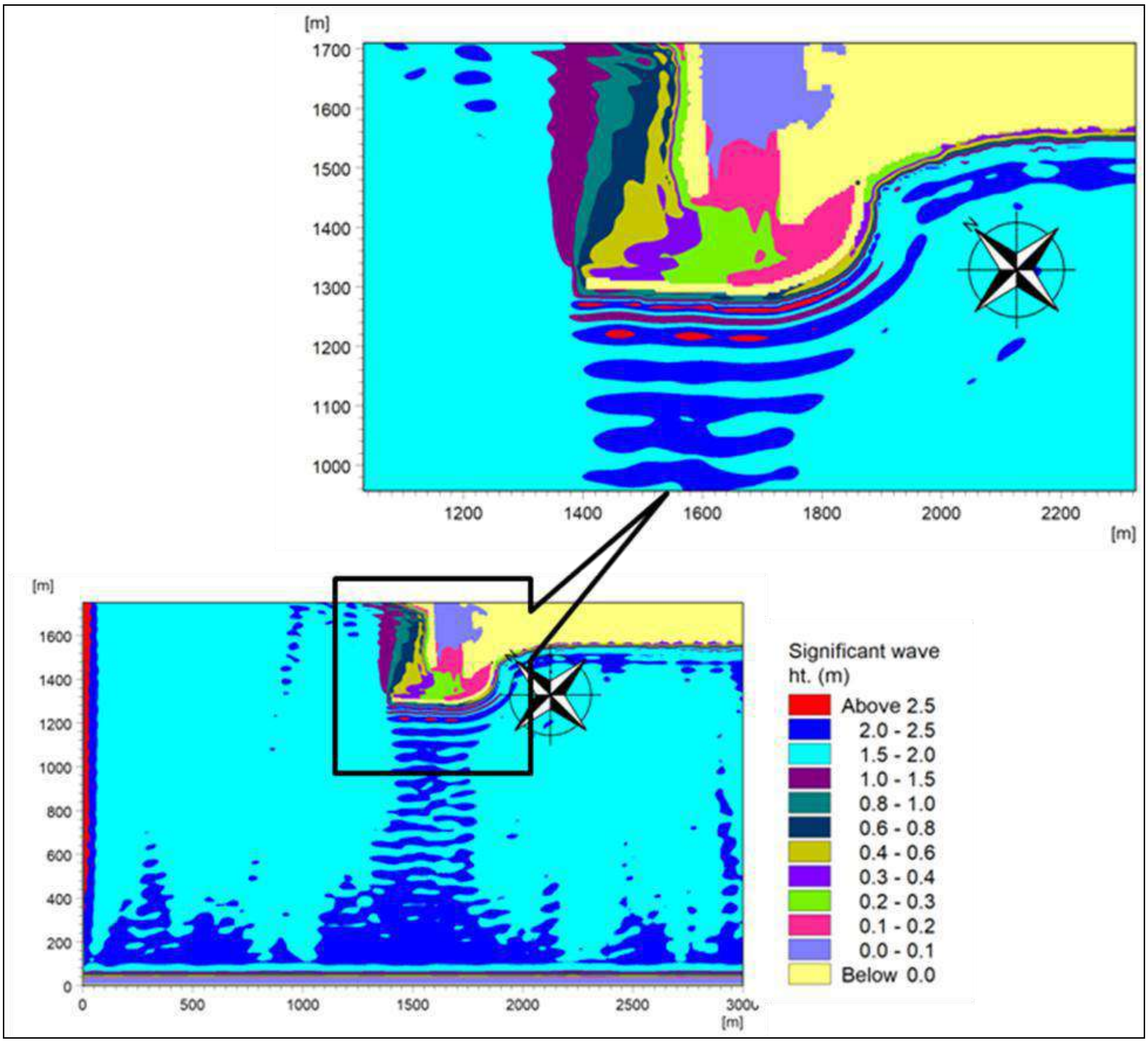

Fig. 14(B). Wave height distribution plot for waves incident from SW (225^0) direction
(Incident wave height: 2m)

(iii) Incident wave 2.5m in WSW (247.5^0) direction

In Fig.14(C), the model results for MPL under the influence of incident wave condition (2.5m height with WSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.6m exist which satisfies the required criteria. However, at the outside of the entrance waves having height of the order of 0.8m are present.

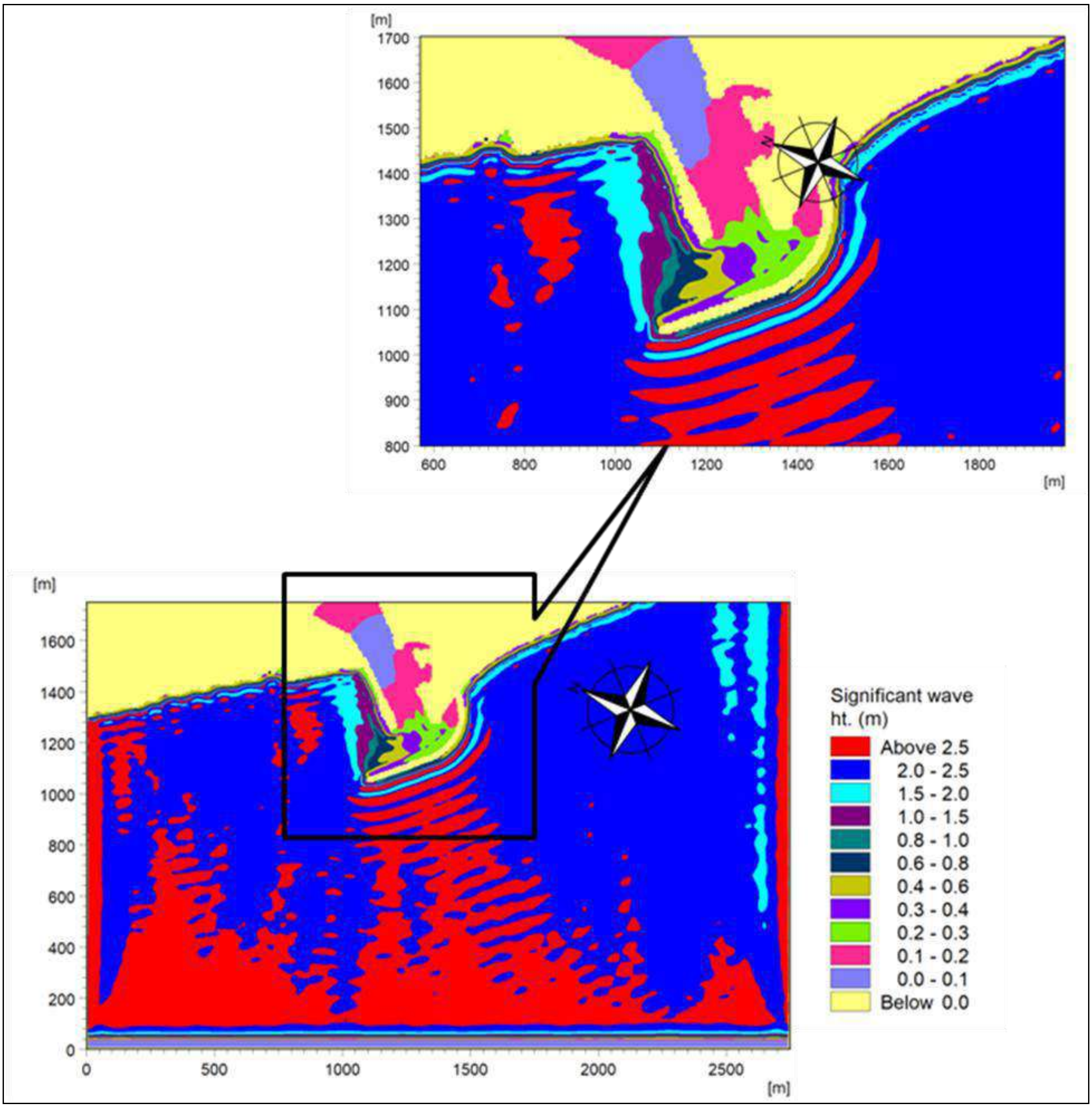

Fig. 14(C). Wave height distribution plot for waves incident from WSW (247.5^0) direction (Incident wave height: 2.5m)

(iv) Incident wave 2.0m in W (270^0) direction

In Fig.14(D), the model results for MPL under the influence of incident wave condition (2.0m height with W direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.6m exist which satisfies the required criteria. However, at the outside of the entrance waves having height of the order of 0.8m are present.

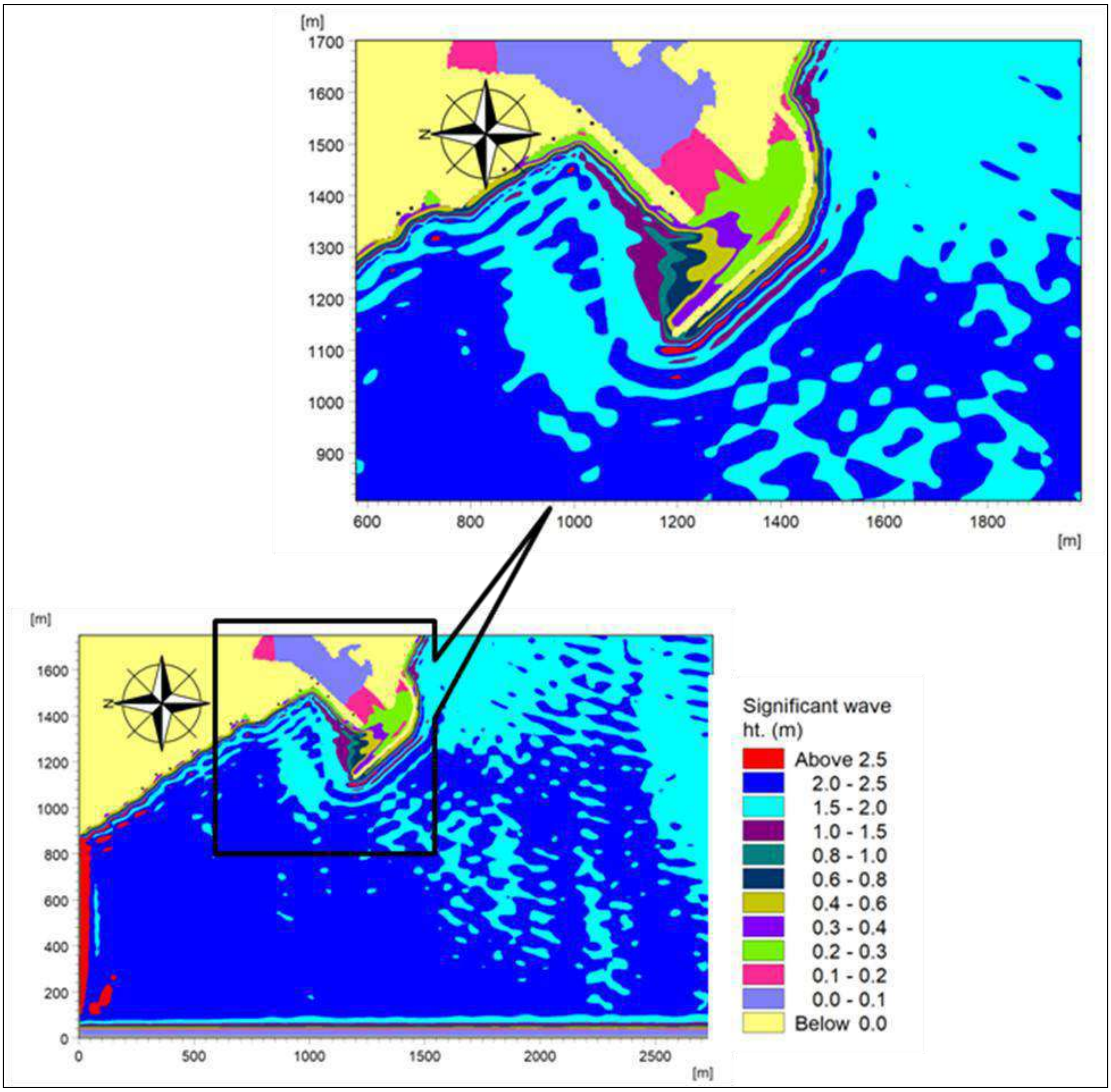

Fig.14(D). Wave height distribution plot for waves incident from West (270^0) direction (Incident wave height: 2m)

Based on the above results and discussions it can be observed that the modified proposed layout (MPL) satisfies the wave tranquility criteria at the entrance of the inlet. However, it is required to be further optimized based on model studies for wave tranquility, hydrodynamics and sediment transport. The results and discussions thereof are based on the model output obtained as per the prevailing conditions at the site and the field data observed and reported thereafter.

7.4 Optimized Proposed Layout (OPL)

A draft report was submitted to the Project Authority (PA) based on the model results obtained from wave propagation and littoral transport studies. In the draft report, the model results were discussed in details for the layouts under IPLs and MPL. Based on the comments of the PA received on the draft report vide letter no. HECE/240/2022/DR8 dated 05/01/2024, the modified proposed layout (MPL) was further optimized. The MPL was optimized as under.

The Load out Facility area (i.e. the cut in the southern breakwater) was closed. Also, the Southern Breakwater in the Optimized Proposed Layout (OPL) was proposed to be started from the tip of its existing position and to be extended upto about 420 m. Due to the proposed extension the tip of the Southern Breakwater under OPL would be at about -10.0m depth contour. Further, in the OPL the free end or the tip of the southern breakwater was bent outward to facilitate wider width at the entrance for the fishing boats. The model bathymetry incorporated with the optimized proposed layout is shown in Fig. 15.

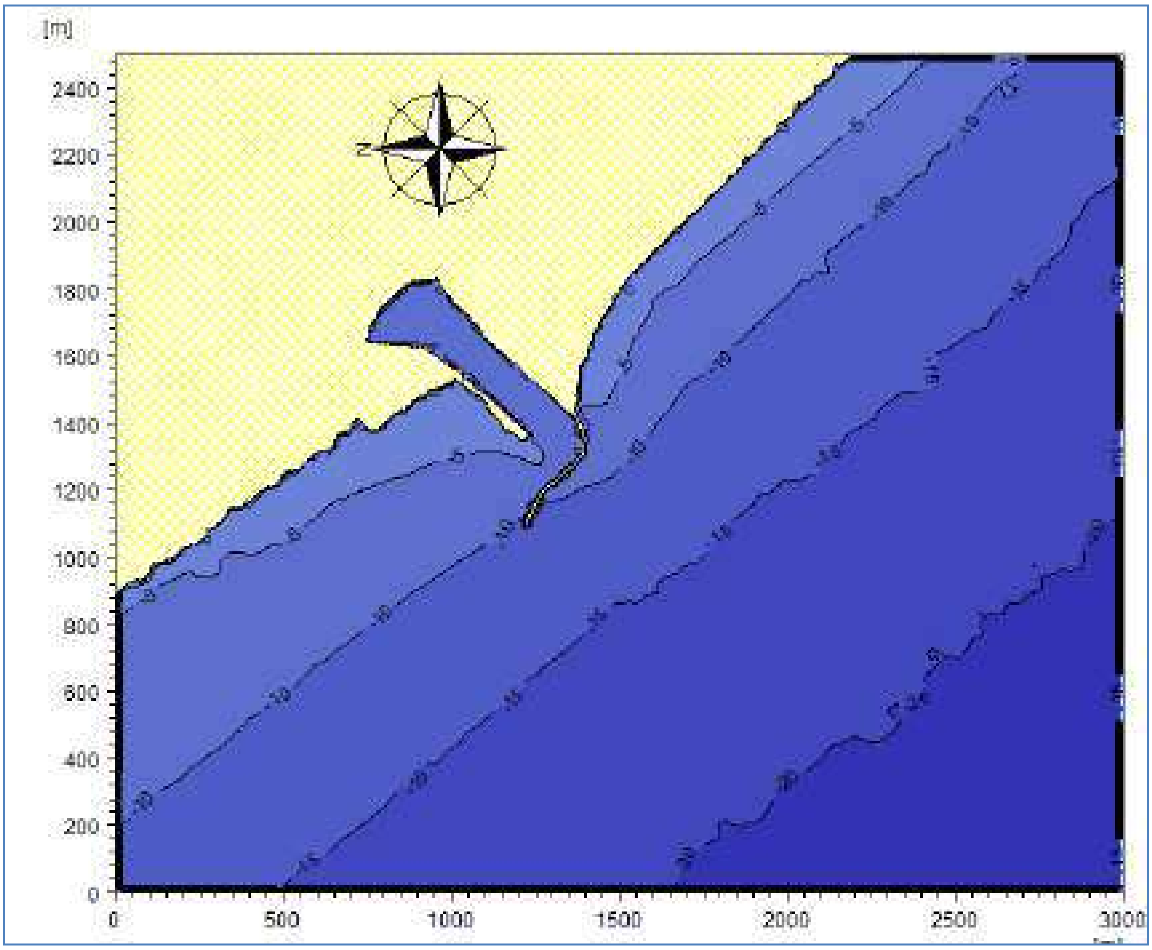

Fig.15 Model bathymetry for Optimized Proposed Layout (OPL)

The model for OPL was simulated under all the nearshore wave climate already enumerated in Table 5. The simulated results are shown in Figs.16(A-D) and discussions are made as under.

(i) Incident wave 1.5m in SSW (202.5^0) direction

In Fig.16(A), the model results for OPL under the influence of incident wave condition (1.5m height with SSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.6m exists which satisfies the required criteria of maneuvering of fishing boats.

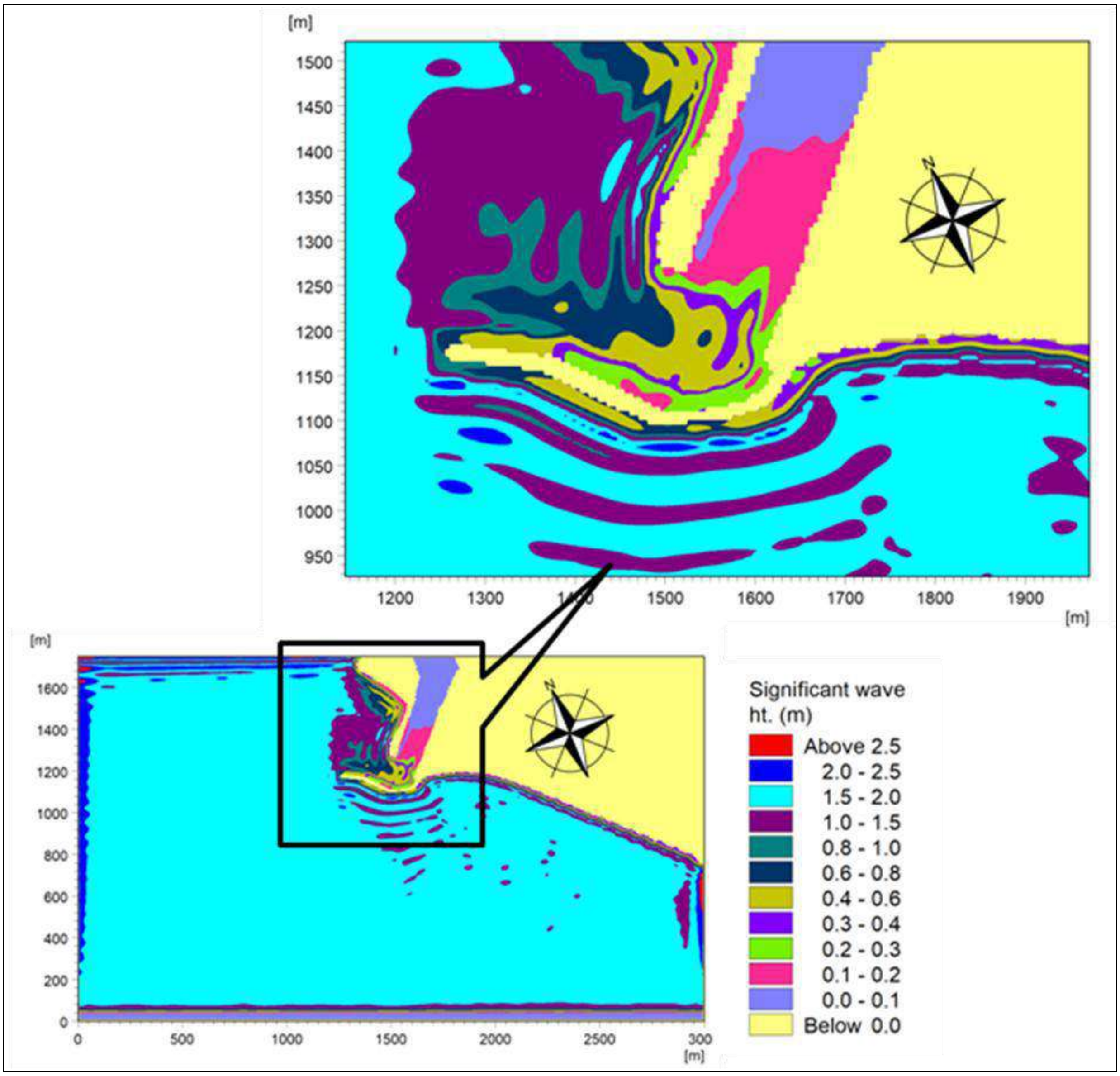

Fig.16(A).Wave height distribution plot for waves incident from SSW (202.5^0) direction (Incident wave height: 1.5m)

(ii) Incident wave 2.0m in SW (225^0) direction

In Fig.16(B), the model results for OPL under the influence of incident wave condition (2.0m height with SW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.4-0.6m exist which satisfies the required criteria.

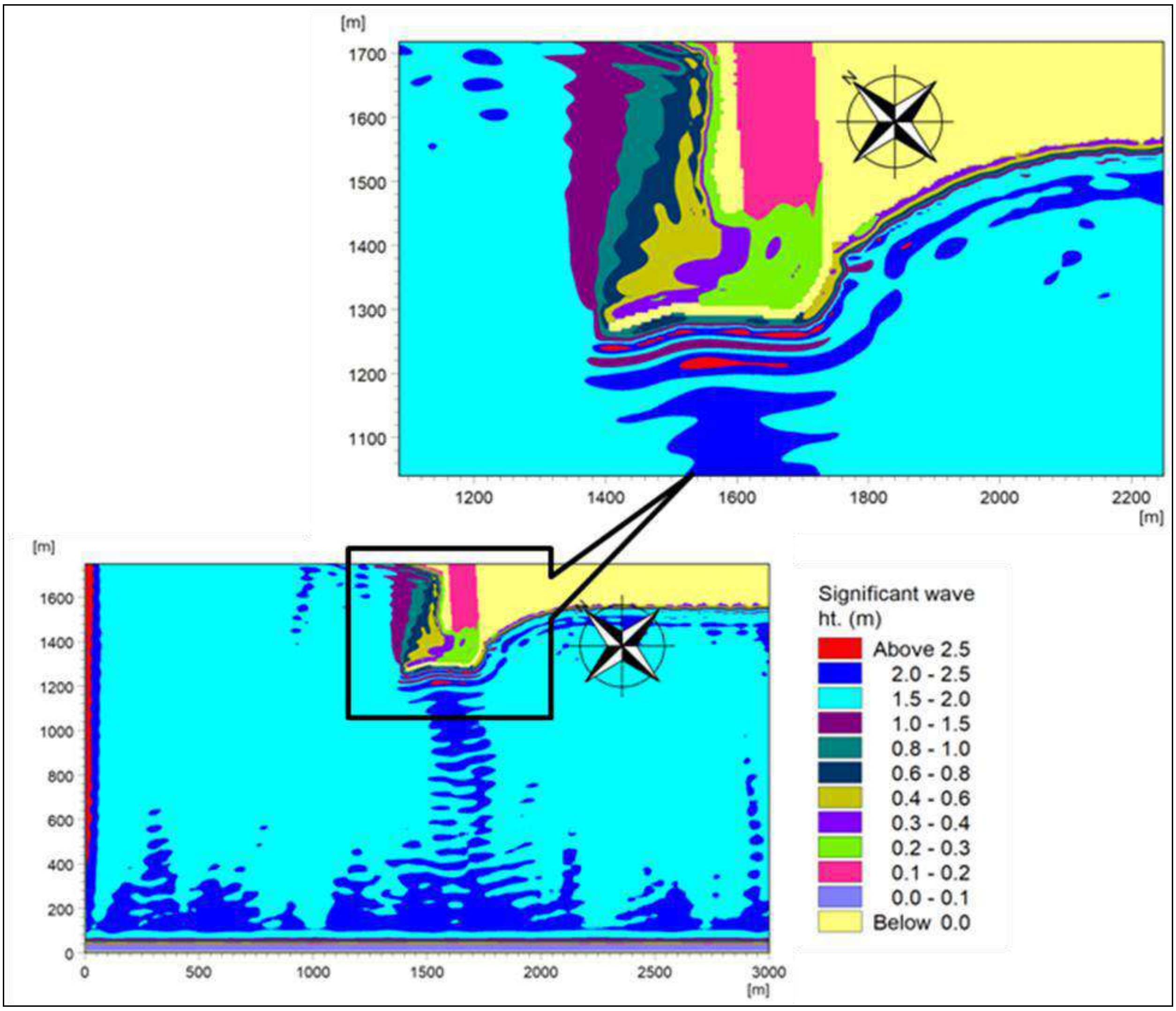

Fig.16(B) Wave height distribution plot for waves incident from SW (225^0) direction (Incident wave height: 2.0m)

(iii) Incident wave 2.5m in WSW (247.5^0) direction

In Fig.16(C), the model results for OPL under the influence of incident wave condition (2.5m height with WSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.6-0.8m exist which satisfies the required criteria.

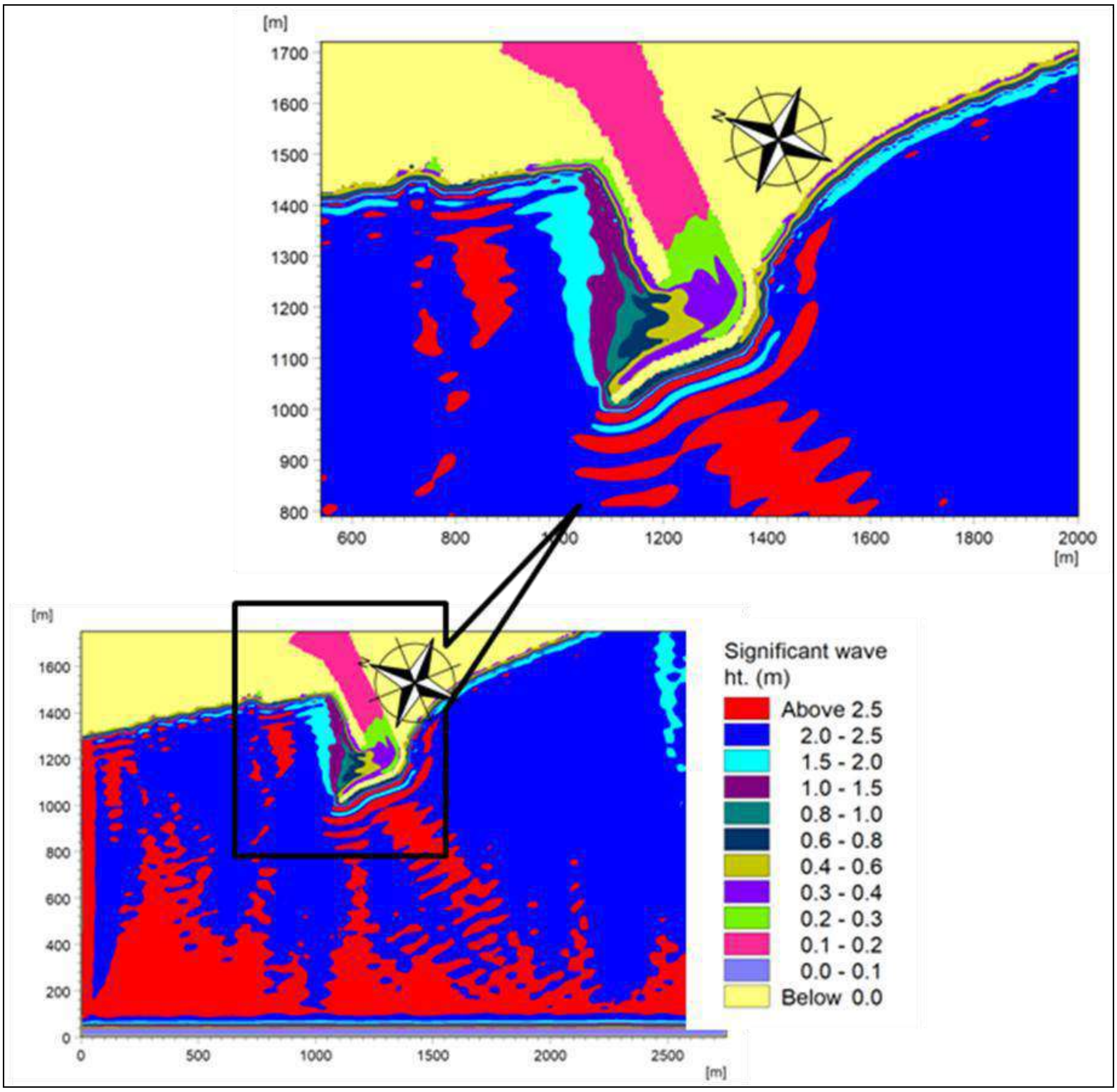

Fig.16(C) Wave height distribution plot for waves incident from WSW (247.5^0) direction (Incident wave height: 2.5m)

(iv) Incident wave 2.0m in W (270^0) direction

In Fig.16(D), the model results for OPL under the influence of incident wave condition (2.5m height with WSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.6-0.8m exist which satisfies the required criteria.

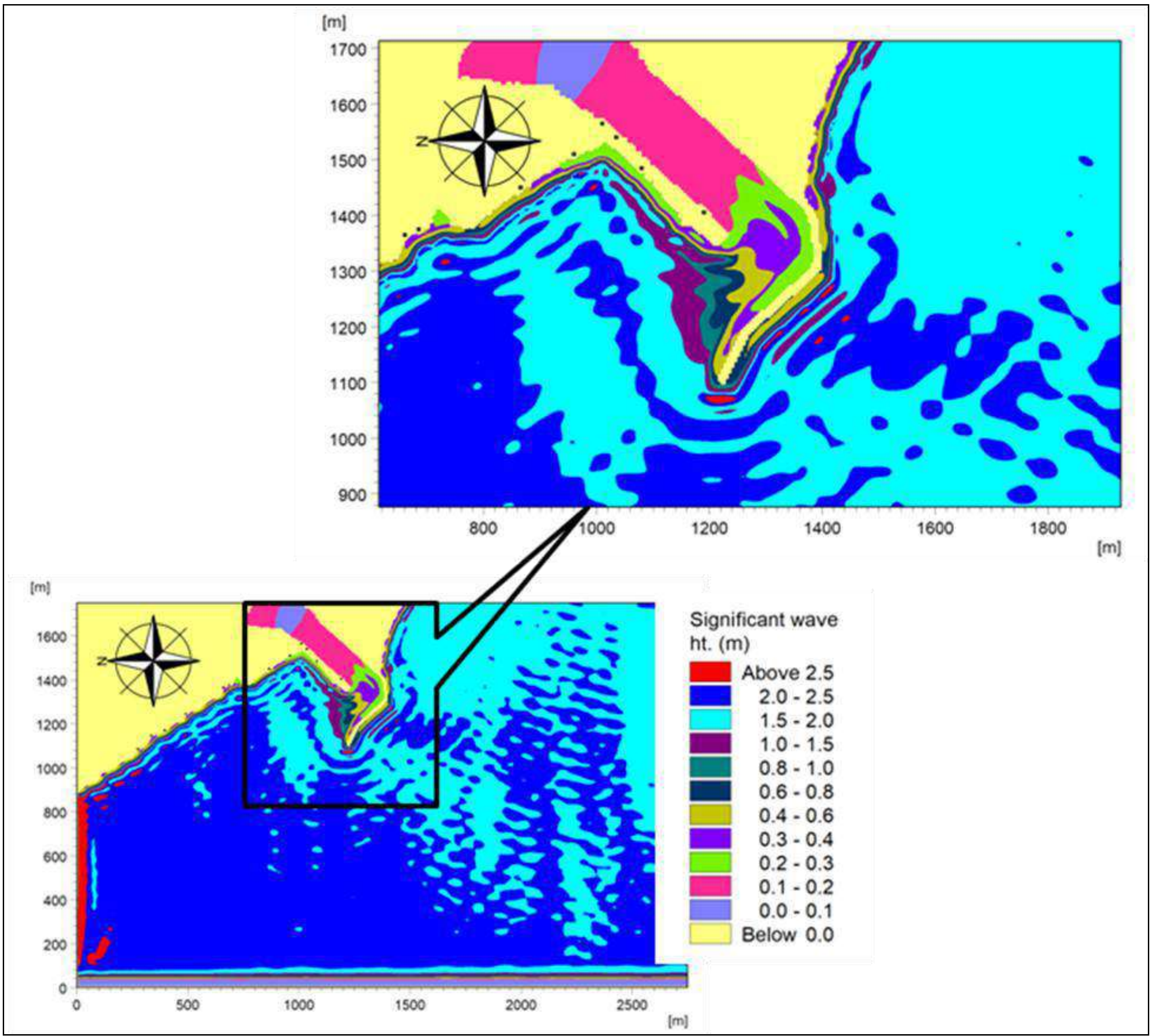

Fig.16(D) Wave height distribution plot for waves incident from West (270^0) direction (Incident wave height: 2.0m)

Based on the model results it can be observed that the Optimized Proposed Layout (OPL) satisfies the wave tranquility criteria at the entrance of the inlet. Hence, the OPL layout is recommended for the Muthalapozhy site. The model results and the recommendation thereof are based on the prevailing site conditions and field data reported.

7.5 Model Simulations of OPL under extreme wave conditions

At the site, it was stated that the extreme wave condition occurred with wave height of 3.5m. It was assumed that the extreme wave height incident from all four prominent directions. The model was simulated for the extreme wave condition (i.e incident wave height 3.5m) for all four incident directions. The simulated plots are depicted in Figs.17(A-D) and the model results are discussed as under.

(i) Incident wave 3.5m in SSW (202.5^0) direction

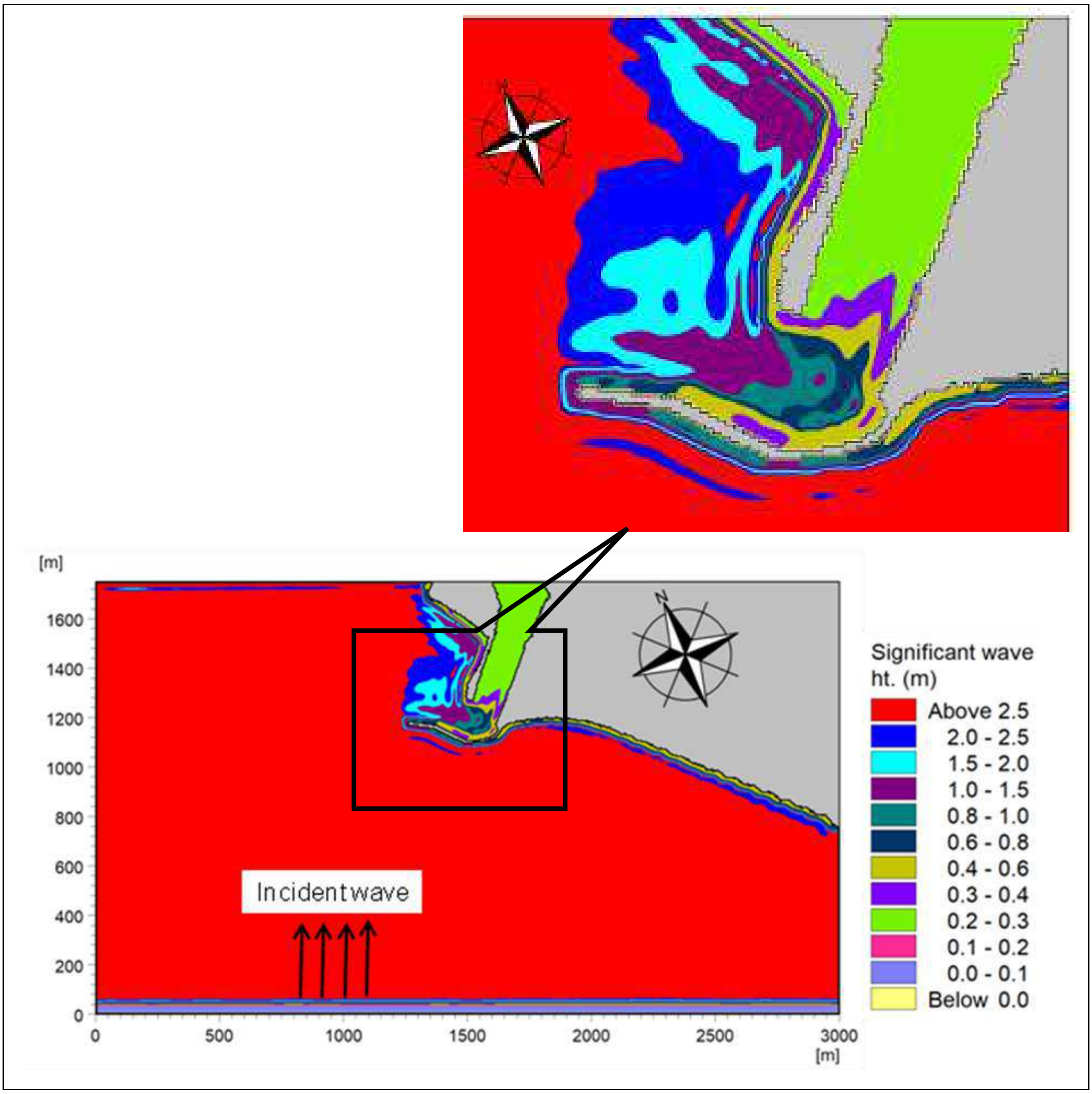

Fig.17(A) Wave height distribution plot for waves incident from WSW (202.5^0) direction (Incident wave height: 3.5m)

In Fig.17(A), the model results for OPL under the influence of extreme wave condition (3.5m height with SSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 1.0-1.5m exists.

(ii) Incident wave 3.5m in SW (225^0) direction

In Fig.17(B), the model results for OPL under the influence of extreme wave condition (3.5m height with SW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.8-1.0m exists.

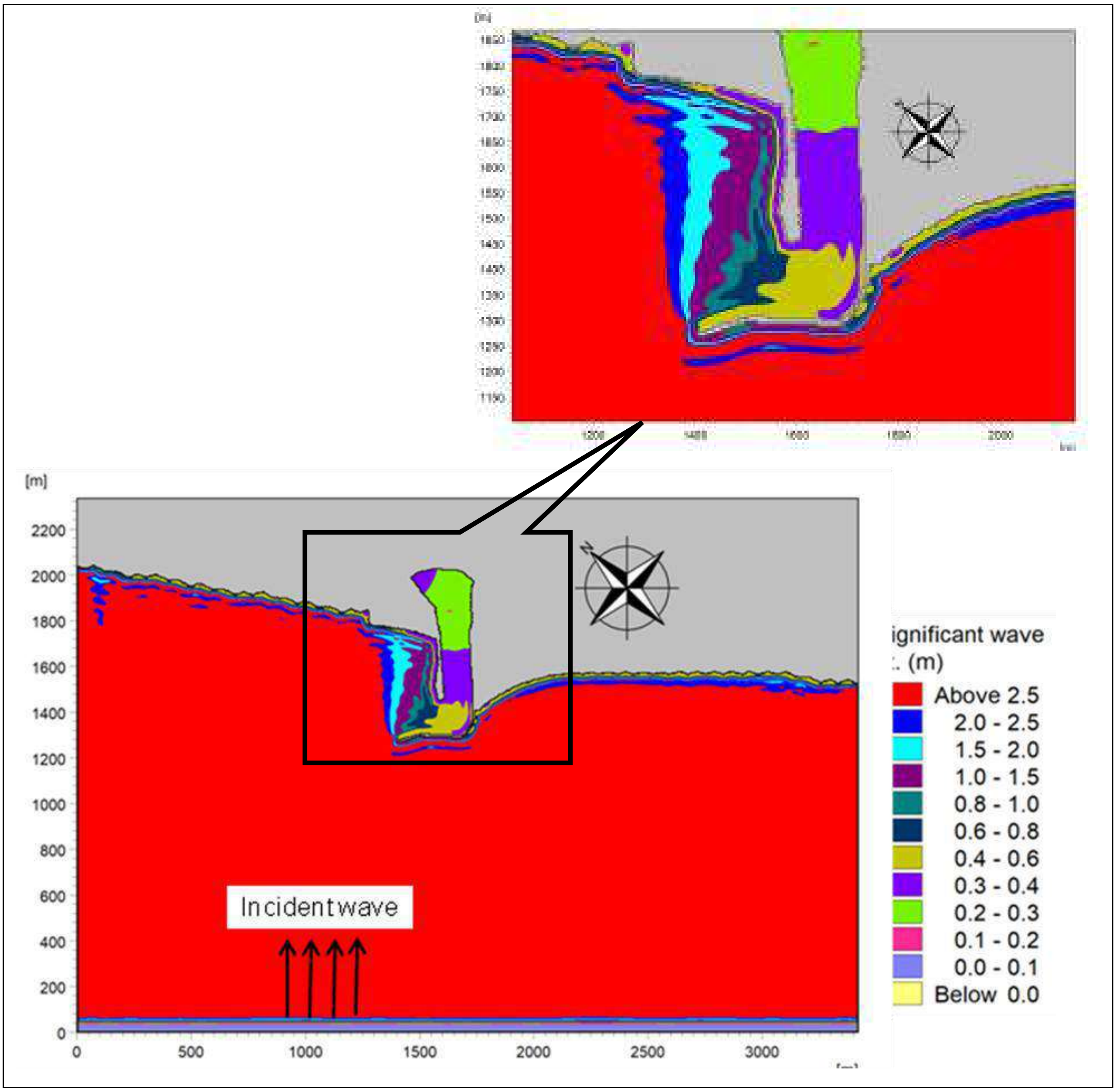

Fig.17(B) Wave height distribution plot for waves incident from SW (225^0) direction (Incident wave height: 3.5m)

(iii) Incident wave 3.5m in WSW (247.5^0) direction

In Fig.17(C), the model results for OPL under the influence of extreme wave condition (3.5m height with WSW direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 0.8-1.0m exists.

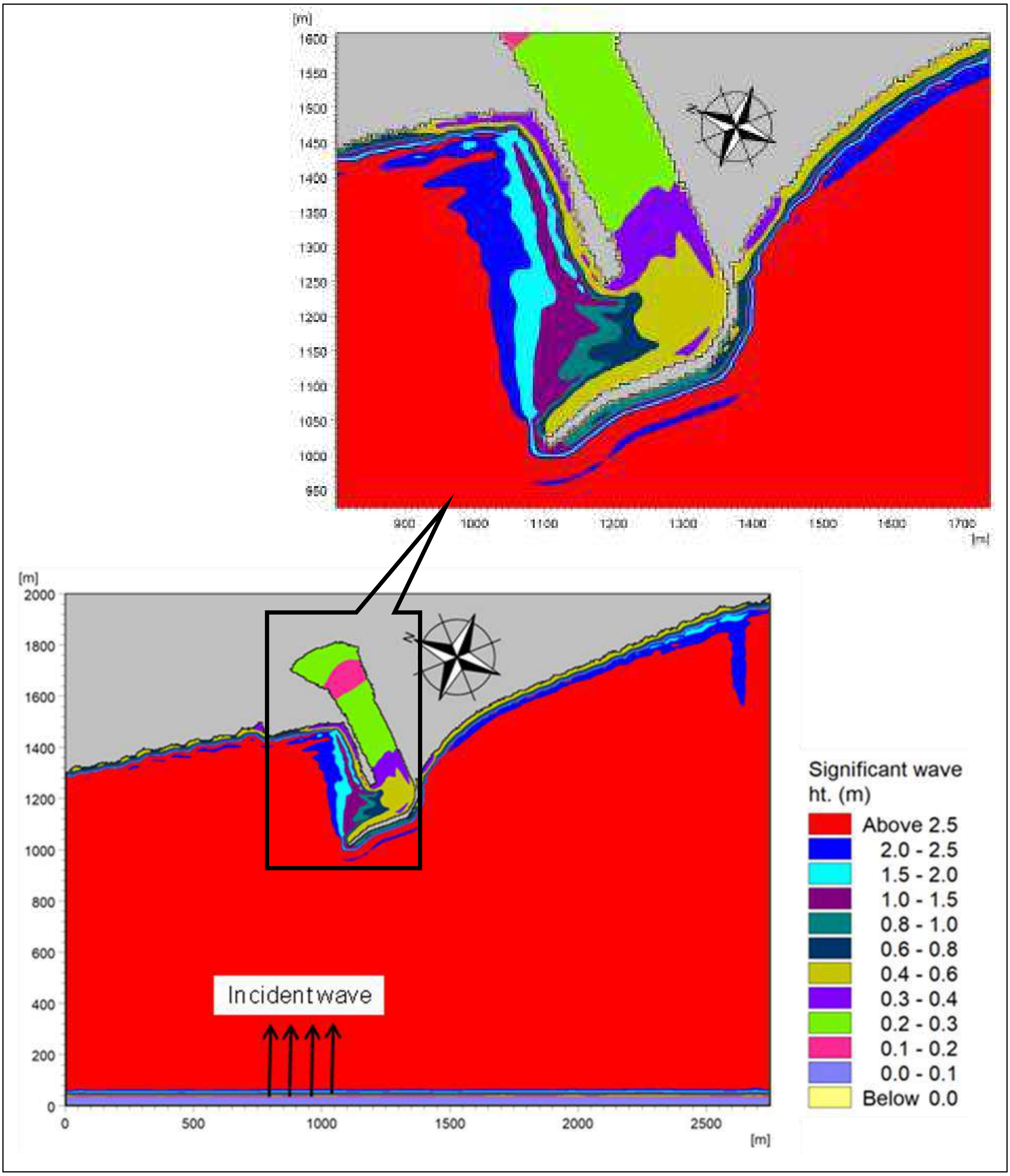

Fig.17(C) Wave height distribution plot for waves incident from WSW (247.5^0) direction (Incident wave height: 3.5m)

(iv) Incident wave 3.5m in W (270^0) direction

In Fig.17(D), the model results for OPL under the influence of extreme wave condition (3.5m height with W direction) is shown. From the figure it can be seen that at the entrance wave height of the order of 1.0-1.5m exists.

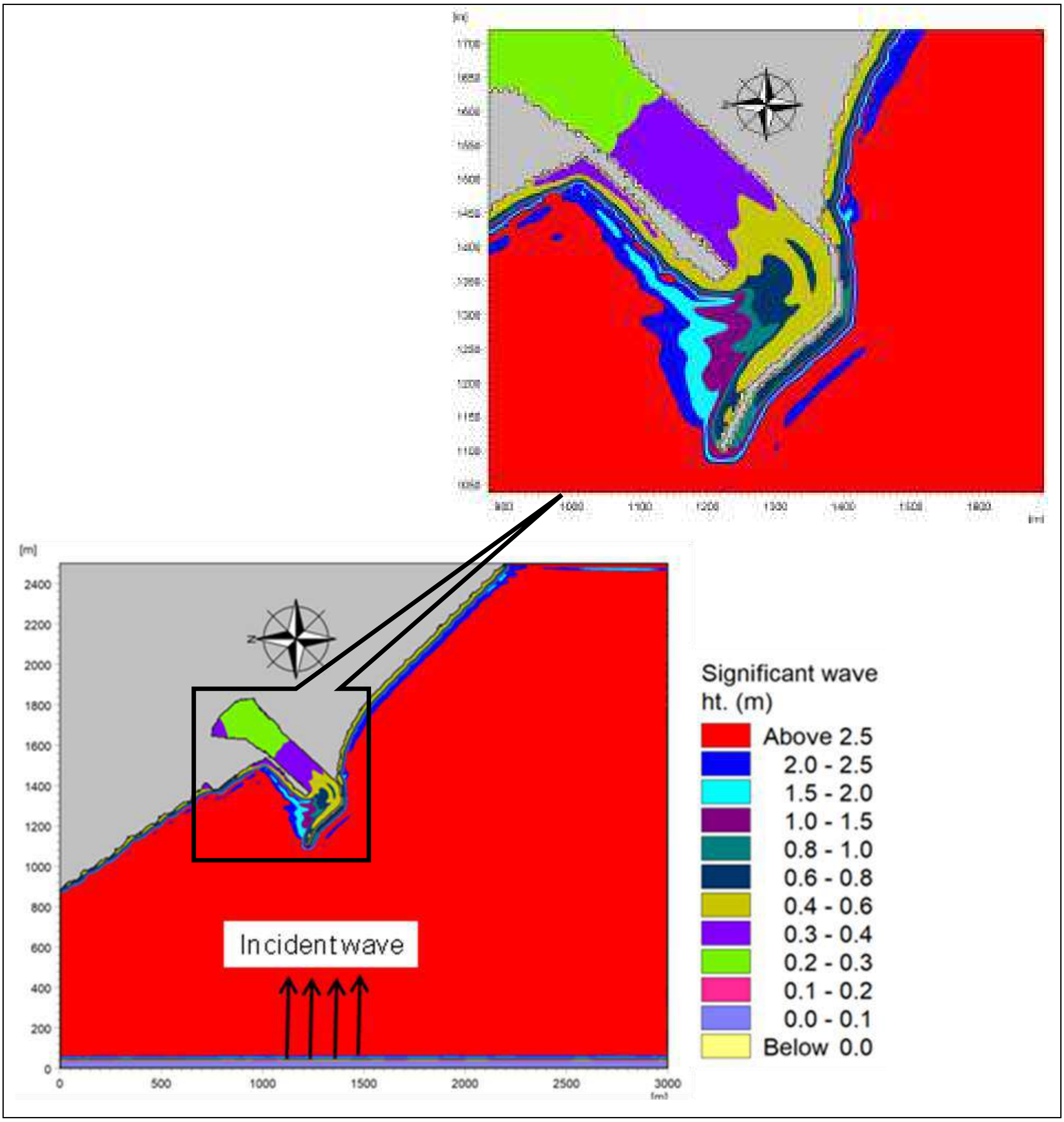

Fig.17(D) Wave height distribution plot for waves incident from West (270^0) direction (Incident wave height: 3.5m)

Based on the simulation results of OPL for the extreme wave condition of 3.5m, it can be observed that the wave height near channel entrance varies from 0.6m to 1.5m.

The simulated significant wave heights obtained at the channel entrance in each scenario of incident wave dealt above is summarized in the Table 7.

Table 7 Significant wave height at channel entrance (in metre)

Incident wave Height & Dir	Initial layout-1 (IPL-1)	Initial layout-2 (IPL-2)	Initial layout-3 (IPL-3)	Initial layout-4 (IPL-4)	Modified layout (MPL)	Optimized layout (OPL)
1.5 202.5 (SSW)	1 to 1.5	1 to 1.5	1.5 to 2	1 to 1.5	0.6 to 0.8	0.6 to 0.8
2 225 (SW)	1.5 to 2	1.5 to 2	1.5 to 2	1 to 1.5	0.3 to 0.4	0.3 to 0.4
2.5 247.5(WSW)	1.5 to 2	2 to 2.5	2 to 2.5	2 to 2.5	0.4 to 0.6	0.4 to 0.6
2 270 (W)	1.5 to 2	1.5 to 2	1.5 to 2	1.5 to 2	0.4 to 0.6	0.4 to 0.6

From the above table it can be observed that wave heights are higher than the permissible limit in initial layouts (IPL-1 to IPL-4) suggested by project authority. However, in modified (MPL) and optimized layout (OPL) wave heights are within permissible limit. The permissible limit of wave at the entrance of the channel herein was assumed to be of the order of 0.4-0.6m. Further, in case of optimized layout (OPL) width at the entrance of the channel is more than that of modified layout (MPL). Also, due to wider opening of the entrance in OPL waves were seen to be penetrating more inside the entrance as compared to MPL.

8. Hydrodynamics and sediment transport model studies

8.1 Hydrodynamic modelling under existing condition

Hydrodynamic model was setup for the Muthalapozhy site to cover domain 45km X 40km for the simulation with global tide in line series along the three boundaries. Model boundary is shown in Fig.18. The local model was set up with domain size 10km x 9km with tide extracted from global model along the all three boundaries. The global and local model boundaries are shown in Fig.19(A-B).

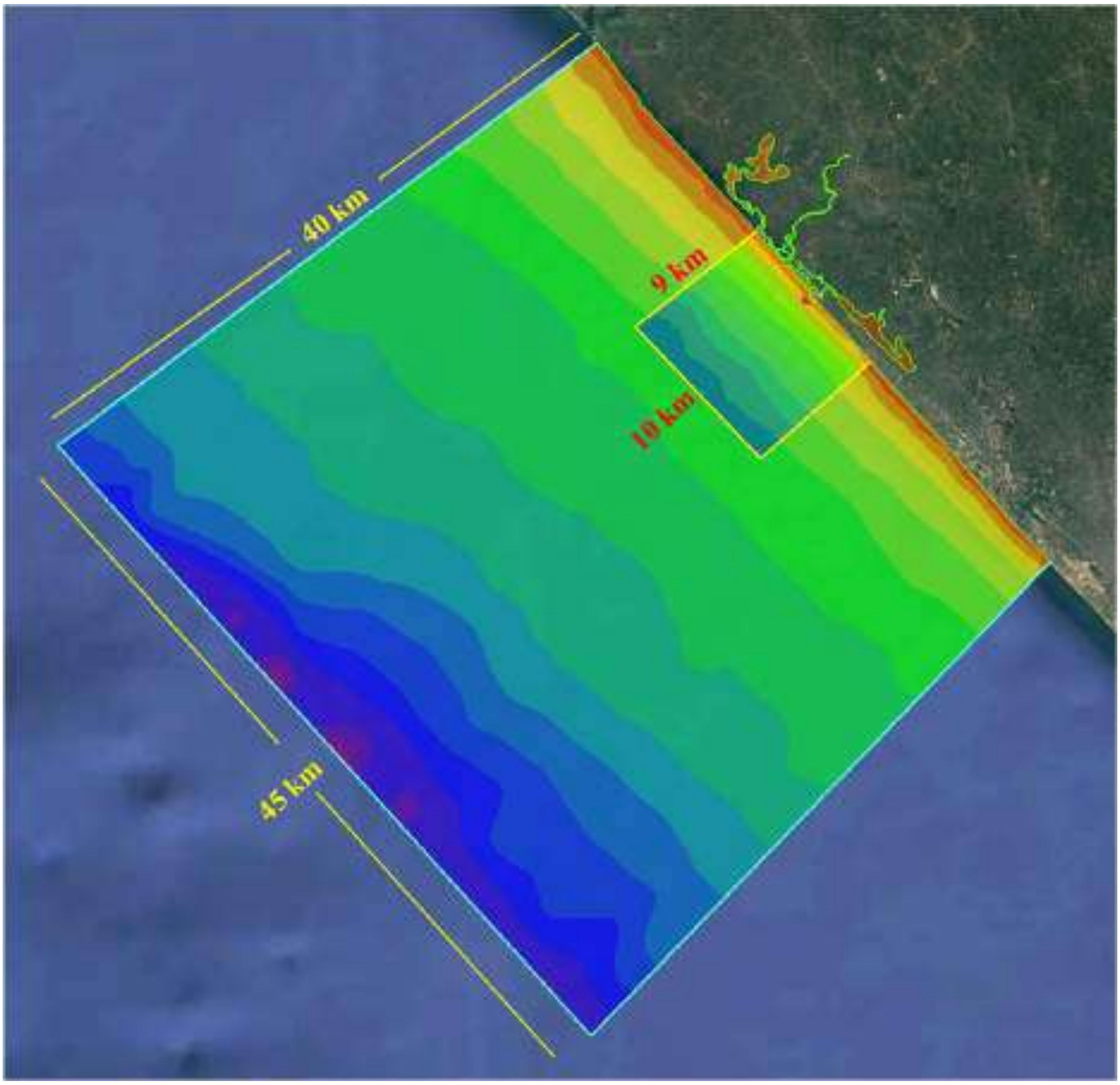

Fig.18 Model domain in Google map

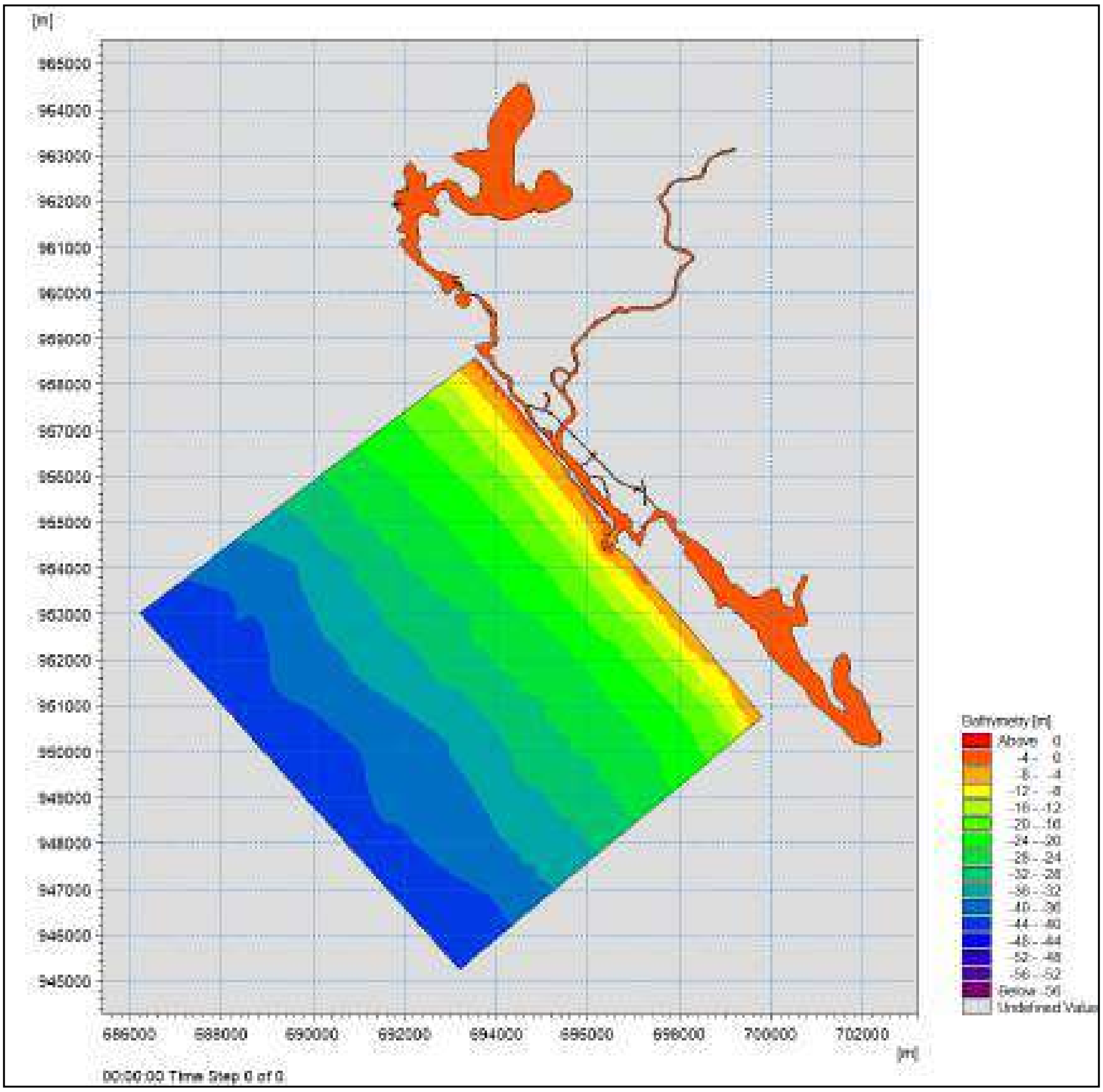

Fig.19(A) Complete Model Bathymetry used in hydrodynamic model

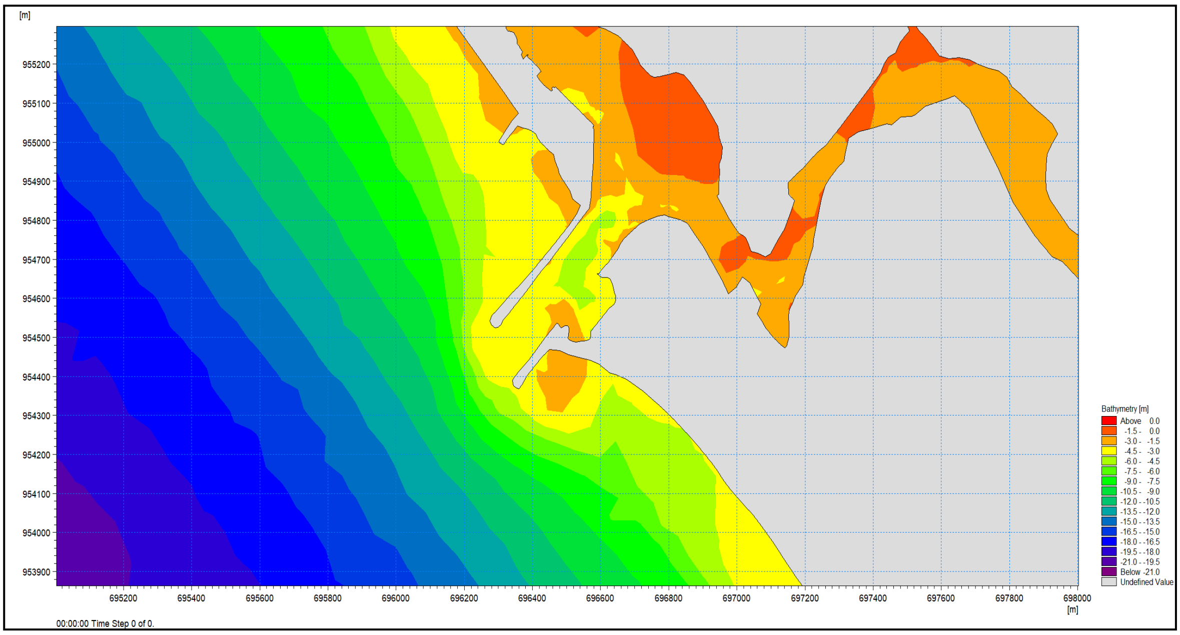

Fig.19(B) Part bathymetry for hydrodynamic model simulations

Field data collection was carried out by CWPRS in the month of October 2023. The collected current data, tide data, grain size distribution and sediment concentration were used for calibration of the hydrodynamic and the sediment transport model. The locations of field data are shown in Fig.20. The collected field data in terms of tidal level, current magnitude and direction are shown in Fig.21. From the figure, it can be observed that spring tidal range is 1.0m and neap tidal range in 0.2m. The maximum tidal currents at the site were observed to be of the order of 0.6m/s.

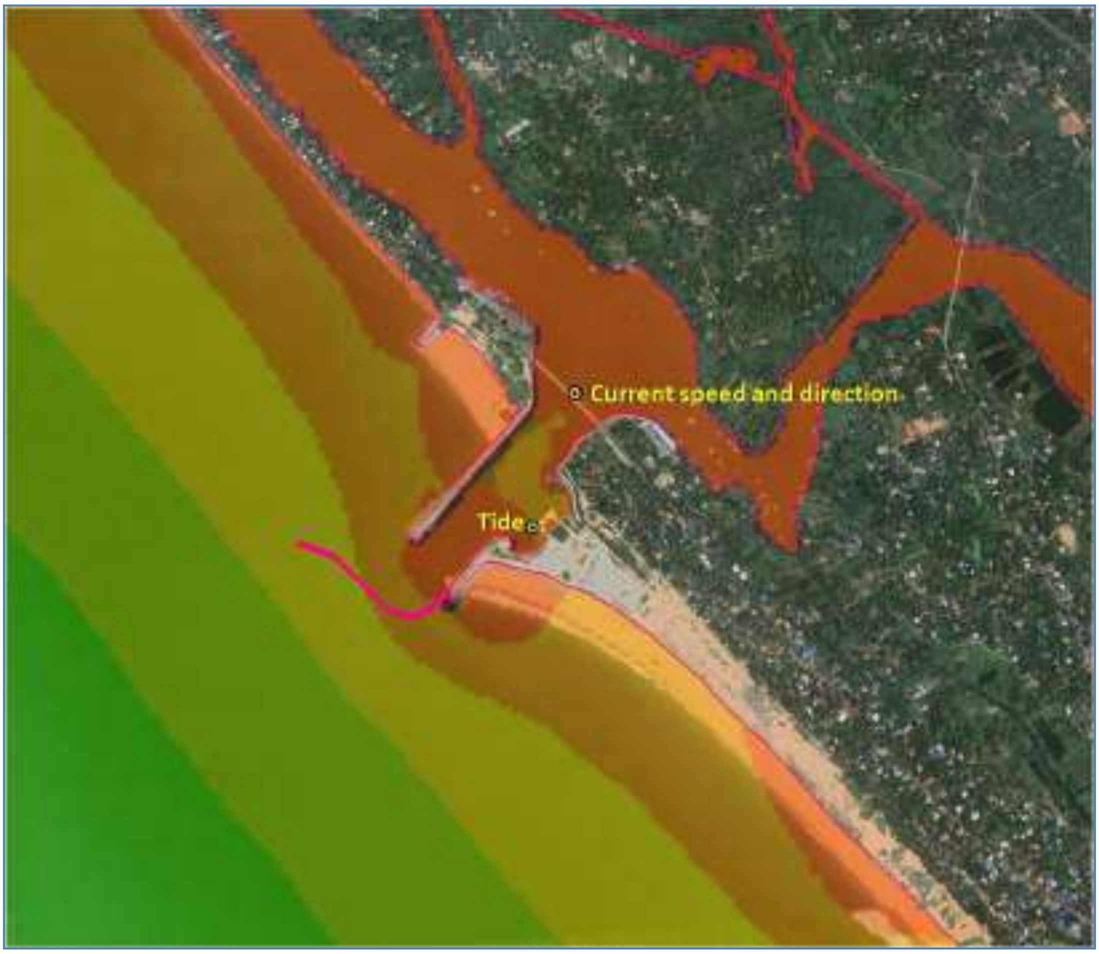

Fig.20 Locations of observed current and tide data

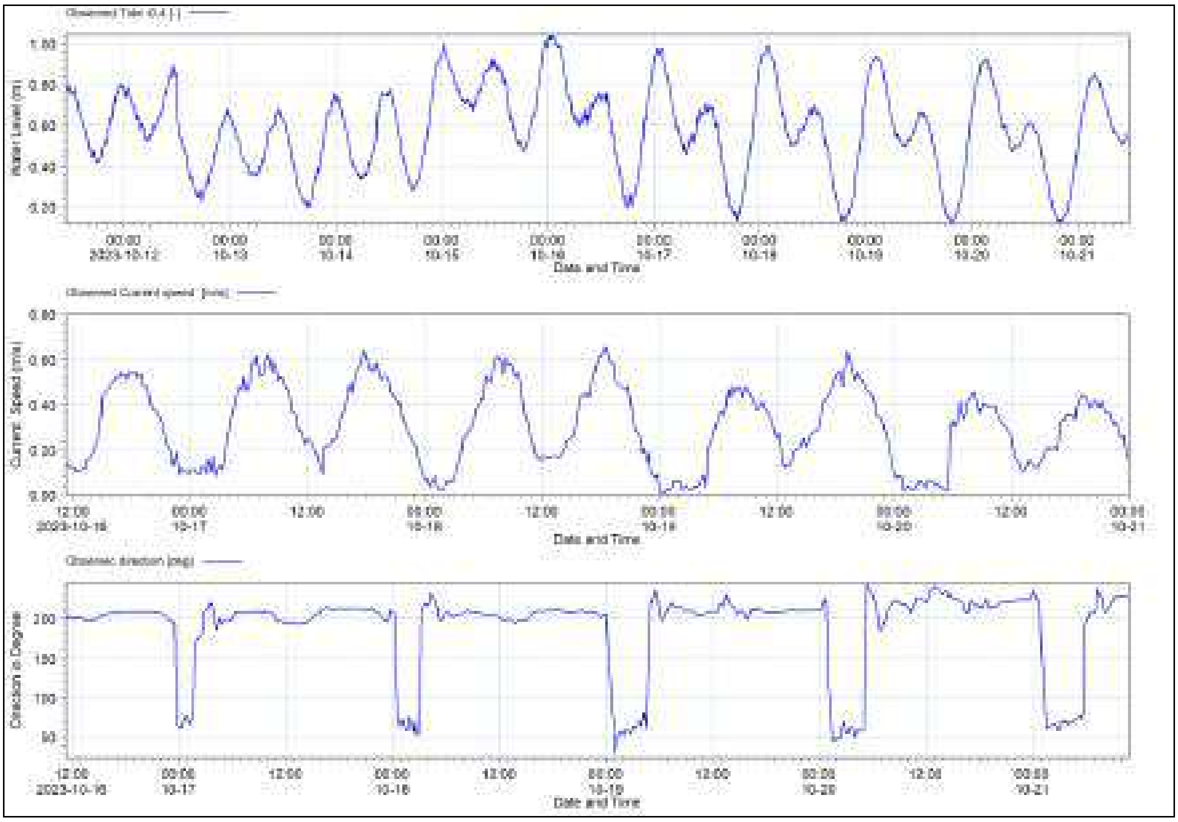

Fig.21 Observed field data (Tide, Current magnitude & direction) at Muthalapozhy

8.2 Calibration of hydrodynamic (HD) model

The site of study area was influenced by both tides and waves. Accordingly, both tide and wave need to be considered in simulating the model. In order to obtain radiation stresses, MIKE-21 SW model was simulated considering wave climate as 2m with incident angle of 215^0N. The simulated radiation stresses were supplied to hydrodynamic model (MIKE21 HD) in order to take into account for wave induced currents. The hydrodynamic model was simulated after imposing tide extracted from the global model. The HD Model was simulated for the existing condition. Model was simulated for 15 days continuously to cover both spring and neap tides. In order to calibrate the model, bed friction coefficient was fine tuned to arrive at desired observed tidal currents in the model. For existing condition, typical flow fields during flooding and ebbing phase of the tide are shown in Fig.22(A-B).The arrow head of vector shows the direction of flow while its magnitude is denoted by its length on the length scale shown on the right side of figure.

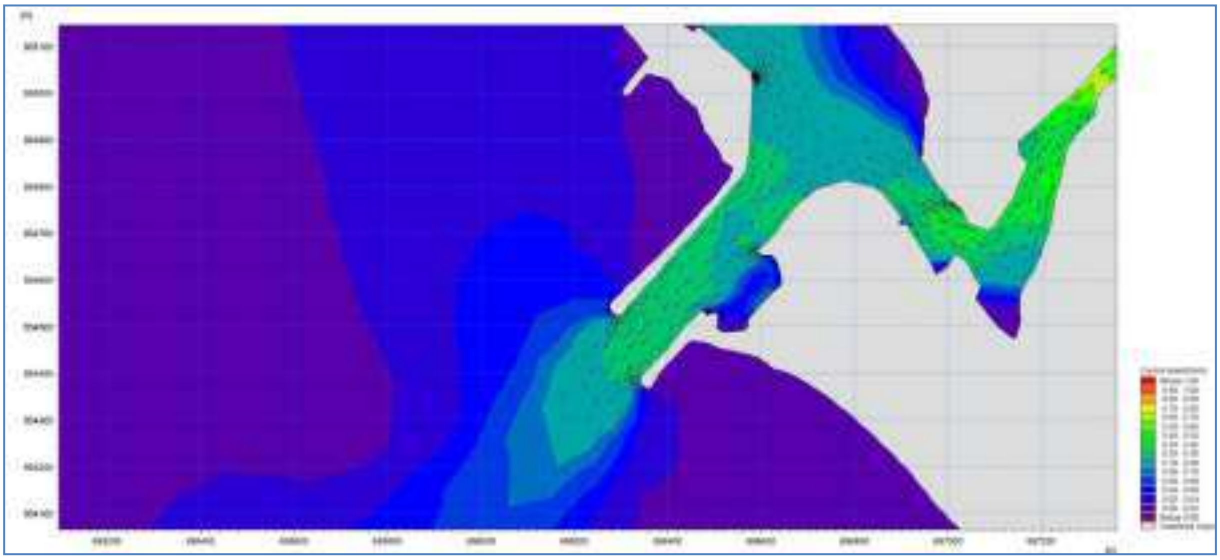

Fig.22(A) Typical flow field during peak ebbing

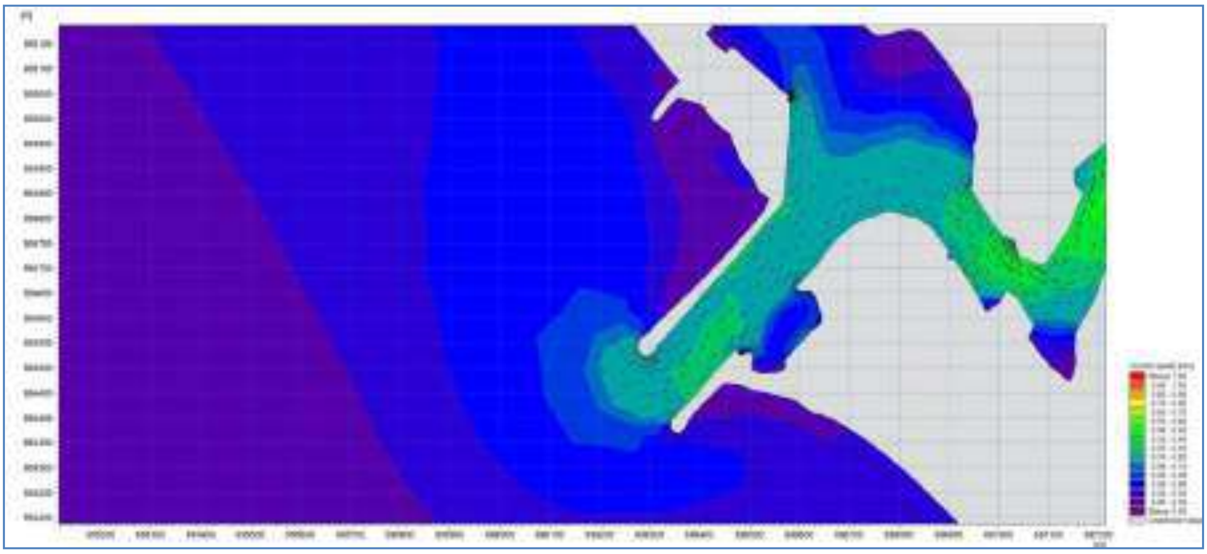

Fig.22(B) Typical flow field during peak flooding

Hydrodynamic model was calibrated based of the tide and current data observed at the site at different locations (see Fig.20). The simulated tidal level near the Adani jetty Site was extracted and compared with the observed tide at the same location. The comparison is shown in Fig.23.

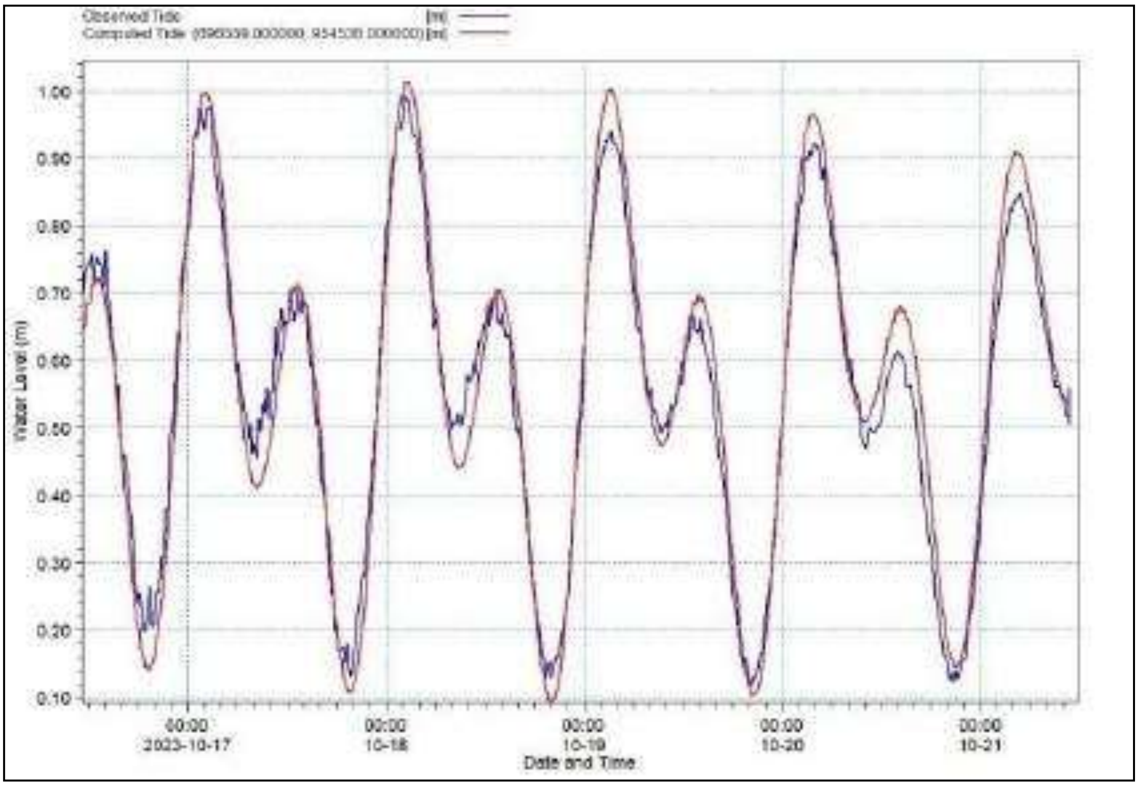

Fig.23 Comparison of tidal level near Adani jetty site at Muthalapozhy

From the figure it is seen that the water levels are matching reasonably well. However, some discrepancies are observed that may be due to positional error as the tide gauge was located near the jetty in the channel. Similarly, tidal current magnitude and direction were also compared with the observed data. The current data was observed in the river near the bridge at Muthalapozhy. The comparison of current and direction have been shown in Fig.24(A-B).

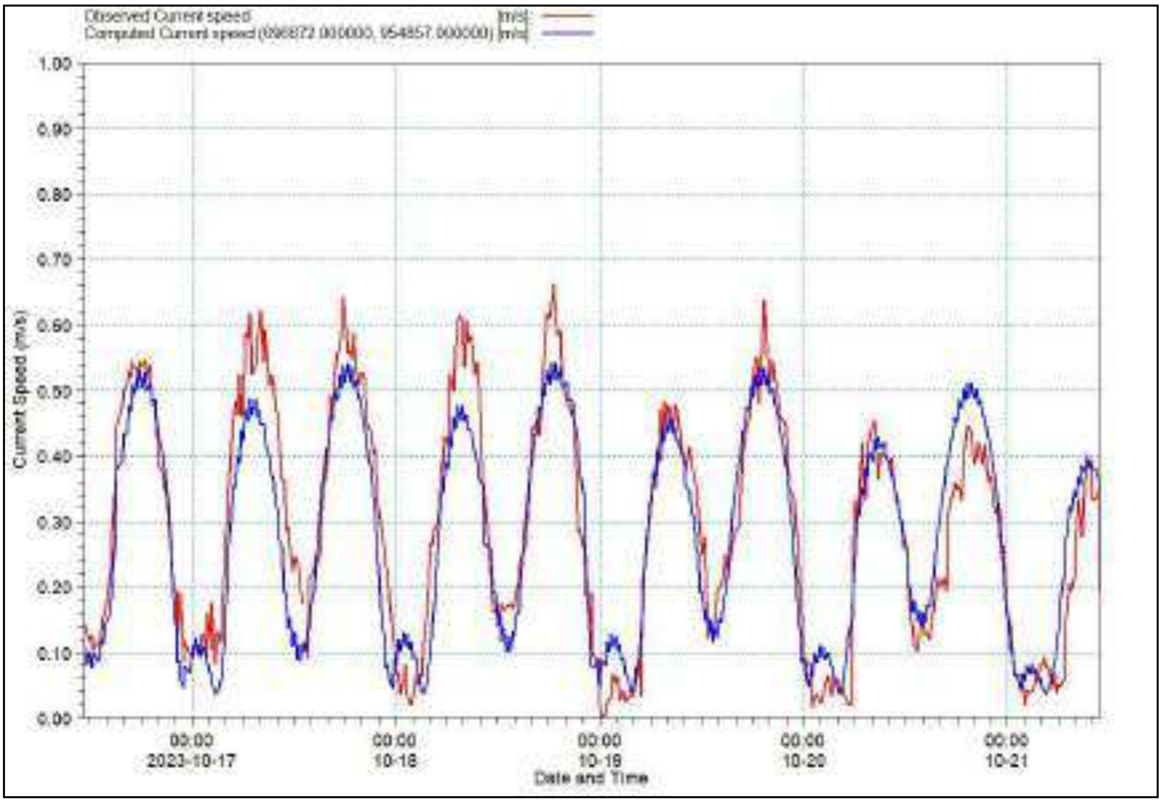

Fig.24(A) Comparison of tidal current magnitude at river near the bridge

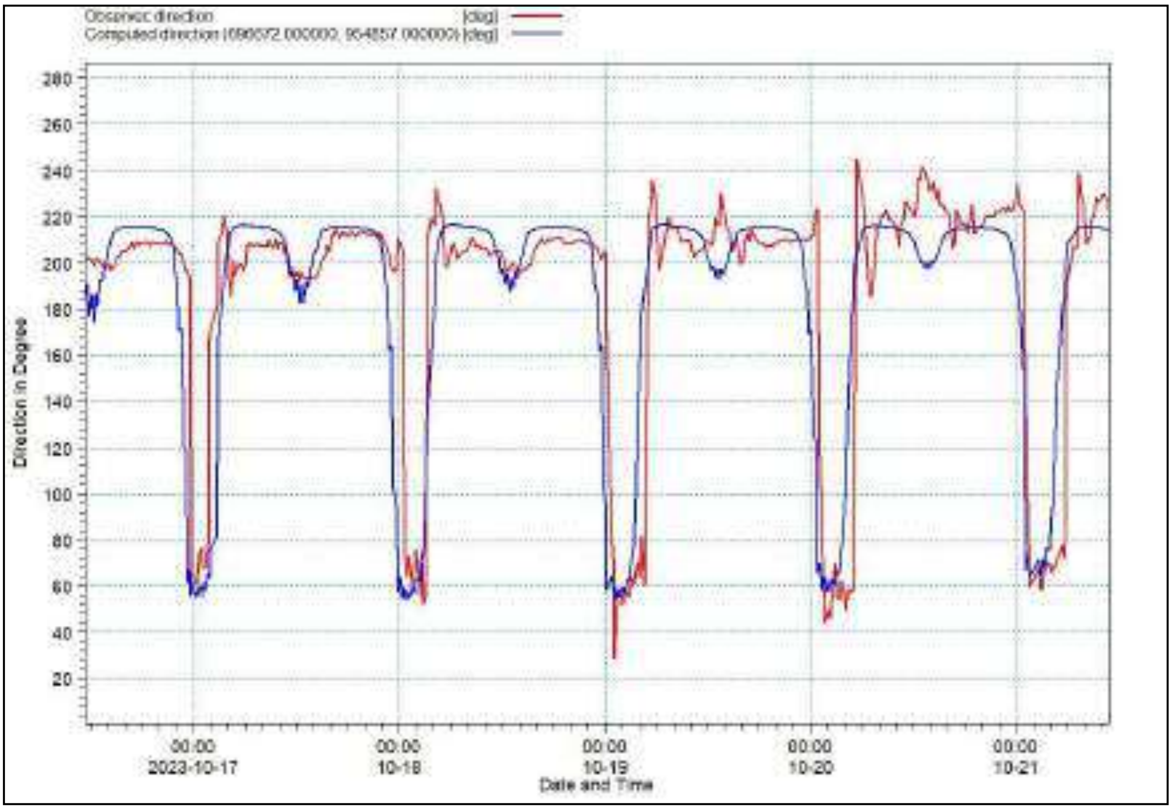

Fig.24(B) Comparison of tidal current direction at river near the bridge

From the figures, it can be observed that the current magnitude and direction matches well. Some discrepancies are observed in the figures due to positional error and point data collected during the observations. The statistical comparison for tide and tidal currents are shown in Figs.25 and 26(A-B), respectively.

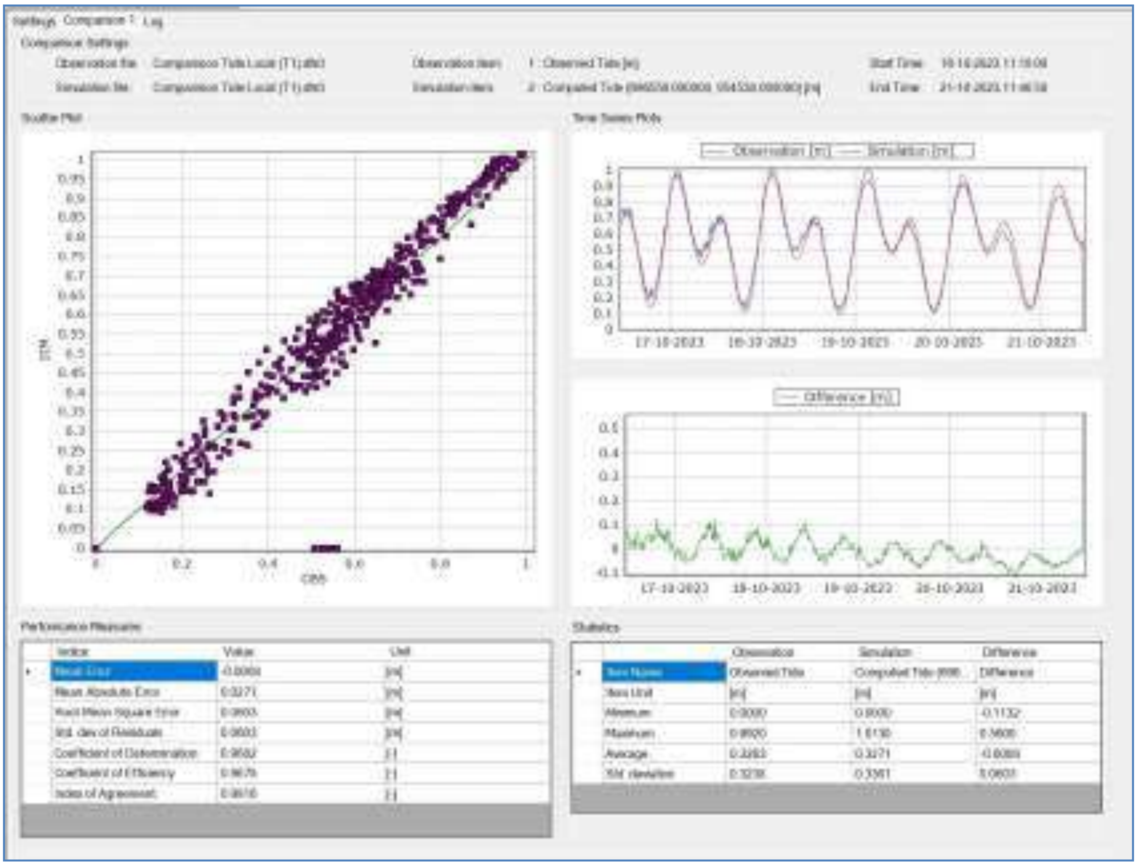

Fig.25 Statistical measures of tide comparison

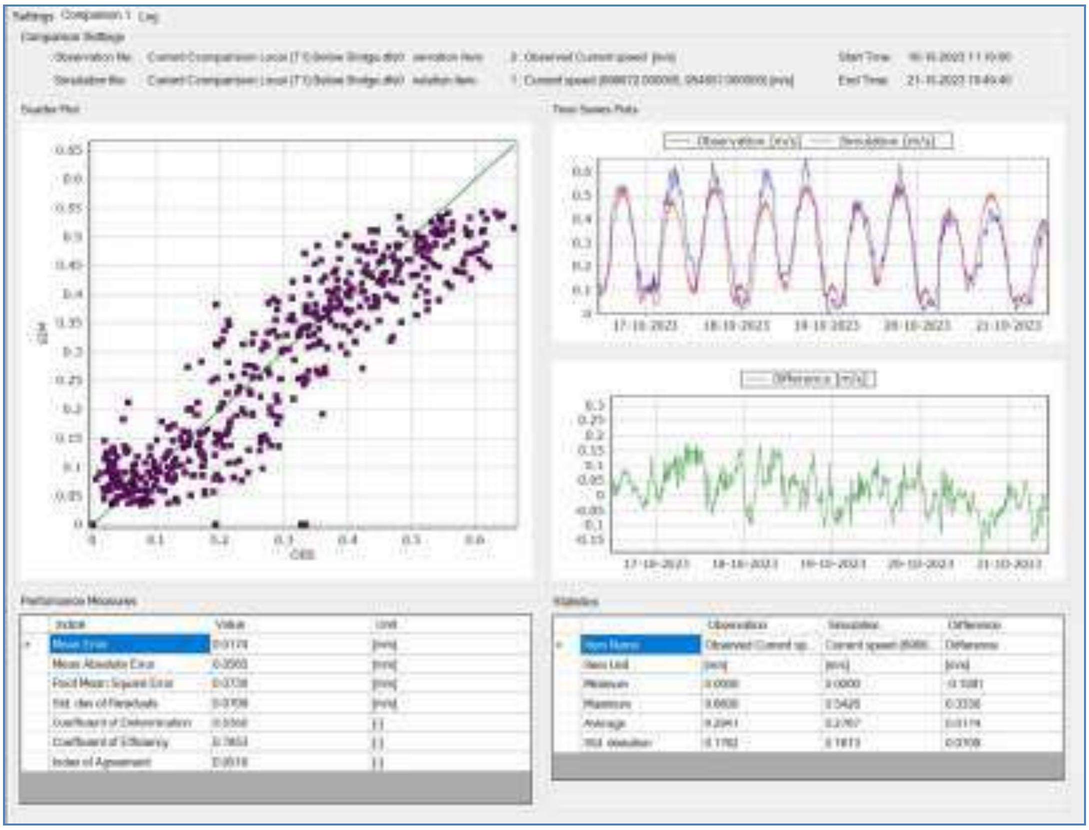

Fig.26(A) Statistical measures of current magnitude comparison

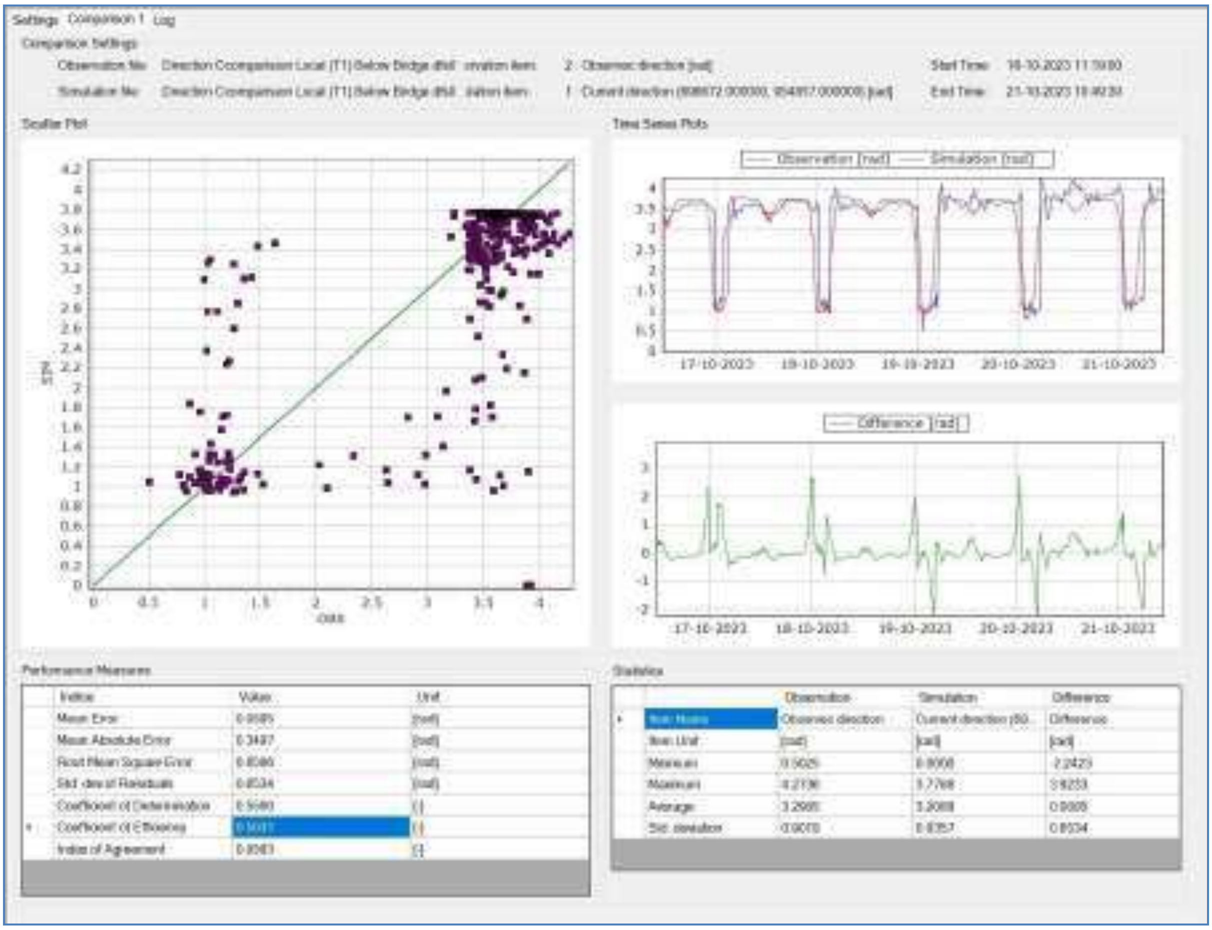

Fig.26(B) Performance evaluation of direction comparison

8.3 Sediment transport studies under Existing condition

After calibrating hydrodynamic model, sediment transport model was simulated for 15 days for river discharges conditions during monsoon and non-monsoon seasons. Sedimentation patterns after 15 days of simulation are shown in Fig.27 and Fig.28 for monsoon and non-monsoon periods respectively. During monsoon river discharge condition (Fig.27), erosion can be observed in the channel area and deposition near the Adani jetty area. At other locations, there are no significant changes in the bed level. Sedimentation pattern during non-monsoon period shown in Fig.28 indicated quite low erosion in the channel. However, at other locations of the site equilibrium condition was observed in the simulation results.

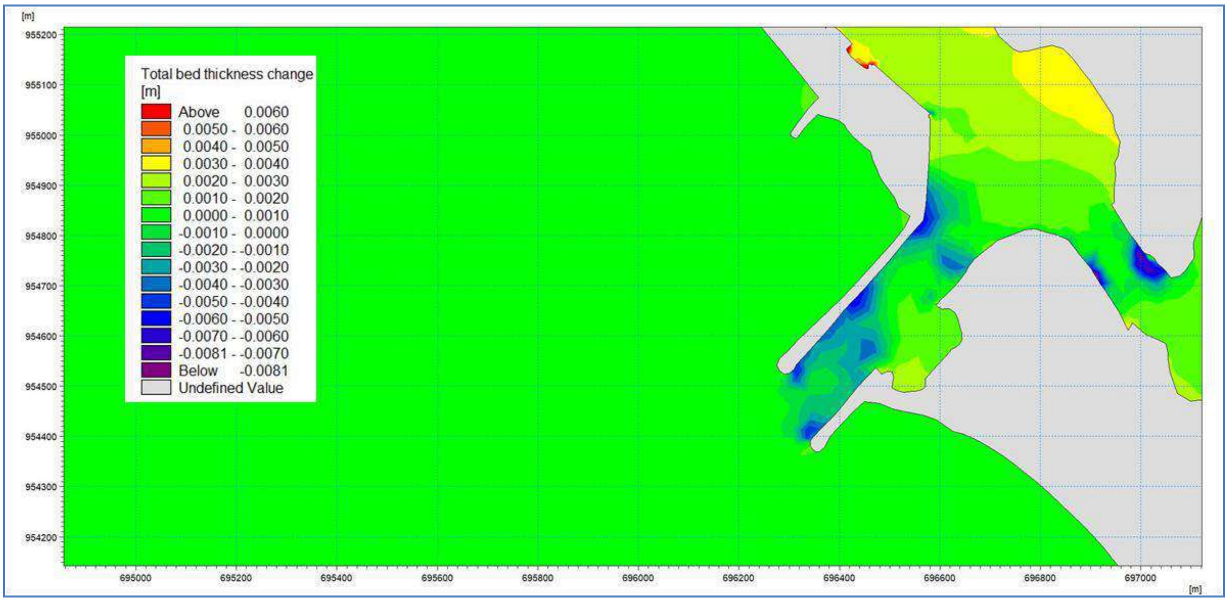

Fig. 27 Sedimentation pattern in existing condition during monsoon

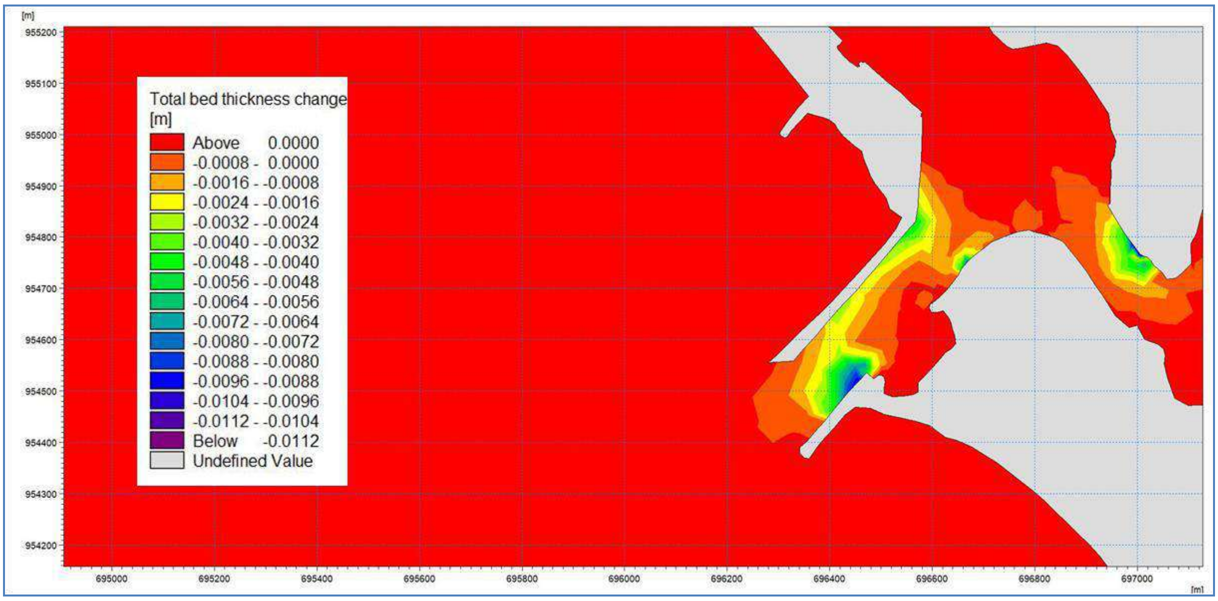

Fig. 28 Sedimentation pattern in existing condition during non-monsoon

9 Hydrodynamics and sediment transport studies for OPL

9.1 Hydrodynamic modeling for OPL

Optimized Proposed layout (OPL) involved extension of southern breakwater for the development of Muthalapozhy fishing harbour with length 420 m and the channel reach dredged upto -6m below CD. After incorporating proposed layout in existing bathymetry, hydrodynamic model was simulated for 15 days without changing the model parameters for which model was calibrated. Model was simulated for monsoon and non-monsoon conditions. It could be observed from simulated results that during monsoon conditions, flow was unidirectional and only ebb current was observed whereas, during non-monsoon condition, reversal of flow could be observed. In order to compare the current magnitudes in the channel in the existing and in the proposed conditions 3 different locations (viz, Point 1, Point 2, Point 3) were considered in the channel as shown in Fig.29. The comparison of currents in existing and proposed conditions during non-monsoon period at three selected locations is shown in Figs.30-32.

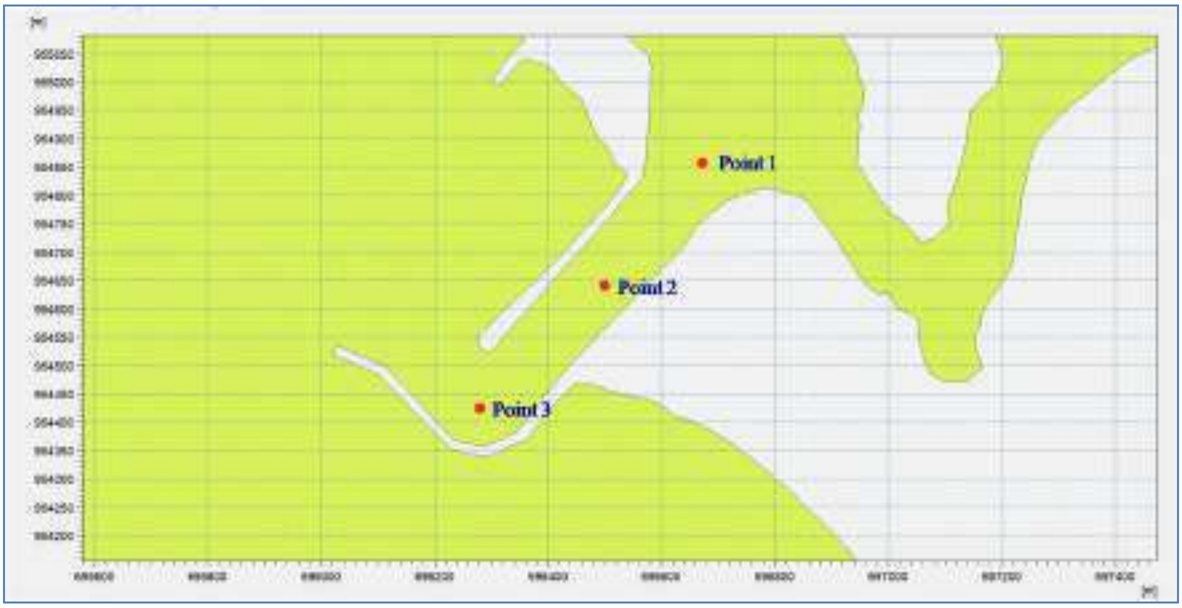

Fig. 29 Current speed extraction at different locations

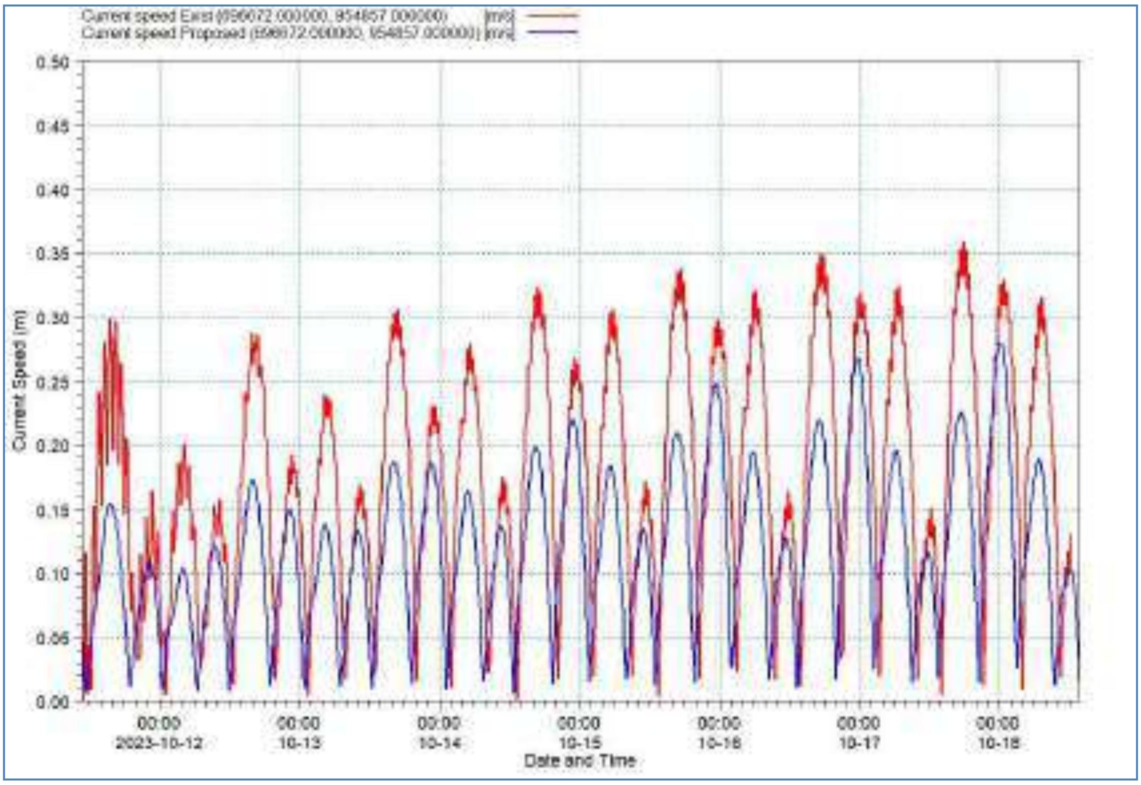

Fig. 30 Comparison of currents at Point 1 in existing and proposed condition during non-monsoon period

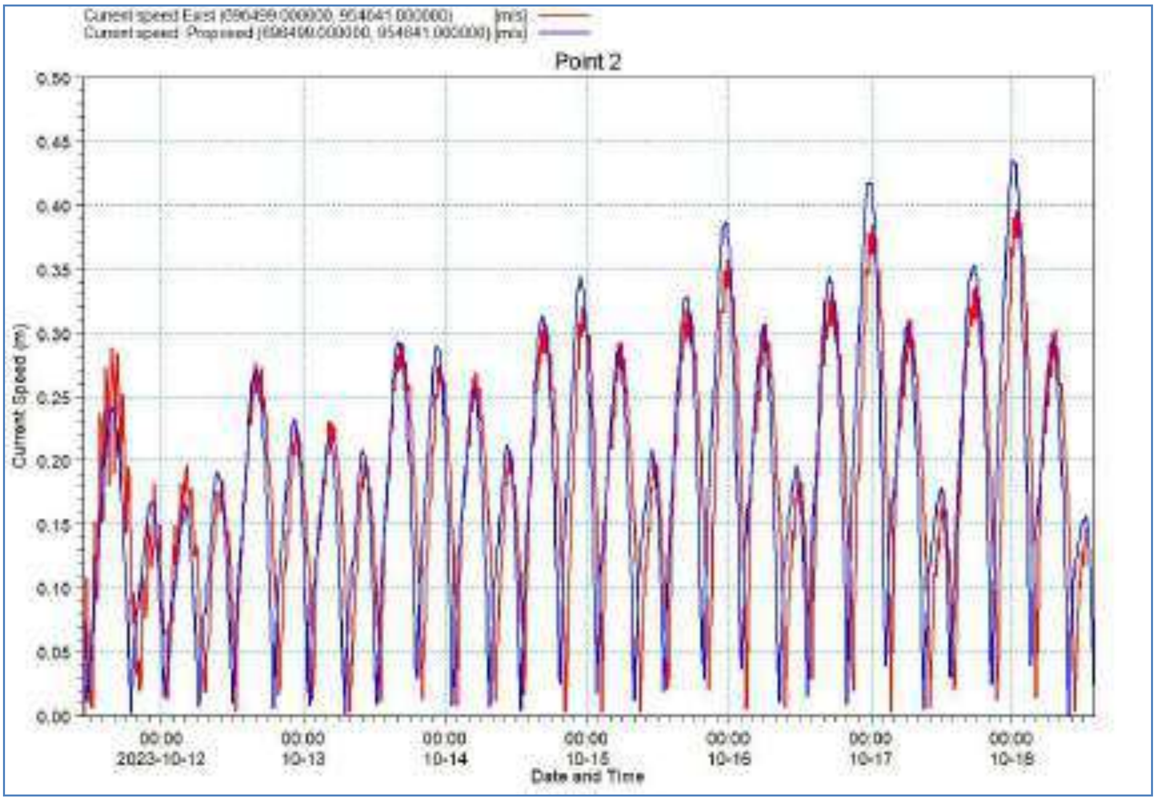

Fig. 31 Comparison of Currents at Point 2 in existing and proposed condition during non-monsoon period

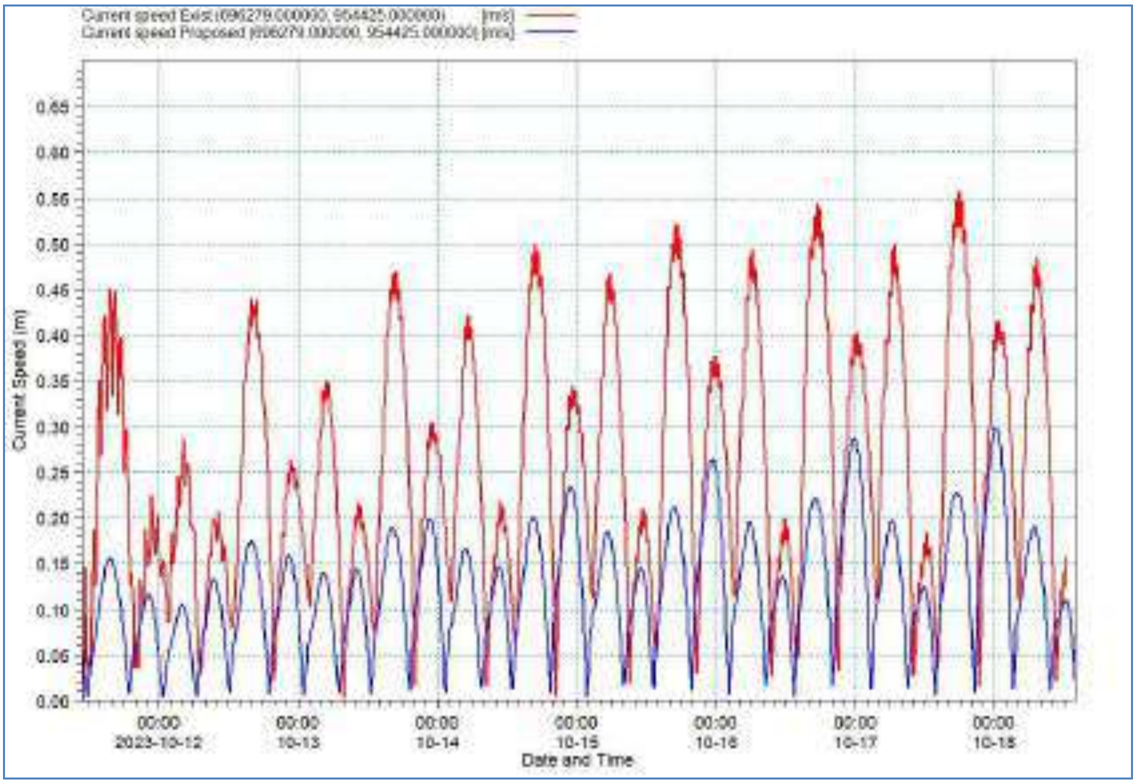

Fig. 32 Comparison of Currents at Point 3 in existing and proposed condition during non-monsoon period

It could be seen from above figures that magnitude of currents at point 1 and point 2 decreased in proposed condition and at location point 3 currents increased due to channelization of flow between breakwaters. At point 1 which lies near bridge, average currents in existing and in proposed condition are 0.3m/s and 0.2m/s respectively in non-monsoon period. At point 2 which lies in middle of the navigational channel average currents in existing and in proposed condition are 0.3m/s and 0.35m/s respectively. At location point 2 currents slightly increased in proposed condition due to closing of Adani jetty. At point 3 which lies at the entrance of channel average currents in existing and in proposed condition are 0.45m/s and 0.2m/s respectively in non-monsoon period.

9.2 Sediment transport studies for OPL

Sediment transport model was simulated for 15 days for proposed condition after incorporating optimum layout (OPL). The approach channel is dredged to -6m below chart datum (CD). The model was simulated for two river discharges conditions i.e. monsoon and non-monsoon conditions. Sedimentation patterns after 15 days of simulation are shown in Fig.33 and Fig.34 for monsoon and for non-monsoon period respectively. During monsoon condition (Fig.33), deposition can be observed in the approach channel of the order of 10cm annually. It could be seen from Fig.34 that slight sedimentation is observed during low flows through the approach channel.

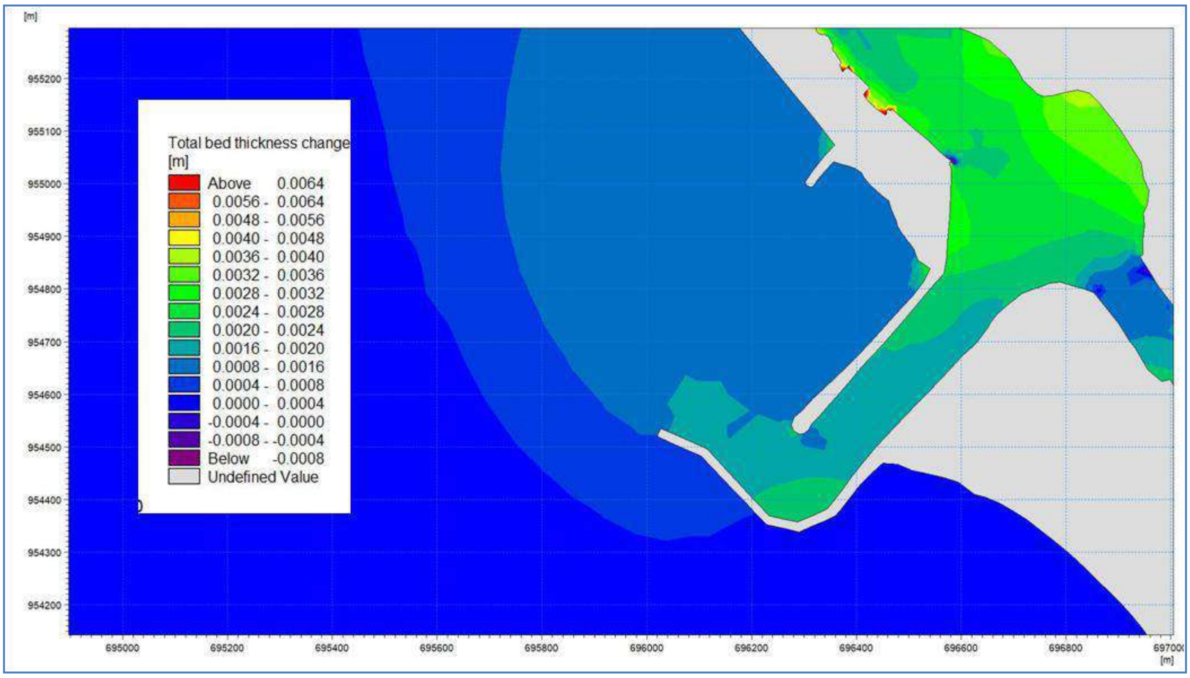

Fig.33 Sedimentation pattern in proposed condition during monsoon

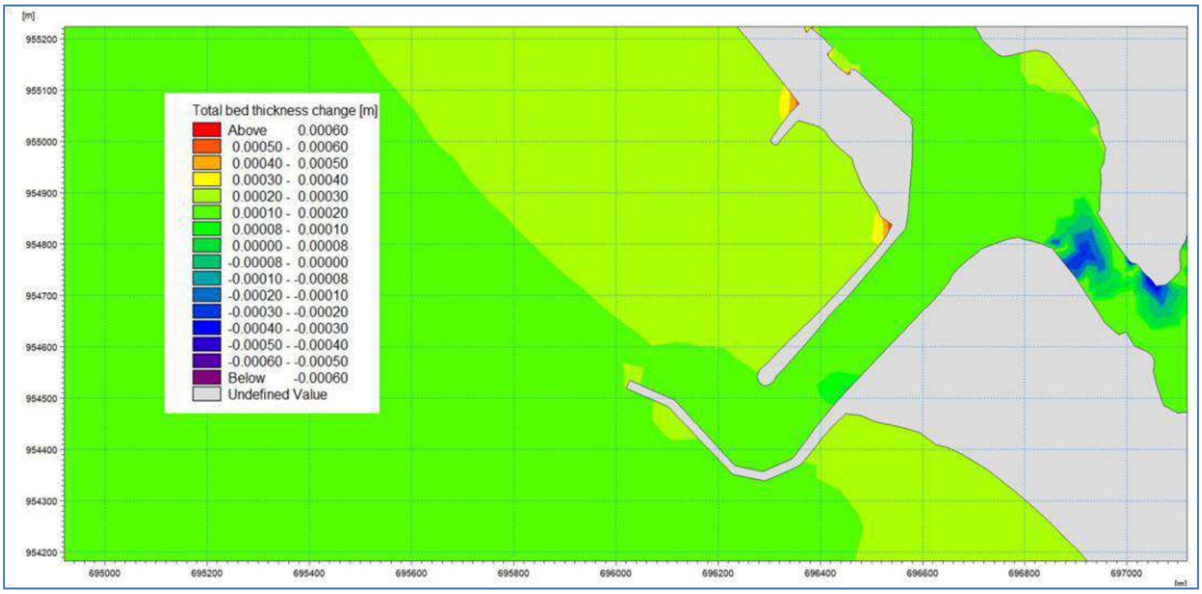

Fig.34 Sedimentation pattern in proposed condition during non-monsoon

10 Shoreline evolution with 10 groynes

The LITLINE model was simulated with 10 groynes as proposed and reported by the project authority. The model was simulated for a period of thirty years. From the results it can be observed that there is no erosion occurs in between the proposed groyne field (10 groynes) due to presence of the existing seawall. It is likely that beyond the proposed groyne field some erosion may occur which can be mitigated by providing the tapered groyne field along with the seawall.

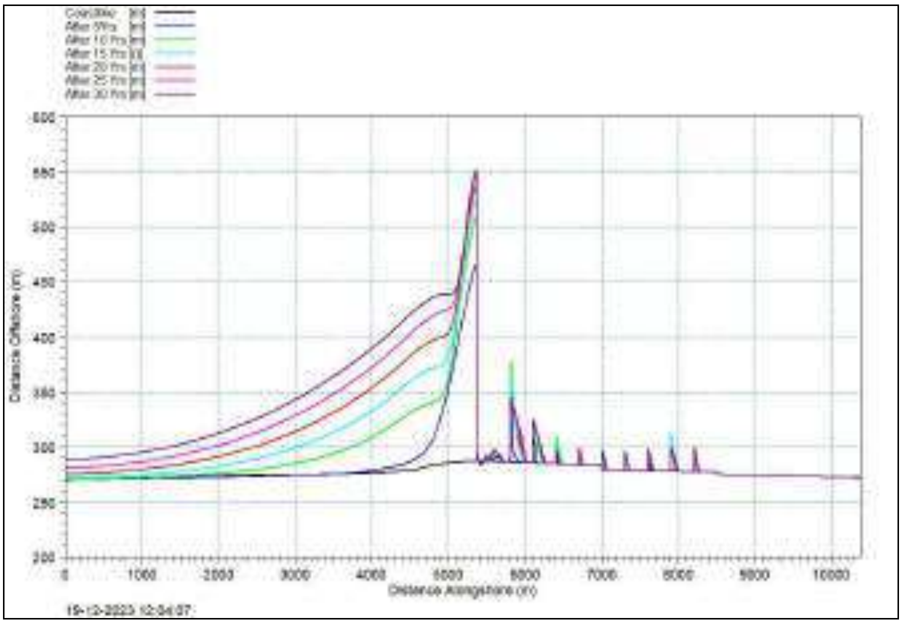

Fig.35 Change in shoreline with groyne field (10 nos.) after 30 years of model simulation

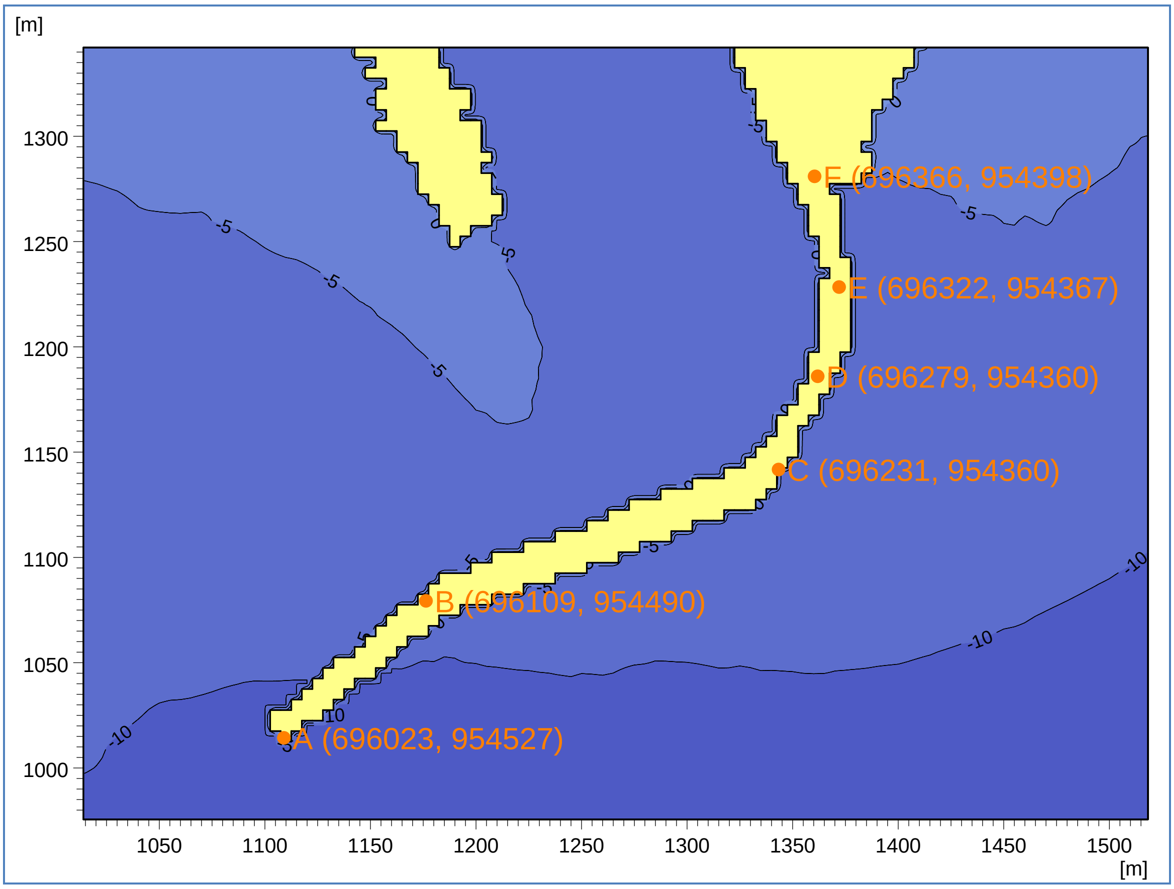

Fig. 36 Recommended layout with co-ordinates at Muthalapozhy site

Table 8 Easting and Northing of proposed extension of southern breakwater of recommended layout

Sl No.	Location	Easting	Northing
1	Point A	696023	954527
2	Point B	696109	954490
3	Point C	696231	954360
4	Point D	696279	954360
5	Point E	696322	954367
6	Point F	696366	954398

11. Conclusions

Following conclusions are drawn from the model studies for wave propagation, littoral drift, hydrodynamic and sediment transport:

1. Littoral drift distribution and shoreline evolution studies indicated that there is no need of sand bypassing for the next 25 years after the construction of the proposed optimised breakwater.
2. The simulated results and the findings there of will get affected if any construction or modification in the shoreline takes place in the 5km vicinity of the proposed site.
3. Wave tranquillity studies were carried out with optimized layout and it was observed that it achieved tranquillity at the entrance.
4. From hydrodynamics study it was observed that current are parallel to the contours and the magnitude of current in existing condition near bridge is of the order of 0.3m/s which reduced to 0.2m/s with proposed development.
5. Sedimentation studies reveal that there is negligible siltation takes place in the existing condition whereas 10 cm siltation was observed in the dredged approach channel annually under the proposed condition.
6. With the optimized layout there is no adverse impact from wave tranquillity as well as hydrodynamic point of view, hence the layout may be recommended.
7. Based on the model studies the recommended layout is depicted in Fig.36 and the co-ordinates in terms of easting and northing of the proposed extension of the southern breakwater is enumerated in Table 8.
8. Results and conclusions are based on the field data observed at site and supplied by the project authority. The model results may vary if the prevailing condition at the site changes significantly.