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A METHODOLOGY FOR THE ENGINEERING DESIGN AND ANALYSIS OF FORESHORE BASINS FOR BEACH EROSION PROTECTION

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ABSTRACT

The conceptual design of a beach as a resort amenity is brought one level higher by considering a beach to be part of a foreshore basin. The basin is designed as an engineered tide pool to promote deposition of sediments on the beach while satisfying an imposed requirement on the frequency of seawater replenishment. The basin structure is designed to withstand wave loadings from tropical storms, while meeting the requirements for seawater replacement frequency, suitable depth for beach swimming, and the need to promote sediment deposition within the basin. The methodology proposed includes the procedure for tide penetration analysis, sediment stability analysis and the hydraulic analysis and engineering design of the basin structure against typhoon-induced wave loadings.

Keywords: *foreshore basin, beach, tides, waves, erosion*

1. INTRODUCTION

The Philippine archipelago is endowed with many excellent natural beaches along its roughly 36,000-km long coastline. However, a large portion of these coastlines are in a state of unmitigated erosion due to both natural and anthropogenic causes. Natural causes include the existence of light and hence easily erodible coastal sediments, adverse offshore bathymetry, tropical storms, high tidal range, and increased water levels due to global climate change. Anthropogenic causes include the existence of steep-faced and highly reflective coastal structures, and unplanned, often ill-designed, modifications of the foreshore seabed. Some recent cases of erosion in Philippine coasts are discussed in Cruz (2009).

A beach coast provides excellent natural protection against waves and currents. It protects the foreshore area from the combined erosive action of high tides and large waves. It also armors the shore from the destabilizing swash motion of broken waves that reach the shore. It also effectively limits the rate and frequency of wave overtopping that contributes significantly to the degree and extent of coastal inundation of low-lying backshores and the inland.

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The shallow waters over the beach induce approaching waves to break on it and thereby dissipate their energy before they otherwise expend their high energies and impact forces on the inland soil or coastal structure. Sediments on the foreshore are slowly but progressively transported shoreward by prevailing waves to form the beach face we see in summers. These same sediments are carried seaward by the higher waves generated by storms to form longshore bars in the following non-storm season. Such bars act as natural sills on which the lower waves break and promote milder inshore waves which allow the beach to form. This cycle allows beachgoers to enjoy the scenic beaches during summers. Over a typical year, a stable beach would ideally have zero net cross-shore transported sediment volume.

Astronomic tides are another forcing that perpetually influence the occurrence of erosion along a coast, be it a beach or bluff coast. Tides result from the gravitational forces between the sun, moon and Earth. At any location, however, these forces are, in turn, affected by the presence of land masses as well as by the earth's rotation. Due to their periodic nature, tides are actually considered to be water waves with periods that are several times longer than ordinary swells or wind waves. Their amplitudes, however, are usually much lower. Depending on location, tides generally have typical periods of about 12.54 hours, much longer than swells or wind waves, which are typically in the range of about 5 to 10 seconds for Philippine coastlines. For a tropical archipelago like the Philippines, tide amplitudes can be amplified by the existence of proximate islands, which can further aid in the erosion of the beach coast. Tides occur unceasingly and during high tides, waves are enabled by the raised sea surface to move further landward and thereby potentially erode a larger area of the beach.

While tides can be used by waves to erode the coast, they act as a natural mechanism to replenish the seawater along the foreshore. Due to their much longer period compared to sea waves, tides facilitate the periodic but gradual replacement of what would otherwise become stagnant waters in the foreshore. Seawater brought by tides to the shore can be trapped, as in tide pools, and retained to sustain various ecological functions, but must be released and replenished to maintain its original quality and other physical characteristics that are needed to support these functions.

Where a coast can potentially erode and where the continuous exchange of seawater is desirable for ecological and environmental functions, as in the case of a beach resort, a foreshore basin may be an ideal engineering intervention to simultaneously mitigate the beach erosion and allow natural rejuvenation of the foreshore waters. Such marine pool, when built as an amenity to a sea resort, can actually function as a beach protection structure by allowing only waves of smaller heights to reach the foreshore and thereby promote the accretion of fine-grained beach materials. At the same time, the structure can be designed so that waves and tides are admitted into the foreshore at prescribed durations to allow the natural renewal of seawater.

Engineered basins in the foreshore zone of storm-exposed coastal beaches have received increasing attention as dual-purpose protection structures of coastal resorts. While this engineered feature is marketed as a resort amenity for beach recreation, its main function is to mitigate an observed erosion of the beach. A foreshore basin is often designed as an engineered tide pool, in which the seawater brought by a rising tide is harnessed to replenish and re-circulate the otherwise stagnant waters of the enclosed basin. The basin wall is functionally designed to retain seawater under normal conditions of the foreshore, but can be thickened or reinforced as a storm-exposed seawall or nearshore breakwater to withstand cyclone-generated wave loadings.

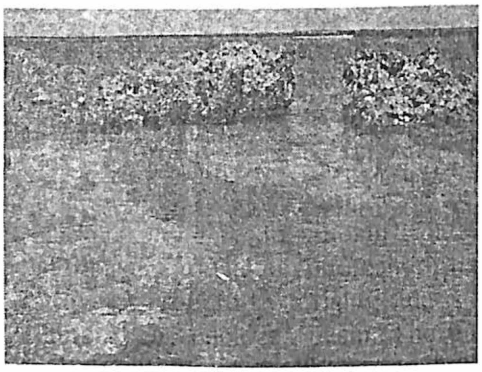
2. OBJECTIVES OF THE STUDY

This study aims to assess the traditional function of a foreshore basin and to support the analysis of such function by numerical computations of tide penetration into the pool. The final objective is to incorporate this analysis in an engineering methodology where the tides are considered in the hydraulic design of the foreshore basin structure. Since the methodology is intended to be applied to a foreshore pool in a storm-exposed coast, a subsequent numerical analysis of typhoon-induced wave loading is integrated into the methodology.

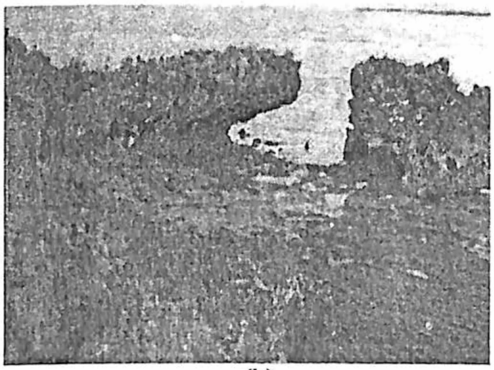
It is also the objective of this study to apply the methodology to an actual Philippine coastal resort project, in which the pool is exposed to high waves generated during the northeast monsoon season.

3. PHYSICAL DATA

Plans are underway for the construction of a foreshore basin along a coastline in storm-frequented Visayan Sea in Central Philippines. The project coast is along the eastern seaboard of Mactan Island in Cebu Province. While its adjoining sea is tracked by typhoons originating mostly from Pacific Ocean, the wave exposure of the project coast is considerably mitigated by Olango Island to the east. This project coast is significantly affected by the northeast-southwest monsoon seasons. The fact that this island is bounded by the other major islands of the archipelago is partially responsible for the relatively high tidal range at the coastline. Figure 1 shows the conditions of the foreshore area during 2 tide conditions, one high and other low tide. It is seen that at these 2 times, the spatial extent of exposure of the coast to wave action is markedly different. The mean tidal range at the project location is 1.02 m and the mean higher range (based on the mean of the higher of two high tides and mean lower low tides) is 1.53 m. The extreme tidal range, inclusive of storm surges during typhoons, is more than 2.46 m (Namria, 2009). At the time of this picture, the day's tidal range caused a shoreline migration of about 60 m, exposing the seagrasses and bottom sediments.



(a)



(b)

Fig. 1 Foreshore area alternately (a) inundated during high tide and (b) dry during low tide

The coast is made up of limestone (Figure 2), which allows efficient drainage of surface runoff from inland into the sea. However, the existing weathered limestone both on the seabed and along the coast, with its visual cover and jagged surface, has been assessed in a negative light in terms of the marketability of the proposed site as a beach area. The project intends to convert the existing steep-fronted rugged coasts and foreshore into a mildly sloping beach face to complement a resort town now under construction. The structure is primarily intended to promote beach formation, but is to be designed to also maintain seawater at all times with periodic replenishment. Finally, the foreshore basin is to be designed to protect the coastline from the high waves during storms that frequent the island.

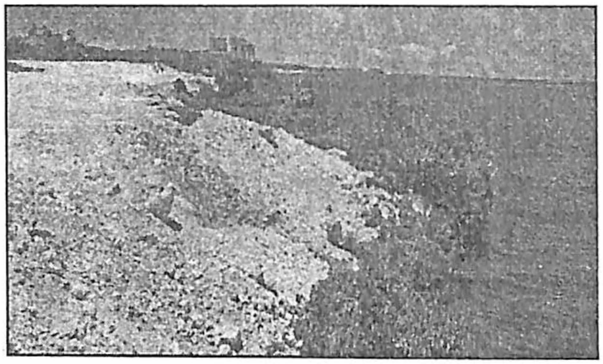


Fig. 2 Weathered limestone coast of study area

Figure 3 shows the time series of astronomic tides at the nearest monitored tide station of Namria. The combined time strips span 13 solar days, in which it is seen that the tide profiles progress from diurnal, semi-diurnal, mixed, then back to diurnal again. At this site, the mixed tides have the highest amplitudes among the three, while the semi-diurnal profiles possess the lowest amplitudes.

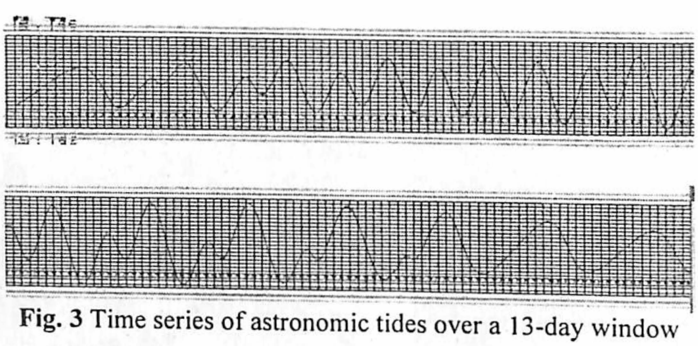


Fig. 3 Time series of astronomic tides over a 13-day window

Statistics of the observed tide elevations at this site, such as those shown in Figure 3, have been processed by NAMRIA and are published yearly (Namria, 2009). For this site, Table 1 presents the tide statistics. The tidal range, or difference between the MHHW and MLLW, is 1.53 m, which is slightly above the average values for most Philippine coasts. In tide penetration analyses of the foreshore basin, however, it is the peaks of the local high tides that determine the frequency of seawater replenishment of the basin.

Table 1 Summary of tide statistical properties

Tide statistics		Value
MHHW	Mean Higher High Water	+0.81
MHW	Mean High Water	+0.47
MTL	Mean Tide Level	0.0
MLLW	Mean Low Water	-0.55
MLW	Mean Lowerv Low Water	-0.72

4. FORESHORE POOL HYDRAULIC ANALYSIS

The crest elevation of the foreshore basin is treated as a hydraulic design parameter which is determined based on a specified desirable daily and aggregate durations of tide penetration into the basin. To analyze the penetration frequency, a time series of the local tides over a year is prepared and digitized. Figure 4 shows a 3-day window of the forecast local tides (Namria, 2009), where it is seen that the tide at the site is a mixed profile, although it is diurnal and semi-diurnal on other days. For this reason, the tide elevation datum used is the Mean Lower Low Water (MLLW). For a specified foreshore basin structure crest elevation, the duration when the water surface is above that elevation, or the so-called run length (encircled portion), is computed for the tide time series over an aggregate period of one year. Figure 4 shows a graphic illustration of the tide penetration analysis for a basin crest elevation of 1.41 m above MLLW.

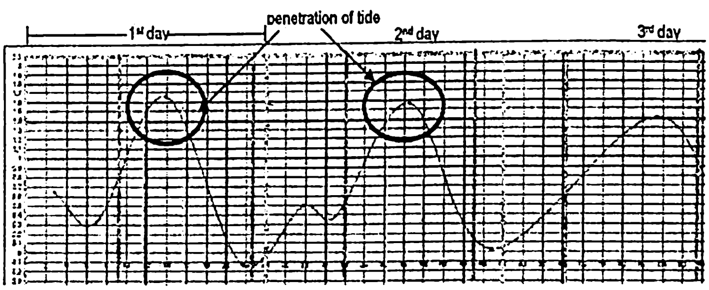


Fig. 4 Illustration of tide penetration analysis

Table 2 shows the resulting average run lengths and number of days of penetration (second and third columns) for each month. Although the run length is a natural measure of tide penetration, a more useful gage from the operational viewpoint is the penetration during the basin's likely period of usage, say 6 am to 8 pm corresponding to the time of usage by the resort's guests. The next two columns show the resulting monthly penetration days and durations for this limited period. These indicate, for example, that at the specified basin crest elevation, the tide penetration during the peak months of March to June will remain the same under the limited period. However, there are now two months when the pool will not renew its waters during this 14-hour period. Tide penetration tables similar to Table 2 can be prepared for other crest elevations, and the results are used to determine the optimal crest elevation for a specified period of probable use.

Table 2 Results of tide penetration analyses for basin crest elevation = MLLW +1.41m

2009	Number of days	Average run length (hrs)	Within 6am-8pm (days)	Average Hours
Jan	14	1.49	3	0.18
Feb	10	1.07	1	0.04
Mar	0	0	0	0
April	15	1.82	15	1.82
May	18	2.34	18	2.34
Jun	18	2.66	18	2.66
July	21	2.78	21	2.63
Aug	4	0.33	4	0.33
Sep	25	2.89	12	1.33
Oct	20	2.33	5	0.27
Nov	19	2.49	0	0
Dec	19	2.37	7	0.20

5. STABILITY ANALYSIS OF BEACH SEDIMENTS

The foreshore basin is designed to promote beach formation by creating a calmer wave climate within the basin. The degree of calmness is measured in terms of the spatial distribution of a suitable index of sediment stability vis-à-vis wave-induced waves and currents. To assess the potential of beach sediments to be dislodged by local waves in the basin, a hydrodynamic stability analysis is carried out based on the threshold of sediment movement for wave-induced fluid motion. Under the balance of drag and inertial forces induced by the fluid pressure distribution on a typical sediment grain, and gravity force on the sediment, the threshold of wave-induced sediment movement can be expressed in terms of the Shields parameter. It is thus possible to express the stability of sediments in terms of the residual grain size d_r, defined implicitly as follows:

$$d_r = \frac{u_b^{\,2}}{(s-1)g\psi_{cr}(d_r)} \tag{1}$$

where u_b is the fluid particle horizontal velocity at the seabed, ψ_{cr} the critical value of Shields parameter, s the dry specific gravity of sediment grains, and g the gravity acceleration, The critical Shields parameter depends on the residual size d_r . In general, the critical Shields value for wave motion is a function of the wave Reynolds number, and is usually empirically derived based on wave flume experiments (Nielsen, 1992). Van Rijn (1984) developed an empirical relation where the Shields parameter depends solely on the normalized grain size which, in turn, depends on the absolute sediment size, its specific gravity, and on the seawater viscosity.

Sediments of size at least d_r are hydrodynamically stable under wave action. Because the residual grain size appears on both sides of Eq.(1), it is necessary to develop an algorithmic procedure to determine d_r for known values of the sediment properties, wave-induced flow field, and empirical relationships of the critical Shields parameter for waves (Cruz, 2008). Sediment properties are based on laboratory measurements of soil samples taken at distributed points along the project coastline, while the bottom velocities are based on the velocity field induced by wave action, which accounts for wave breaking and other physical processes.

To determine this velocity field under wave action, the nearshore wave field is simulated using a nonlinear Boussiness-type model of wide frequency dispersion range (Cruz et al., 1997) described by the model equations:

$$\frac{\partial \eta}{\partial t} + \nabla \cdot [\mathbf{u}(h+\eta)] = 0 \tag{2}$$

$$\begin{aligned} &\frac{\partial \mathbf{u}}{\partial t} + (\mathbf{u}\cdot\nabla)\mathbf{u} + g\nabla\eta + \frac{h^2}{6}\left(\nabla\cdot\frac{\partial \mathbf{u}}{\partial t}\right) \\ &-(\tfrac{1}{2}+\gamma)h\nabla\left(h\nabla\cdot\frac{\partial \mathbf{u}}{\partial t}\right) - \gamma g h\nabla[\nabla\cdot(h\nabla\eta)] \\ &+\mathbf{F}_b + \mathbf{F}_s + \mathbf{F}_w = 0 \end{aligned} \tag{3}$$

where $\eta(x,y,t)$ is the water surface displacement, u = (u,v) the depth-averaged horizontal velocity vector, x, y the horizontal coordinates, t time, $\nabla = (\partial/\partial x, \partial/\partial y)$ the horizontal gradient operator, γ the dispersivity extension factor, $\mathbf{F}_b$ the wave-breaking energy-dissipation vector, $\mathbf{F}_s$ the structure-induced damping term, and $\mathbf{F}_w$ the bottom friction resistance vector. The input to the model includes the mean depths $h(x,y)$, initial values of the water surface elevations and velocities, wave conditions at the offshore boundary, and discretization parameters. The model solves for the unknown η and $\mathbf{u}$ at all grid points and time steps, from which the bottom velocities are determined.

The above model has been applied to a number of two-dimensional and three-dimensional wave propagation problems. Simulated wave quantities have been verified with relevant theories under the same idealized conditions, and with available empirical data or field measurements (Cruz and Aono, 1997; Cruz, 2002; Cruz and Aono, 1998; Cruz and Chen, 2006). The above numerical wave model is used to obtain wave quantities needed for the engineering design in the absence of available measured data from secondary sources such as government-run monitoring stations. It should be noted however, that such wave quantities can be determined from other valid nonlinear wave propagation models.

6. APPLICATION OF METHODOLOGY

The wave conditions needed to compute the nearshore wave fields are the height, period, and direction of the waves at the offshore boundary. These conditions are hindcast based on data of surface wind speeds and directions, effective fetch and wind duration (CERC, 1984). Table 3 shows the potentially critical offshore significant wave conditions obtained from this wave hindcast and using surface wind data from the nearest wind station of PAGASA.

Table 3 Offshore prevailing wave conditions

Wind Dir.	Wind Speed (m/s)	Annual Freq. (%)	Wave Height (m)	Wave Period (s)	Remark
NE	1-4	31.6	0.37	2.57	Prevailing direction
NE	5-8	1.7	1.04	3.99	3rd highest wave
ENE	5-8	0.1	1.37	4.41	2nd highest wave
SW	5-8	1.2	1.52	4.72	Highest wave

Figure 5 shows the water surface snapshot and wave height distributions simulated for the highest prevailing offshore wave approaching from the southwest (see Table 3). The plan-form design of the foreshore basin is clearly seen. Figure 6 shows the simulation results for the second highest offshore waves, approaching from the east northeast. These reveal that the basin's interior experiences calmer waters than its exterior, and that part of the incident wave energy is reflected seaward by the basin's structural boundaries. The results also indicate that the ENE approach, though with a lower incident wave, induces higher waves in the pool.

The wave fields above have been used to obtain the bottom velocities which, together with properties of the sediment at the site, yielded the distributions of the residual grain sizes in Figure 7. It is seen that the required sediment sizes in the basin are bigger for the ENE than the SW wave approach. The beach face, in particular, requires coarser than 10-mm (mean grain diameter) sediments. Also, the seabed requires large-size stones in front of the basin due to the local amplitude-increasing effect of reflected waves.

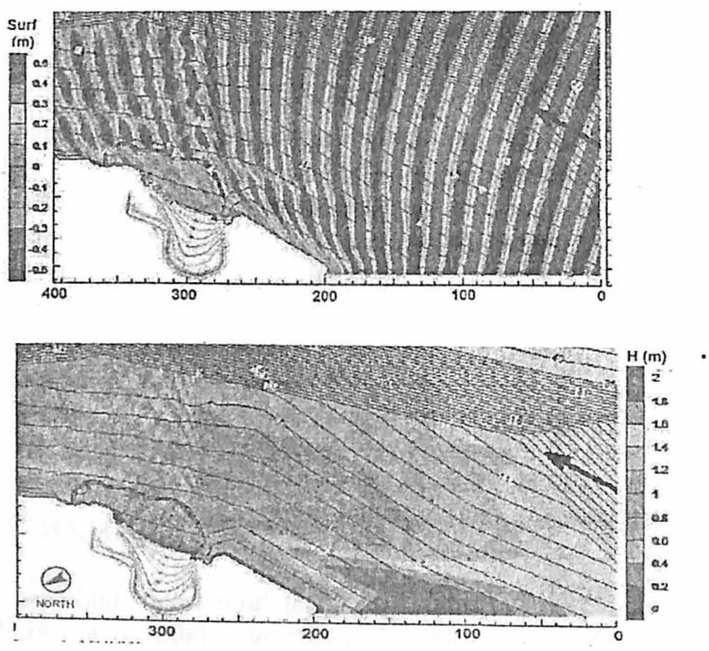


Fig. 5 Simulated wave fields of highest prevailing waves from SW at high tide:
(a) Transient water surface, (b) resulting wave height distribution

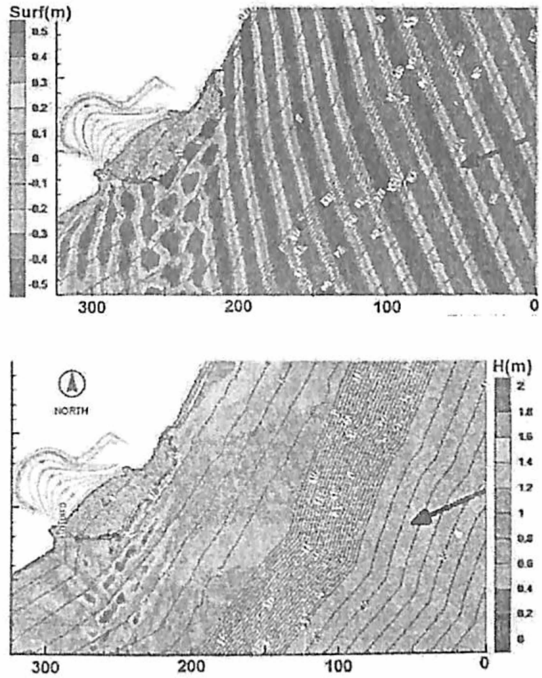


Fig. 6 Wave fields of highest prevailing waves from ENE at high tide:
(a) transient water surface, (b) wave height distribution

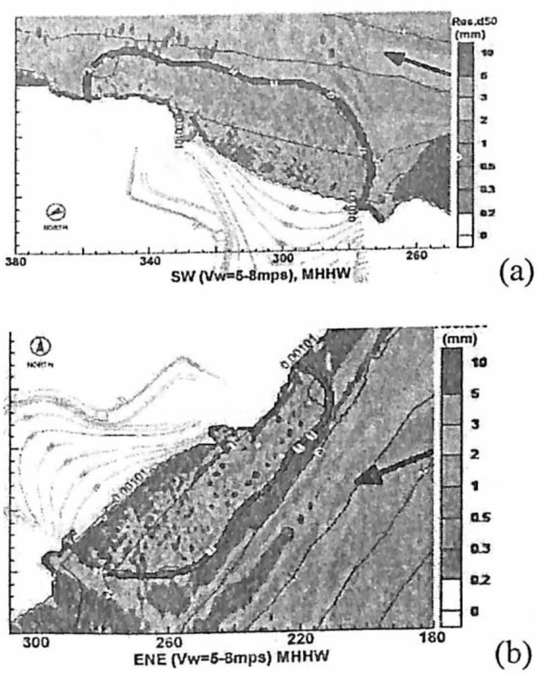


Fig. 7 Residual grain size due to prevailing waves from:
(a) SW, and (b) ENE, both at high tides

7. ENGINEERING DESIGN OF POOL

Tropical storms generate high waves that approach a distant coastline without being dissipated until they break. When storms turn into typhoons with gust speeds exceeding 32 mps, the potential damage can be catastrophic. Historical typhoons that generated the strongest winds in 3 approach directions are summarized in Table 4.

Table 4. Data of strongest historical typhoons that tracked close to project area

Cyclone	Date	Highest Wind (mps)	Dir
Typh. Nitang	Sept-02-1984	48	NE
Typh. Ruping	Nov-12-1990	55	S
Typh. Bising	Apr-04-1994	30	SW

Including the typhoon tracks, Figs. 7and 8 respectively show the simulated wave fields Typhoons Nitang and Bising, which generated waves approaching from the NE and SW respectively. In these simulations, the storm surge is incorporated as an uplifted sea surface at the highest recorded level, HWL. Due to the higher offshore waves, longer wave periods and deeper crest, wave refraction and wave breaking both occur earlier in deeper waters so that higher percentage reduction of wave energy takes place prior to the breakwater. Additional energy dissipation is caused by wave breaking on the crest and turbulent entrainment within the porous structure. The simulated wave heights are synthesized to obtain the design wave height.

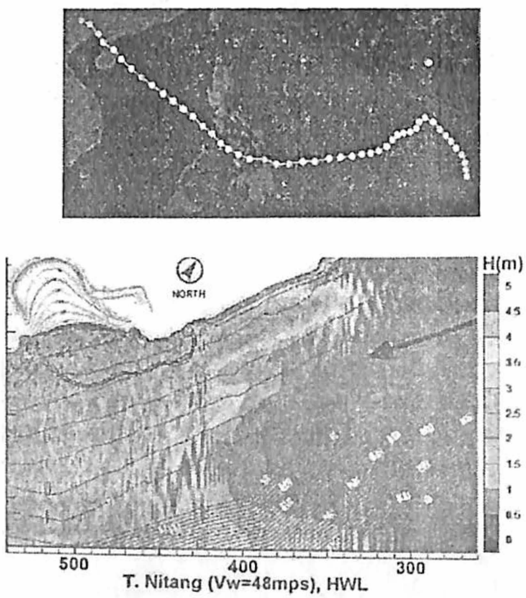


Fig. 8 (top) Track of Typhoon Nitang, (bottom) simulated wave height distribution

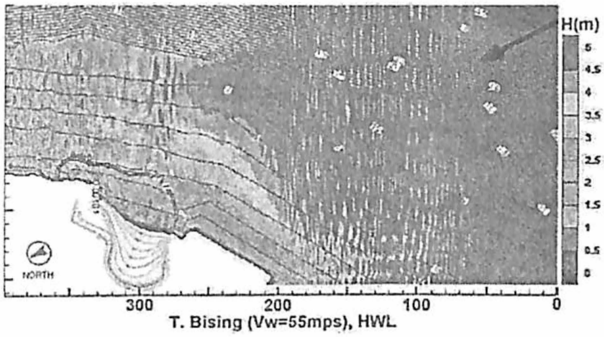


Fig. 9 (top) Track of Typhoon Bising, and (bottom) simulated wave height distribution

The foreshore basin structure is designed as a shore-attached breakwater with a porous core and armor layer. Since seawater must be retained during no-penetration periods, the pool side of the structure must be watertight. The crest elevation is based on tidal penetration requirements discussed above. To reduce wave reflection and improve damping, the front face of the structure is rendered with a sloping layer of armor blocks of suitable weight (Figure9).

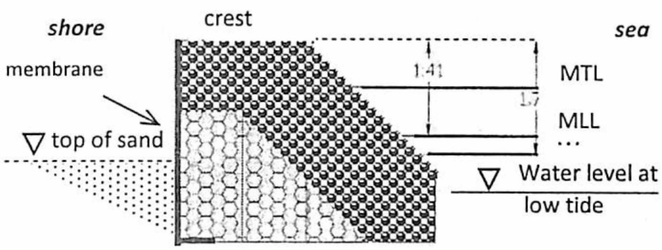


Fig. 10 Cross-section design of foreshore basin breakwater

8. CONCLUSIONS

The hydraulic design of the foreshore basin is initiated by carrying out an analysis of tide penetration into the basin that results from a specified basin crest elevation. This elevation is a project-specific design input that considers beach use and business development objectives of the resort. The optimal crest elevation of the basin ideally meets the desired frequency of tide penetration into the basin intended to achieve the required seawater replacement frequency.

The methodology incorporates a wave modeling procedure to determine the velocity field needed to compute the required beach sediment grain size to promote the deposition of sediments within the basin. A nonlinear wave model is proposed to account for the generally shallow depth in the foreshore area and within the basin.

By properly carrying out the hydraulic analyses, it is possible to design a foreshore basin that protects the beach from damaging storm-induced waves, creates adequate-depth swimming area, promotes formation of a mild beachface, while allowing the ambient tides to replenish the waters in the basin.

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