APPLICATION OF MATHEMATICAL MODELING IN OPTIMIZING LAYOUT OF A LARGE INDUSTRIAL FISHERY HARBOUR

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Abstract

Mathematical modelling of nearshore oceanographic conditions was carried out, in connection with a large industrial fishery harbour planned to be implemented in a coastal stretch protected by two reefal systems, on the western coast of Sri Lanka. MIKE 21, a two dimensional mathematical modelling system with a wide range of coastal engineering applications, and LITPACK, a one dimensional coastal processes modelling system, were extensively used in this study. The major features of modelling were the successful simulation of wave transmission over reefs and wave overtopping mass flux induced currents behind the innermost reef.

The validated model under existing conditions was used to optimise the harbour layout with safe navigational access, acceptable wave agitation within the basin and adequate sediment by-pass capacity at the entrance. The wave and other hydrodynamic parameters for the economical design of harbour breakwaters were also obtained. A well calibrated hydrodynamic model covering most of the western coast of Sri Lanka also evolved as a by-product of this study.

Introduction

Dikkowita is a small coastal town located on the western coast of Sri Lanka, 6 km north of the capital city, Colombo (Figure 1.). The coastal environment around Dikkowita is characterised by the presence of two distinct sandstone reefal systems (Figure 1.). The outer reef, known as the "Offshore Reef", is located at an average distance of 1 km from the

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coastline, about 2.5 m to 3 m below mean sea level (MSL). This reef has a significant gap in it fronting Dikkowita. The inner reef, commonly known as the "Secondary Reef", is generally located under or in front of the beach. This reef has got disconnected from the coastline south of Dikkowita and again merges with the coastline further northwards, forming a kind of a narrow basin. The maximum distance between the coastline and the secondary reef within this basin is about 200 m. The level of the secondary reef is quite variable consisting of exposed segments above sea level, and certain submerged segments as much as 4 m below MSL. The dual protection provided by these two reefal systems has made Dikkowita an attractive site for locating a coastal fishery harbour.

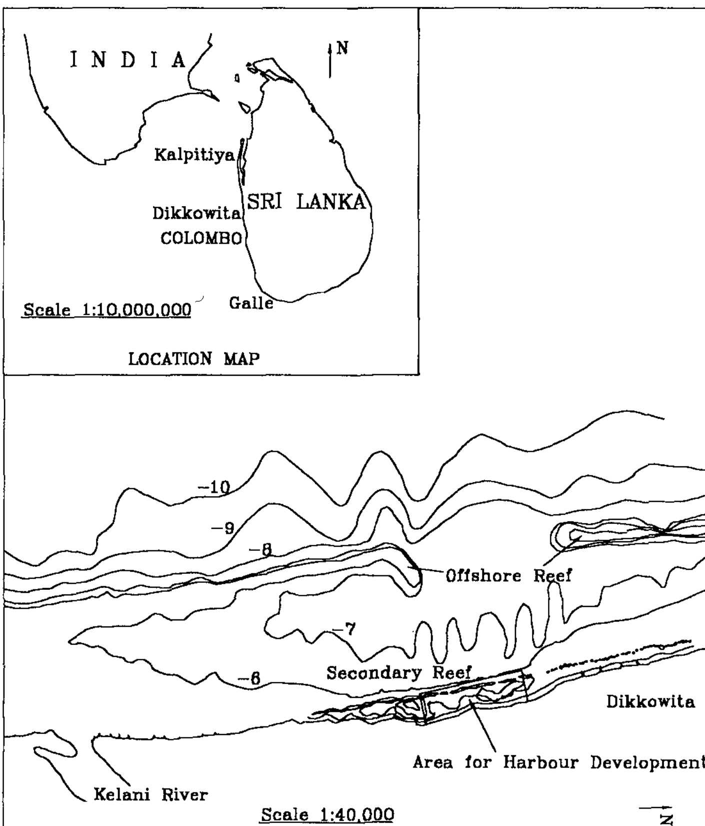


Figure 1. Coastal Environment Around Dikkowita, Sri Lanka

About 2 km south of Dikkowita lies the outfall of the Kelani River, which is the major source of sediment supply to a long coastal stretch northward of it. However, due to intensive sand mining taking place in the lower reaches, the sand supply from the river to the downdrift coasts has declined. Based on the findings of a National Sand Study (Delft Hydraulics and Netherlands Economic Institute, 1992) the present rate of sand supply may be estimated to be around 100,000 m^3/year compared to 400,000 m^3/year in 1961. This has led to severe erosion of down drift beaches and has required the implementation of coast protection measures.

Under the Asian Development Bank (ADB) assisted Sri Lanka Fisheries Sector Development Project, the site at Dikkowita was identified as having the potential to develop an industrial fishery harbour, which will cater to large deep sea fishing vessels. The technical feasibility study in this connection was carried out by Lanka Hydraulic Institute Ltd. (LHI), Sri Lanka in association with the Danish Hydraulic Institute (DHI) and PortConsult Consulting Engineers, Denmark. Coastal Engineering Investigations formed a major component of this feasibility study.

Mathematical Modelling

Most of the coastal engineering aspects were addressed through the application of MIKE 21 and LITPACK mathematical models developed at the Danish Hydraulic Institute (DHI), Denmark. In addition, certain numerical desk calculations were carried out to supplement the model computations. A substantial volume of data acquired through a dedicated field investigation campaign (LHI et al., 1996) and from previous studies (Behnsen, 1994 and LHI, 1994), was used in the setting up and validation of the mathematical models.

MIKE 21 is a two dimensional modelling system with a wide range of coastal engineering applications. A variety of computational modules are available within MIKE 21 for simulating hydrodynamics, wave dynamics, sediment transport, advection-dispersion processes, etc. LITPACK is a one dimensional coastal processes modelling system mainly used for computation of sediment transport and associated beach profile and coastline changes.

Offshore Wave Climate and Extreme Waves

The offshore wave climate within the study area is characterised by the presence of year round swell waves, approaching from the narrow directional band 210° - 230° north and locally generated sea waves strongly influenced by the south west monsoonal winds. Based on findings of a wave study for the south west coast of Sri Lanka (Scheffer et. al., 1994) and on-site measurements (LHI, 1994), the wave climate representative around 15m water depth could be established. The average significant wave height of swell waves was found to be 0.9 m during the south west monsoon season (May to September) and 0.5m outside this period. The sea waves were seen to dominate during the south west monsoon season with an average significant wave height of 1.5 m. Apart from this, certain extreme occurrences of sea waves due to local depressional storms, with average significant wave height exceeding 3.5 m have been recorded during the intermonsoon period October to November.

The extreme occurrences of wave heights around 15m water depth were established through extrapolation of recorded significant wave heights. Several probability distributions (Fischer and Tippet, Weibull etc.) were fitted and monthly maximum wave heights as well as the actual distribution of wave heights were considered in arriving at the final estimates of extreme waves. The significant wave height (H_{m0}) and the mean wave period (T_{02}) of 100 year return period wave were determined as 5.47 m and 7.8 sec, respectively. The corresponding values established for 1 year return period wave were 2.82 m and 6.3 sec, respectively.

Nearshore Wave Propagation Modelling

Nearshore wave propagation modelling was carried out to establish spatial variation of wave climate within the reefal system for wave driven littoral current computations as well as for establishing wave parameters for the design of breakwater structures. For this purpose, two wave modules within MIKE 21 (NSW - Nearshore Spectral Wave and PMS - Parabolic Mild Slope) were used. Both these wave modules are based on an irregular and directional description of wave field and considers shoaling, refraction and energy dissipation due to wave breaking and bottom friction. The PMS module has the added capability to account for diffraction effects caused by coastal structures and bathymetric features.

A nested set up of NSW and PMS modules was employed with the NSW model extending offshore up to an average water depth of 15m, and the PMS model contained within it, with its offshore boundary positioned some distance outside the offshore reef. Both models were oriented with offshore boundary parallel to the general direction of shoreline in the study area (255° north). In the NSW model, the grid spacings used were 50 m in the direction parallel to offshore boundary and 10 m in the perpendicular direction. In the PMS model a 5m grid resolution was used in both directions.

The model computations with respect to wave transmission over the offshore and secondary reefs were verified using simultaneous wave recordings within and outside the reefal system. The model simulations were seen to over predict wave heights inshore of the offshore reef when compared with actual wave recordings, for moderate offshore wave heights. This discrepancy was seen to get reduced with increasing offshore wave height. The model parameters which were used to control wave transmission over the offshore reef were those which govern wave breaking.

In MIKE 21, wave breaking is governed by a maximum allowable wave height (H_m) given by:

$$H_m = \gamma_1 \, k^{-1} \tanh \frac{\gamma_2 kd}{\gamma_1} \qquad (1)$$

where, k is the wave number, d is the water depth and γ_1 and γ_2 are wave breaking parameters. The parameter γ_1 which control breaking due to wave steepness was set to 1.0 (Battjes and Jensen, 1978) and γ_2 which controls water depth limited wave breaking was set to 0.8 (Holutuijsen et al., 1989). The wave attenuation over a steeply sloping submerged reef cannot be simply described by conventional breaker parameters. The main reason for simulated wave heights being somewhat higher compared to actual wave recordings can be attributed to this reason. However, since simulations provided a conservative estimate of wave heights behind the offshore reef, allowing for this discrepancy as a margin of safety, further refinement of wave transmission computations was not effected.

The wave heights behind he secondary reef was found to depend on wave heights outside of it as well as the tidal elevation. Empirical relationships were obtained for this wave height dependency through the analysis of simultaneous wave measurements on either side of the secondary reef for different water level ranges. The model simulated wave heights were seen to closely follow these wave height dependencies (Figure 2.). In order reproduce this wave transmission over the secondary reef, it was necessary to artificially lower exposed segments of the reef, as model computation ceases once an exposed "land" grid point is encountered.

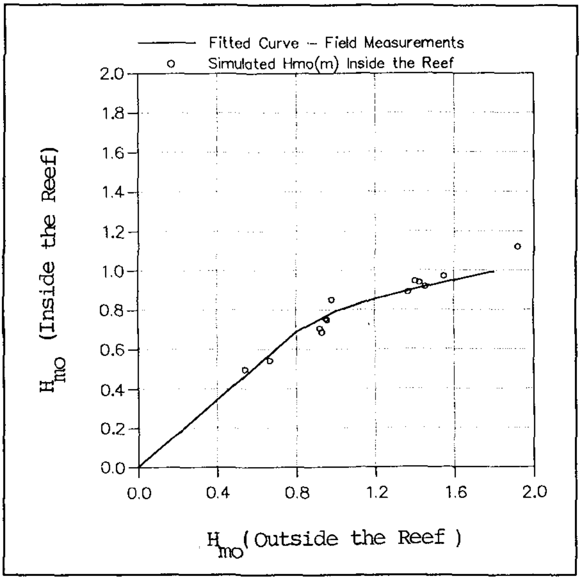


Figure 2. Verification of Wave Transmission Over the Secondary Reef H_{m0} (Outside the Reef) vs H_{m0} (Inside the Reef)

The calibrated wave model setup was used to simulate wave incidences which characterise year round wave climate as well as extreme wave occurrences. The simulation of 100 year extreme wave event is illustrated in Figure 3. The proposed harbour layout is also included in this illustration. The attenuation of waves over the offshore reef and penetration of waves through the gap in this reef is clearly seen from the significant wave height contour pattern.

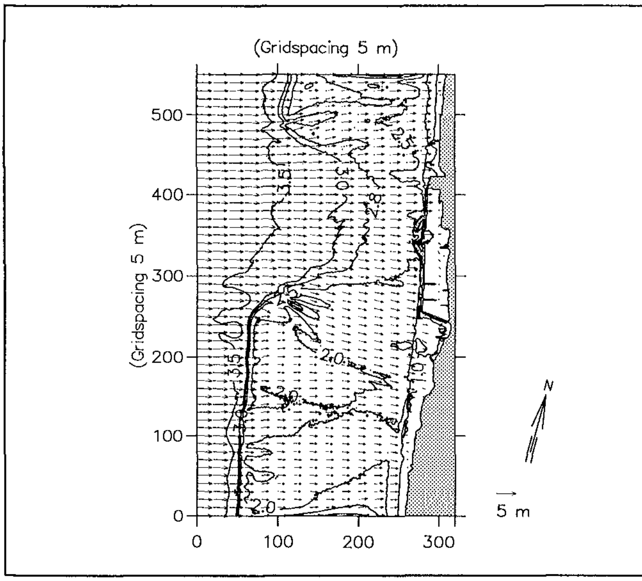


Figure 3. Simulation of 100 Year Return Period Wave at 0.0 m MSL

Hydrodynamic Modelling

MIKE 21's hydrodynamic module (MIKE 21 HD), based on the finite difference solution of full non-linear equations of conservation of mass and momentum integrated over the vertical was used to establish hydrodynamic conditions within the study area, under pre and post harbour construction stages. The hydrodynamics off the western coast of Sri Lanka is characterised by weak tidal velocities combined with intermittent wind influences. The average tide and wind combined flow velocities are in the range 0.15 to 0.25 m/s. The variation of tidal range along the coast is also found to be marginal with about 0.7 m during spring tide and 0.15 m during neap tide. In order to establish compatible boundary conditions for hydrodynamic modelling in such an environment, it was necessary to originate hydrodynamic modelling from a "Regional Model" covering a large sea area (Gunaratna et al., 1997).

The bathymetry and location of the Regional Model, which spans across most of the western coast of Sri Lanka is illustrated in Figure 4. This model was 230 km x 71 km in extent and was set up on a 1000 m x 1000 m grid. The model boundary conditions for the simulation of tidal flows was obtained through a detailed analysis of tidal wave propagation pattern of principal tidal constituents in the Indian Ocean (Gunaratna et al., 1997). The model computations were validated by comparison with tidal data available from a number of stations alongshore. Having validated the model for tidal flows, certain wind driven flow events were simulated by adjusting the wind friction coefficient.

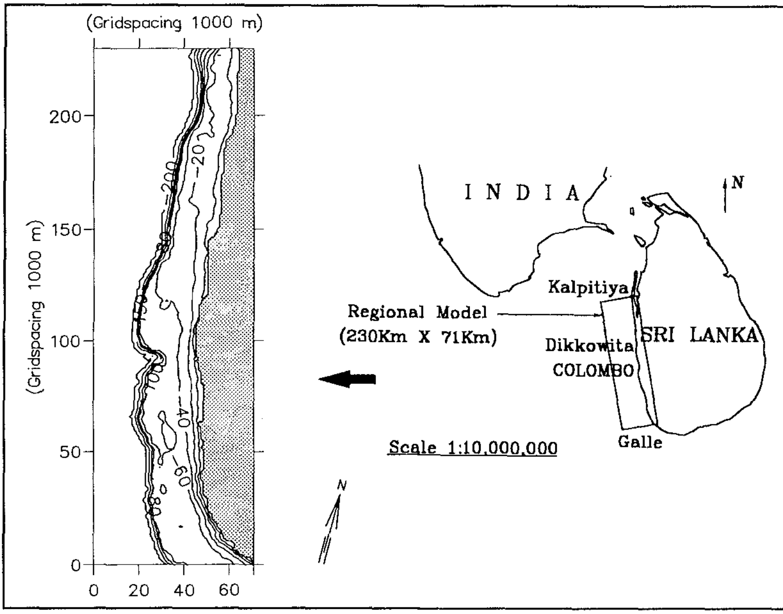


Figure 4. Regional Hydrodynamic Model

The hydrodynamics within the study area were simulated by stepping down from the Regional Model through a set of nested sub-models. The basic details of this model setup is given in Table 1. The "transfer boundary conditions" for carrying out simulations within a sub-model were extracted by simulations within the next highest level model. The simulation of wave overtopping currents and littoral currents were carried out in the lowest level HD2 and HD3 sub-models, respectively.

Hydrodynamic Model	Size	Grid Resolution	Time Step
Regional (HDR)	71 km x 230 km	1000 m x 1000 m	360 sec
HD0	25 km x 13.75 km	125 m x 125 m	90 sec
HD1	4.35 km x 2.75 km	50 m x 50 m	60 sec
HD2	2250 m x 600m	10 m x 10 m	30 sec
HD3	900 m x 500 m	5 m x 5 m	2 sec

Table 1. MIKE 21 Hydrodynamic Model Set Up

In the modelling of wave overtopping induced currents, the wave overtopping mass flux over reef segments was assumed to be described by the following expression for a discharge in the wave direction in the roller of a spilling breaker (Fredsøe, et al., 1992)

$$q = \frac{k_1 H_{m0}^2 \cos\beta}{T_{02}} \tag{2}$$

where, q=wave overtopping mass flux per unit length; and H_{m0} , T_{02} are the significant wave height and mean wave period, respectively, outside the secondary reef; β is the angle of wave incidence; and k_1 is a non-dimensional calibration coefficient.

The simulations were carried out at HD2 model level by employing a strategically synthesised combination of point sources behind the secondary reef that represent wave overtopping mass fluxes over reef segments. An identical number of point sinks were introduced on the other side of the reef segments for conserving the mass. An instantaneous picture of the simulated current pattern is illustrated in Figure 5. Simultaneous instrument recordings of two dimensional currents inside the reef and directional waves outside the reef were used as the basis for validation of model computations (Figure 6.).

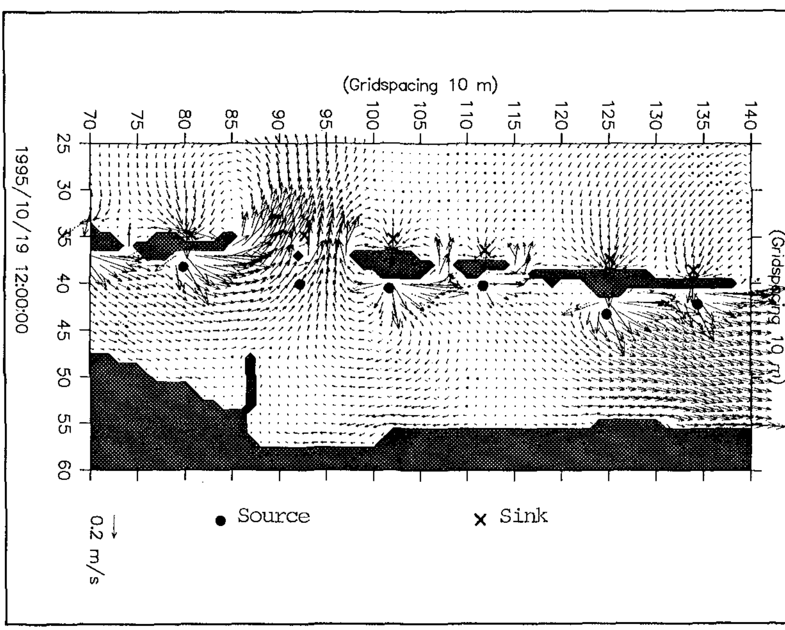


Figure 5. Simulation of Wave Overtopping Currents Behind the Secondary Reef

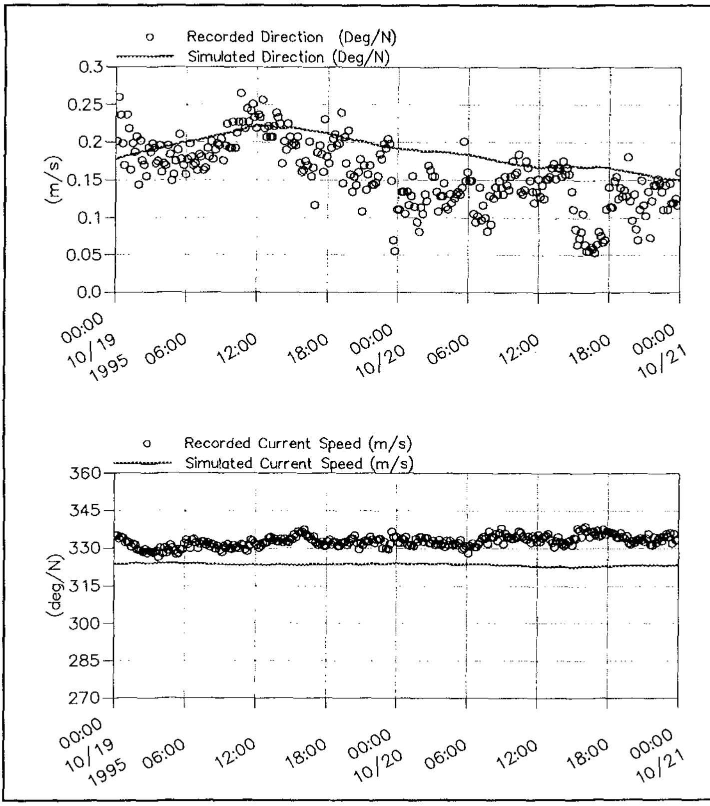


Figure 6. Simulated and Recorded Currents Behind the Secondary Reef

Optimization of Harbour Layout

A preliminary harbour layout was conceptualised by incorporating an optimal basin area of 11.7 hectares (117,000 m^2) behind the secondary reef into the harbour basin. The secondary reef itself was to function as the outer toe of the main breakwaters connected by return breakwaters to land. The harbour was planned to accommodate 400 fishing craft ranging in size from 40 feet (12.2 m) to 100 feet (30.5 m).

The calibrated hydrodynamic and wave propagation models under existing conditions were used as the primary basis, to simulate post-harbour construction scenarios through additional model computations. The remaining sections of this paper outline the wave disturbance, sediment transport and sedimentation computations carried out for layout optimization.

Wave Disturbance Modeling

The wave conditions within the harbour basin and harbour entrance was modelled using MIKE 21's Boussinesq Wave Module (MIKE 21 BW). MIKE 21 BW is primarily used for modelling of wave disturbance within harbours due to penetration of regular and irregular wave trains, taking into account shoaling, refraction, diffraction, and partial reflection and absorption of wave energy by harbour structures. The input wave conditions for wave disturbance modelling were obtained by NSW/PMS model simulations.

The simulations for selected swell and sea wave incidences which characterise the year-round wave climate were carried out within the BW model set up incorporating the entire harbour basin and the approach channel. The overall wave heights were derived by combing sea and swell wave heights:

$$H^2_{m0,overall} = H^2_{m0,swell} + H^2_{m0,sea} \tag{3}$$

where, $H_{m0,overall}$, $H_{m0,swell}$ and $H_{m0,sea}$ are the overall, swell and sea significant wave heights, respectively.

Having established the annual wave height exceedances from model simulations, the internal layout of the harbour and the entrance configuration was suitably modified to maintain wave disturbance within acceptable limits. The allowable significant wave height limits (not to exceed 1 week per year) used were 0.4 m to 0.6 m for loading/unloading operations at a quay and 0.6 m to 0.8 m for safe mooring in the basin.

In Figure 7, the significant wave heights for a wave incidence with an estimated exceedance of 1 week per year is illustrated for the optimised harbour layout. It was necessary to effect several modifications in the basic layout to limit wave disturbance within acceptable limits. The modifications carried out included, the provision of a forebay area at the entrance and a sandy beach fronting the entrance for energy dissipation and sloping rubble mound faces as much practically possible as the internal faces of harbour structures for effective wave energy absorption.

In addition to wave disturbance due to short period waves, the possibility of harbour resonance due to long period oscillations was also examined by using a combination of MIKE 21 BW and HD models. Several simulations carried out with different wave periods indicated that undesirable wave heights due to long period oscillations will not result within the harbour basin.

At the entrance, curved breakwater arms extends up to a water depth of 5.4 m below MSL. The main operational area of the harbour is to be dredged up to 4.5m below MSL while in the southern part of the basin a depth of 3 m below MSL is to be provided. The model simulations confirmed that under the wave conditions that may be expected throughout the year, these water depths are sufficient to ensure safe navigation into and out of the harbour and safe operation within the harbour for the range of anticipated craft sizes.

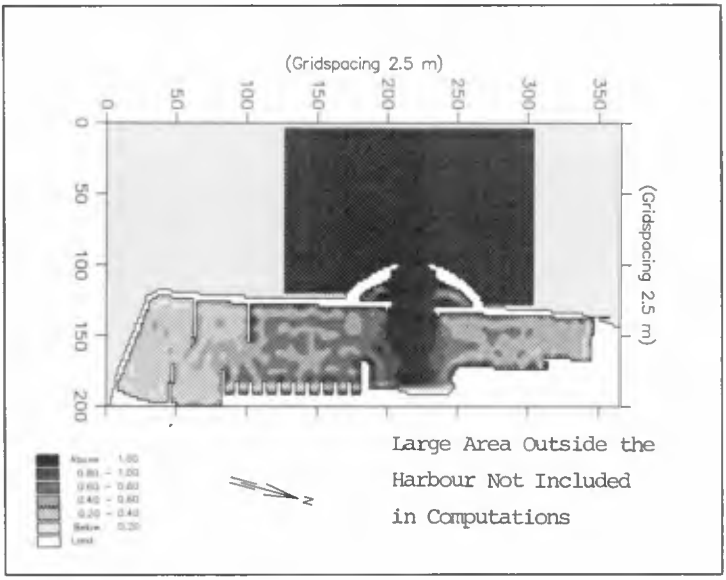


Figure 7. Simulated Significant Wave Heights Exceeded 1 Week per Year

Sediment Budget Computations

A sediment budget for the study area was obtained in the initial phase by quantifying different components as accurately as possible. The establishment of the sediment budget was based on LITPACK model computed sediment transport capacities across selected transects, volumetric bed level change computations based on repetitive surveys and available information on sediment supply from the Kelani River.

The analysis of all available sediment data from the study area and sediment transport mechanisms led to an identification of four distinct transport zones. These were the inner trough area behind the secondary reef, outer trough between the two reefs, the wave breaking zone on the outer slope of the secondary reef and the area around the Kelani River mouth. This analysis also identified that the average characteristics of sediments in motion due to longshore currents can be represented by that of a graded sediment mixture with a mean grain size (d_{50}) of 0.6 mm.

LITPACK model computations carried out using this information revealed that sediment transport capacities northwards and southwards of the proposed harbour site, where the secondary reef has merged with the coastline, are around 200,000 m^3/year. This is in excess of the present estimated sediment supply rate from Kelani River of 100,000 m^3/year. Therefore, progressive erosion of fine material can be expected to take place until bed armouring result in an equilibrium situation.

LITPACK computations indicated variable sediment transport capacities along the stretch where secondary reef has got disconnected with the coastline, but in general being in excess of the sediment supply rate. Diver inspections of the secondary reef carried out in this area revealed that it consists of fragmented sandstone with hardly any sediment accumulations within the crevices. This observation itself is an indication of the sediment transport potential being higher than the sediment supply rate.

The volumetric bed level change computations showed that on an average about 3,500 m3/year of sediment is lost from the basin area planned to be occupied by the harbour. This figure represents the net difference of sediment eroded by the wave overtopping currents and the sediment spilling over the secondary reef into the basin. The construction of the harbour will result in a loss of these erosion products from the basin area to down drift coasts. However, such a moderate deficiency in sediment discharge will be unlikely to create significant erosional problems.

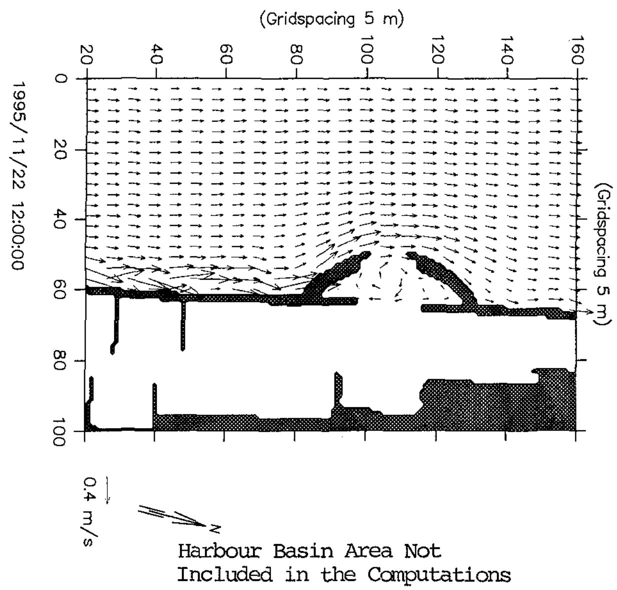
Assessment of Sedimentation Impacts of the Harbour

Two dimensional sediment transport computations were carried out for the post-harbour construction stage to ascertain whether a sufficient sediment transport capacity exist in front of the harbour entrance to by-pass the incoming sediment supply. For this purpose, MIKE 21's sediment Transport Module, MIKE 21 ST was used.

MIKE 21 ST computes sand transport capacity at each node of a rectangular grid computational domain for which hydrodynamic and wave characteristics have been established. The model could account for effects of both breaking and non-breaking waves on transport of non uniform spatially varying sediment sizes. The currents could be tidal, wind-driven, wave-driven or a combination of the three. The grain size and the gradation of sediments may vary throughout the model area.

The hydrodynamic scenarios for computing sediment transport considering selected wave incidences combined with tide and wind driven flows were established within MIKE 21 HD3 model (Figure 8.). The computed sediment transport potentials for the selected wave incidence scenarios were extrapolated considering their seasonal occurrence percentages and the fractional contribution to the transport potential as indicated by LITPACK computations. These calculations revealed that the entrance configuration of the proposed harbour with curved double breakwater arms will be able to by pass the present estimated sand supply of 100,000 m3/year. This also implies that there would be no additional lee-side erosion due to the construction of the fishery harbour, apart from the first few years, which is required for the adjustment of bathymetry adjacent to the entrance.

The sedimentation within the harbour basin was assessed through desk calculations, considering three basic mechanisms of sedimentation, viz., tidal exchange, eddy exchange and long period oscillations. The calculations made under conservative assumptions estimated the annual siltation within the harbour basin to be about 5 cm. The long period oscillations was found to be the most important mechanism contributing to about 80 percent of the estimated siltation.



Tide and Wind Driven Flow Combined with the Wave Incidence:
H_{m0} = 1.25m, T_{02} = 11.2 sec and Mean Wave Direction = 202.5° from north at -15m MSL

Figure 8. A Simulated Hydrodynamic Scenario for Sediment Transport Computations

Concluding Remarks

The hydrodynamic, wave and sediment transport phenomena in a complex reef protected coastal stretch could be successfully simulated through the application of MIKE 21 and LITPACK mathematical models. The study also resulted in the evolution of a hydrodynamic model for the western coast of Sri Lanka. This model which has undergone further refinement and validation since its initial development, has been subsequently used as the primary basis in a number of costal engineering applications at other locations.

Acknowledgments

The work described in this paper was conducted as a part of the technical feasibility study for the proposed fishery harbour at Dikkowita, Sri Lanka. The authors of this paper wish to thank Ministry of Fisheries and Aquatic Resources Development, Sri Lanka, for providing the opportunity to be involved in this study, conducted under the ADB Sri Lanka Fisheries Sector Development Project.

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