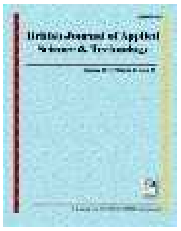

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Evaluation Studies for Shore Protection Design

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Authors' contributions

This work was carried out in collaboration between all authors. All authors read and approved the final manuscript.

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ABSTRACT

A study has been conducted to develop safe swimming conditions along the North-West coast of Egypt. Investigations have been made for the perched beach approaches. Both numerical models and actual scale field data have been employed in this study. Attention has also been given to the empirical models available in literature for computing the transmitted wave energies and lessons learned from the application of protection structures in Egypt and overseas.

An actual scale model of a perched beach is designed and constructed to provide safe swimming conditions using submerged breakwater and two jetties. The project area is constructed along the west boundary of the Alexandria governorate of Egypt. The latter area has long suffered from rip currents as large as 0.7m/sec and a limited safe

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swimming strip of less than 40m during the prevailing wave conditions in the summer season. Several design alternatives have been studied using a numerical model, namely known as Surface water Modeling System (SMS), adopting the actual wave rose of El-Dekhila port and bathymetric survey of the project area. Field measurements were carried out in May 2009 before construction of the perched beach, considered as the baseline condition. The model has been calibrated and validated against the collected and measured field data. Based on all the collected data and information, the numerical model has been applied to investigate different alternatives of the proposed structure including submergence ratio, crest level, the effects of the gap size (if used) and permeability of the structure on the wave height, radiation stresses, current velocities, sediment transport rates and shoreline changes.

It has been found that using the guidelines prepared by the Shore Protection Authority (2002) a safe swimming zone, acceptable flushing condition and minimum flooding of the shoreline due to wave damping effect of the perched beach, especially in case of Low Crested Structures (LCS), could be provided. However, some negative impacts have been noticed both numerically and in field, including some shoreline erosion in the down drift zone and possible trapping of floating debris. The latter impacts can be mitigated by the use of some openings to allow the flow of water and sediments from the up drift to the down drift side to minimize possible erosion in the down drift. It has also been found that the maximum length of the impacted beaches is about 3 times the length of the jetties on both sides. The latter length could be lessened by the aid of lower jetties and/or openings in the jetties. The residual possible impact could be alleviated through a routine sand nourishment program of the shoreline at a frequency that can be decided on a case-by-case basis.

Keywords: Perched beach; semi-open groin; long shore sediment transport; low crested structures; numerical models; safe swimming; environmental impacts and shoreline changes.

1. INTRODUCTION

The coastal landscape west of Alexandria is one of the most attractive recreational sites in Egypt. The coastline is characterized by wide beaches consisting of white Oolitic carbonate sand. The shore is generally linear with few protective configurations. However, the beach is not suitable for swimming because of the steep beach profile ranging from 1:30 to 1:50 [1]. Strong offshore direct rip currents regularly lead to hazardous situations, and at some locations swimming is prohibited for a considerable time of the year especially during summer storm. The swimming may be prohibited at locations where the wave height is too large or the rip shore currents are too strong. So, these conditions may have adverse effects on recreational facilities along the coast. [2] concluded that breaker heights smaller than 0.6m and current velocities smaller than 0.2m/s are considered as comfortable swimming conditions, but it is hard to swim against a rip current of 0.5m/s and breaker height greater than 2.0m even for good swimmers.

The current study concentrates on the use of perched beach. This scheme was tested by Delft Hydraulics, Netherlands as an ideal scheme in the Al-Arab Bay zone in 2003 to create safe swimming conditions. This scheme consists of a submerged breakwater enclosing a sheltered basin. The top level of the offshore part of the submerged breakwater is close to the sea water surface level to ensure water flow from open sea to the basin and to prevent obstruction of the sea view, i.e. minimal visual impact. Basins enclosed by submerged breakwaters exist naturally, e.g. a rocky submerged shoal enclosing a shelter basin, such as at Stanley beach in Alexandria (named "Al Bahr al Sagir") and artificially or semi-artificially, such as at Sela beach in Bat-Yam, as illustrated in (Fig. 1).

Fig. 1. Sela beach development scheme at Bat-Yam, constructed since 1969

The enclosed basin at Sela beach is 400 meters alongshore by 175 meters perpendicular to the shoreline. The breakwater followed the lines of existing submerged beach rock to minimize construction costs; the submerged breakwater was partially constructed on a sandy sea bed. Outside the enclosed basin, accretion fillets take place on both sides. The accretion fillets undergo seasonal changes due to the offshore/onshore sand movement. The littoral drift bypass around the breakwater results in a negligible impact on the neighboring shoreline evolution due to breakwater construction. Periodic water quality tests had been performed since the construction of the Sela beach scheme in 1969, showing that water quality inside the enclosed basin is similar to the open sea due to the outflow of water mainly through the rubble mound breakwater as the still water level is higher inside than outside the basin. This was due to the inflow of water by currents generated by waves breaking on top of the submerged breakwater. Wave height inside the enclosed basin were significantly reduced compared to the open sea. Wave heights are about 0.5 meters inside against 1.5 meters outside the basin. During winter storms, wave heights in the basin may reach up to 1.0 to 1.5 meters and currents of significant velocities were reported by good swimmers [3].

During most days of the summer months, the eastern coast of the Mediterranean Sea is exposed to onshore wind resulting in waves having heights from 0.6 to 1.2 meters and sometimes up to 2 meters, as indicated by [3], based on long term observations. Prior to the construction of the Sela beach breakwater, the nearshore bar was sounded and found to have a crest elevation of about 1.0m below the sea water surface level quasi-parallel to the shoreline, as illustrated in (Fig. 2). Wave breaking occurs on the sea side or on the top (crest) of the sand bar and almost the entire width of the foreshore is covered by surf. The area between the coastline and the inner (onshore) side of the bar acts as a velocity field in which flow takes place in all directions (Fig. 2). Along the near shore bar there are breaks (channels) or rip passes through which rip currents are forced out to the sea. On top of the bar the water seemed to be air entrained, absorbing currents from both sides. The bar configuration, rip passes, currents velocities and directions are changing under different waves conditions. This flow field, the rhythmic bathymetry and the difficulty of returning to the shore due to rip currents are the reasons of drowning accidents, conditions very similar to the north western Egyptian Mediterranean coast.

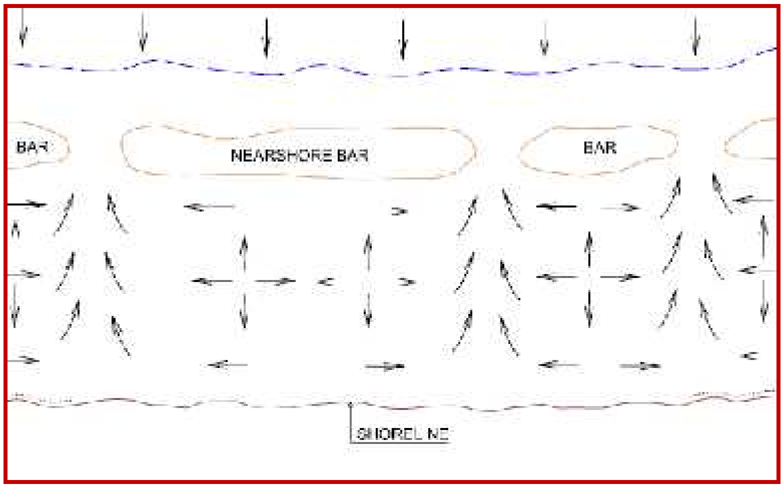

Fig. 2. Schematic diagram of nearshore bar and nearshore current

Due to the increasing demand for safe swimming conditions with minimum impact on the shoreline and keeping acceptable water quality, new studies have been conducted to meet these requirements using appropriate coastal structures. [4] conducted a study in 2002 for the development of the North-West coast of Egypt and recommended the use of a perched beach as a possible alternative for safe swimming conditions along the Al-Arab bay zone, located from Al-Agamy at Km 21 to Al-Alamin city at Km 120 (along the Alexandria-Matrouh road). However, none of the perched beach designs have been constructed to date. Thus, the actual performance of perched beach is not well-known yet. Recently in 2011, one design was approved by the Egyptian Environmental Affairs Agency (EEAA) at Km 38 along the Alexandria-Matrouh road. The construction of this perched beach started in 2012 and was completed by 2013. Thus, the actual field effect of the perched beach can be monitored and compared with the results predicted by numerical models.

2. STUDY AREA

The pilot area is the coast of a new tourist resort located at Sidi Krir, kilometers 39.774/40.078 west of Alexandria along the Egyptian Northwestern Mediterranean coast, as illustrated in (Fig. 3). In this study, a location has been selected where the shoreline makes an angle of 40º with the North direction. Thus, the predominant waves are almost perpendicular to the shoreline and consequently the slope of the beach is steep, making it a dangerous location for swimmers. Due to the steep slope in the upper part of the beach profile in the study area (water depth <8.0m below Mean Sea Level (MSL)), waves break close to the shoreline where people tend to swim. The breaking waves make it difficult for swimmers to move freely and may even cause them to remain in place due to seaward drifts caused by the undertow and the rip currents. As a result of the adverse effect of the steep beach profile, swimmers reach relatively large water depths within a short distance from the shoreline. If they reach 30-50m from the shore line, it is too deep to stand on the sea bed. This, in combination with the breaking waves, allows for a very narrow usable strip for most swimmers along the beach.

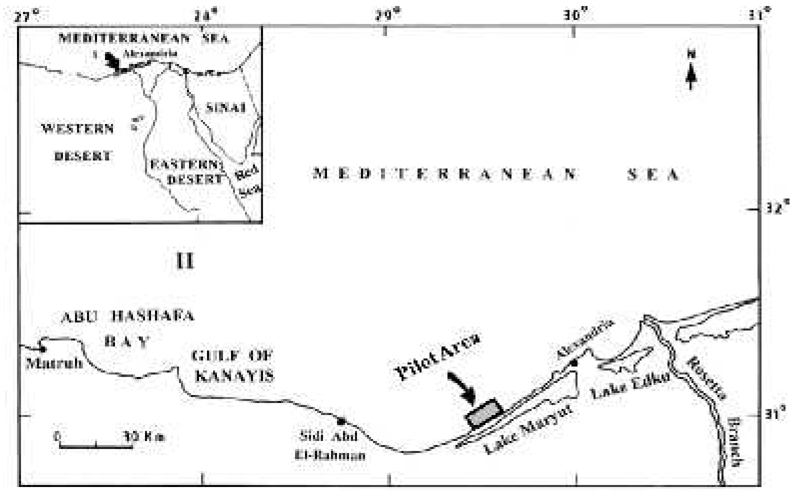

Fig. 3. Location map of the pilot area

The layout and dimensions of the proposed perched beach is shown in (Fig. 4). The resort beach is 300m long, but the protected length is 200m, leaving 50m on both sides of the groins open to the sea. However, simulations using the SMS model have been made for 3000m along the shore line, extending for about 1500m normal to the shoreline to limit the boundary effects on the study area. The perched beach generally has two bounding groins and shore parallel breakwater at water depth equal to 3.5m. The groins extend for 120m normal to the shoreline and have some opening in its body, such as either a pipe or a gap.

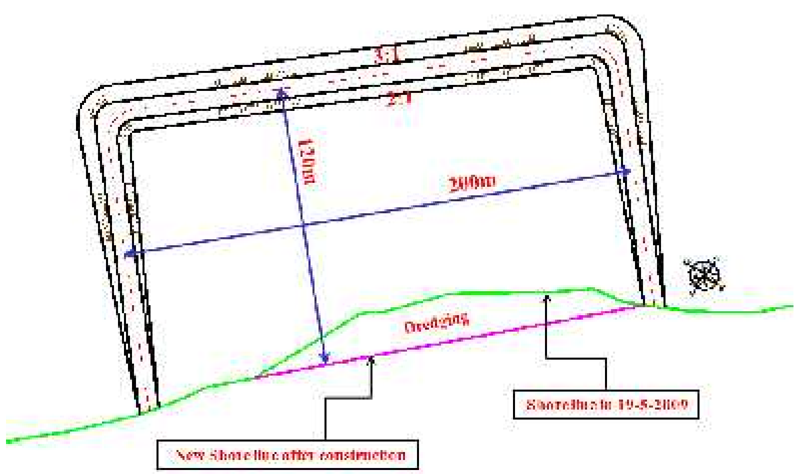

Fig. 4. The layout of the proposed perched beach

Simulations have been made for the case of the perched beach before construction in order to evaluate the problems encountered in the beach. This case was called the baseline condition and shall be used as a reference condition. Based on the results of the wave rose in the El-Dekhiela port (Fig. 5), it has been found that three wave conditions are of major importance, i.e., two during the summer season and one in winter, as shown in (Table 1). The dominant wave height all over the year in deep water is 1.77m and it occurs for (20-30) days during the summer season. However, in the summer period the most relevant for beach recreation and swimming condition, wave heights reach a significant wave height (Hs) of 2.5m for (4-5) days.

Table 1. The dominate wave conditions

Wave condition	Hs (m)	T (s)	Duration (days)	Notes
1	1.77	5.0	20-30	In summer
2	2.50	7.5	4-5	In summer
3	4.50	10.0	0.86	In winter

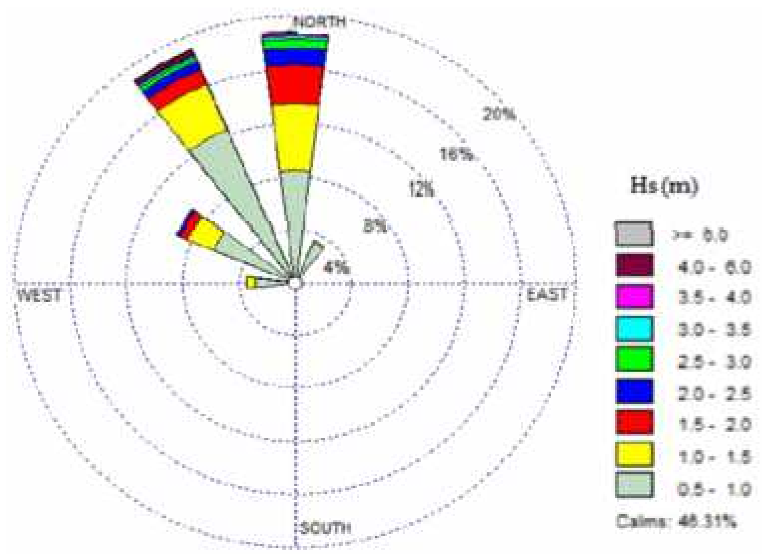

Fig. 5. The wave rose at a water depth of 13m in El-Dekhiela Port (2001-2005)

Various configurations of the perched beach have been described and simulated and the results analyzed to provide general guidelines for the design of a perched beach within the Al-Arab Bay zone denoted as Cell 5 of the North West coast of Egypt. The proposed alternatives have various configurations, including: submergence ratio of the breakwater, groin with/without gap and emerged/submerged groin. The alternatives have been compared from the point of view of wave height, currents velocities, and flushing and shoreline changes. It should be noted that the constructed perched beach is denoted as Alternative (04) in the current study.

2.1 The Proposed Alternatives

The proposed alternatives have various configurations, as follows:

Alternative (01): Submerged groins and breakwaters:

This alternative proposes a shore parallel breakwater and two groins having their crest at -0.50m MSL. (Fig. 6) shows the layout of the structures and the bed levels as presented by the SMS model. This alternative has all the merits of submerged structures, e.g., an aesthetic view, moderate waves, good flushing, etc.

Alternative (02): Submerged breakwater and surface piercing groins:

This alternative is similar to Alternative (01) except that the groins are constructed higher than the water surface at +2.0m MSL (Fig. 7). This alternative enables the possible economic use of the groins for recreational activities, but it affects the sight distance of beach visitors to some extent due to interruption of the sight line at the boundaries.

Alternative (03): Submerged breakwater and surface piercing groins with a gap:

This alternative is similar to Alternative (02), but the west groin has a gap located at 1/4 of the groin length measured from the offshore head and the east groin has another gap at 1/4 of the groin length measured from the shoreline (Fig. 8). The proposed gaps are sought to help flush the protected area and allow less trapping of long shore sediments.

Alternative (04): Submerged breakwater and groins with a gap:

This alternative is similar to Alternative (03), but the crest level of the two groins is at -0.50m MSL. (Fig. 9) shows the layout of this alternative and the topography of the sea bed generated by the SMS model.

Alternative (05): Very low crested breakwater and groins:

This alternative is similar to Alternative (01), but the crest level of the two groins and the submerged breakwater is at -1.25m MSL (Fig. 10).

Alternative (06): Medium low crested breakwater and groins:

This alternative is similar to Alternative (01), but the crest level of the two groins and the submerged breakwater is at -0.9m MSL (Fig. 11).

The baseline and each alternative have been tested with three wave conditions: 1.77m, 2.5m and 4.5m approaching from the North West direction and represent moderate to severe wave conditions. The presented results are mainly focused on the flow field inside the perched beach and the adjacent area along the shoreline. Moreover, the shoreline changes (erosion/accretion) have been investigated.

2.2 Field Measurements

The survey is performed from onshore the high water line to 10 meters and more offshore relative to Admiralty Chart Datum (A.C.D.) The survey was performed using echo-sounder and differential GPS. Total station stands with reflectors are used for the beach face and backshore. The water depths are corrected to account for sea water level fluctuations to be relative to A.C.D.

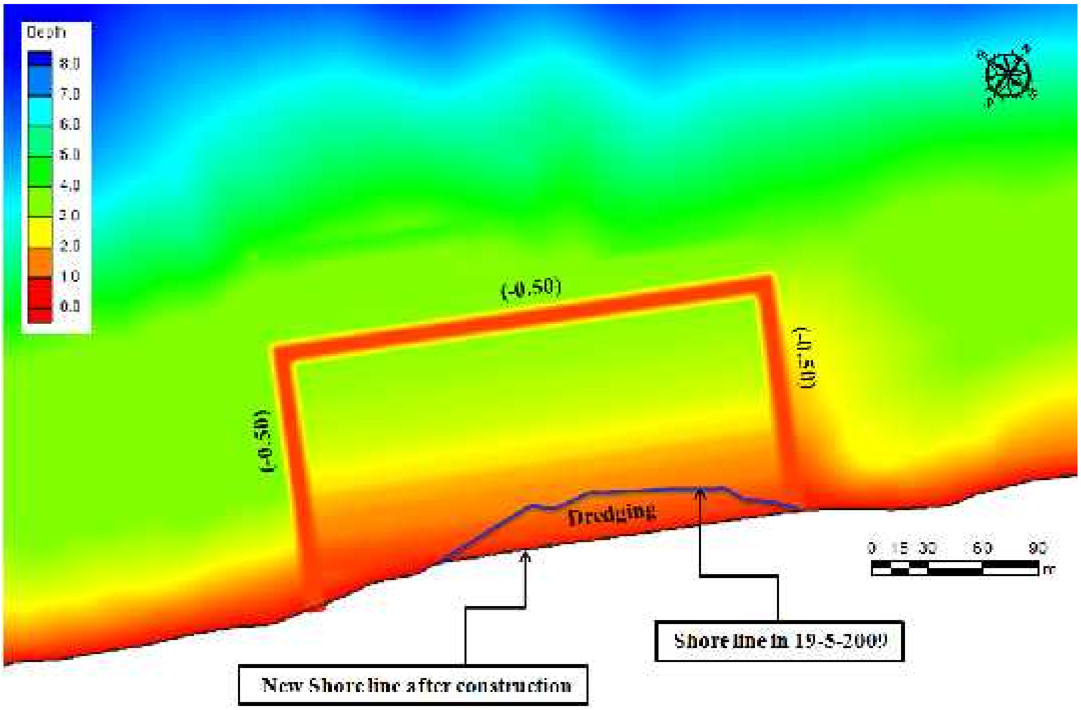

Fig. 6. Layout of perched beach and sea bed levels for alternative (01)

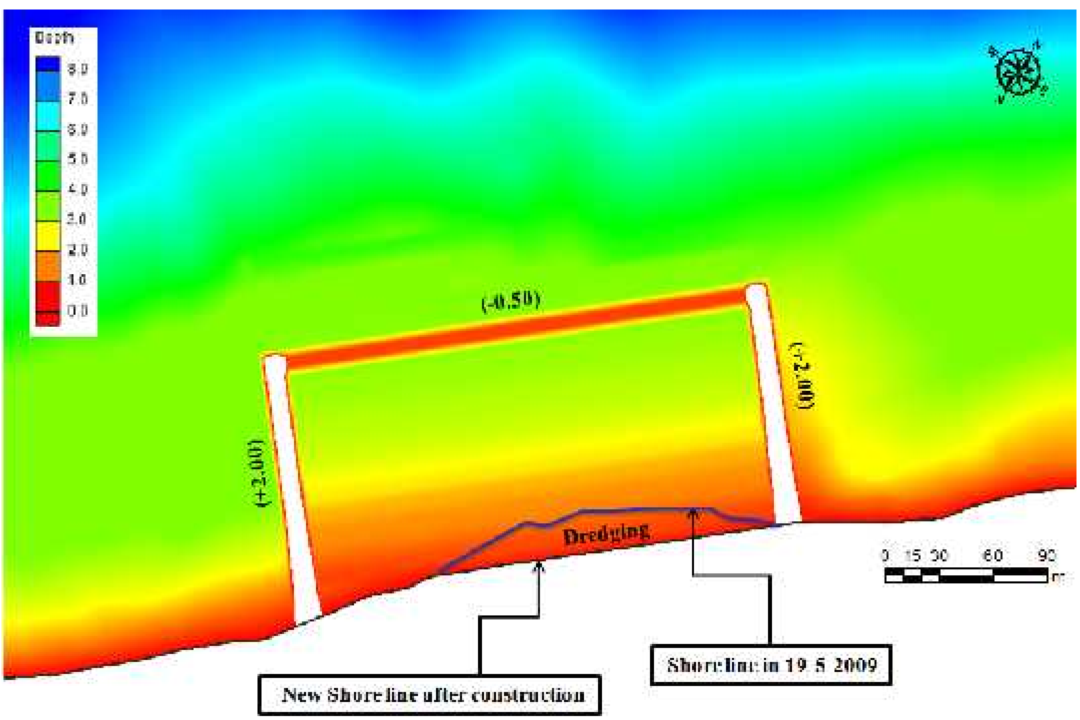

Fig. 7. Layout of structures and sea bed levels for alternative (02)

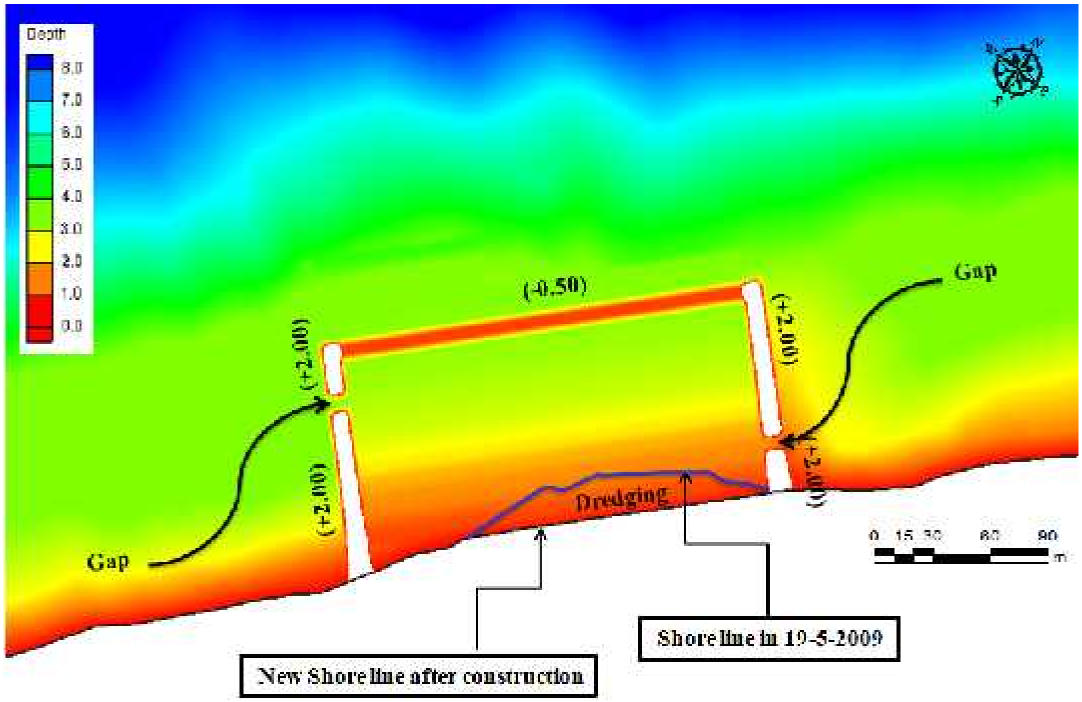

Fig. 8. Layout of structures and sea bed levels for alternative (03)

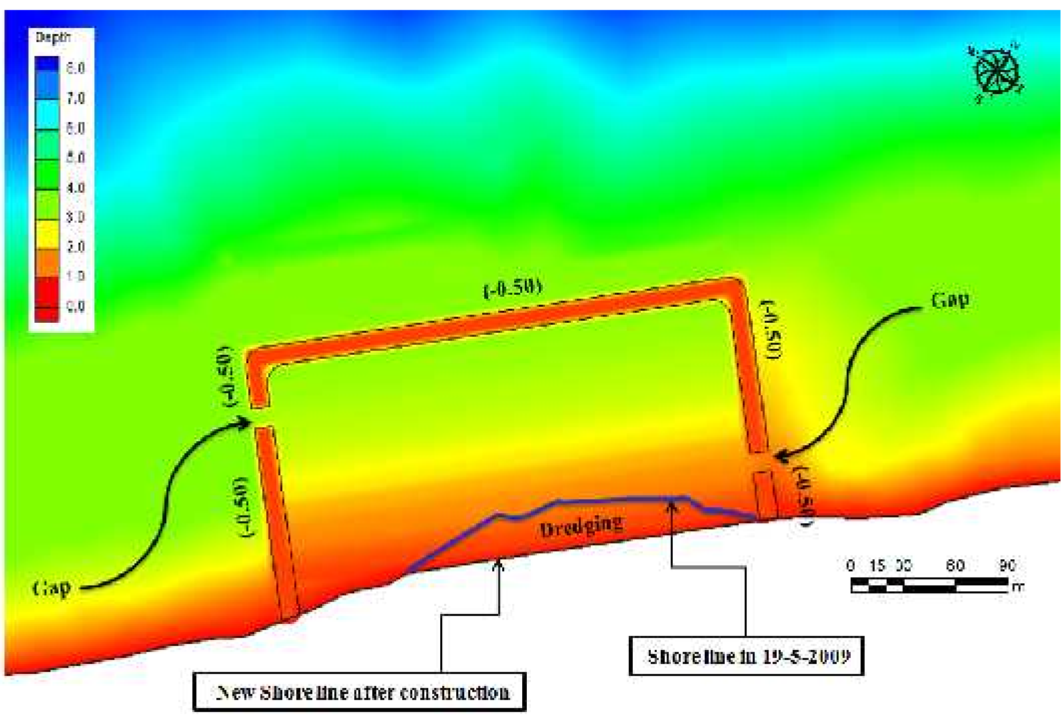

Fig. 9. Layout of structures and sea bed levels for alternative (04)

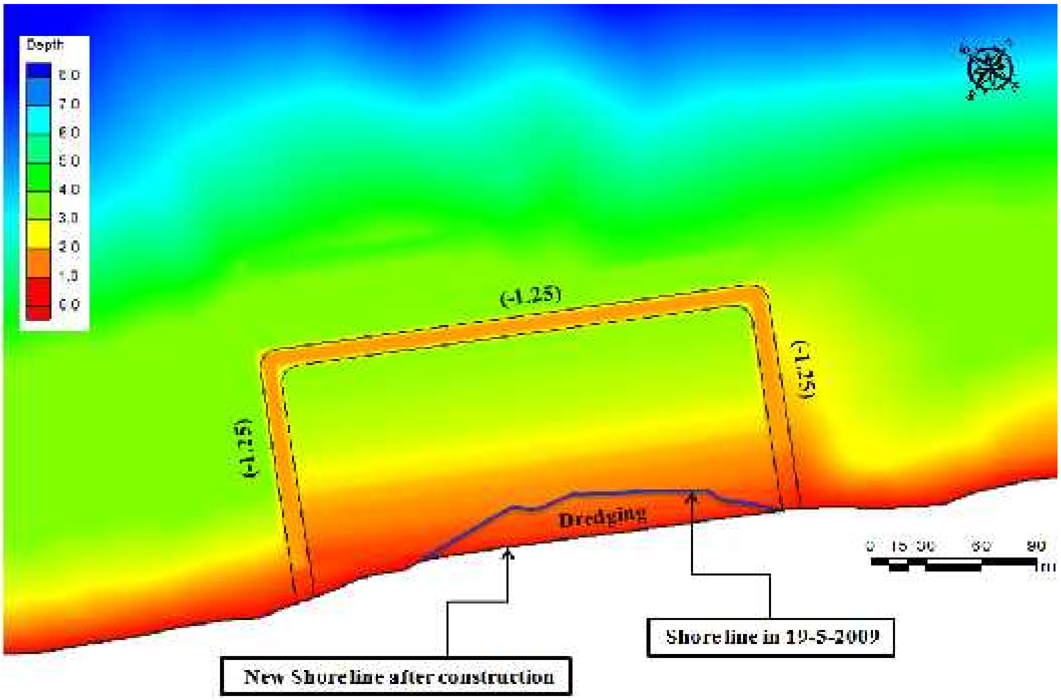

Fig. 10. Layout of structures and sea bed levels for alternative (05)

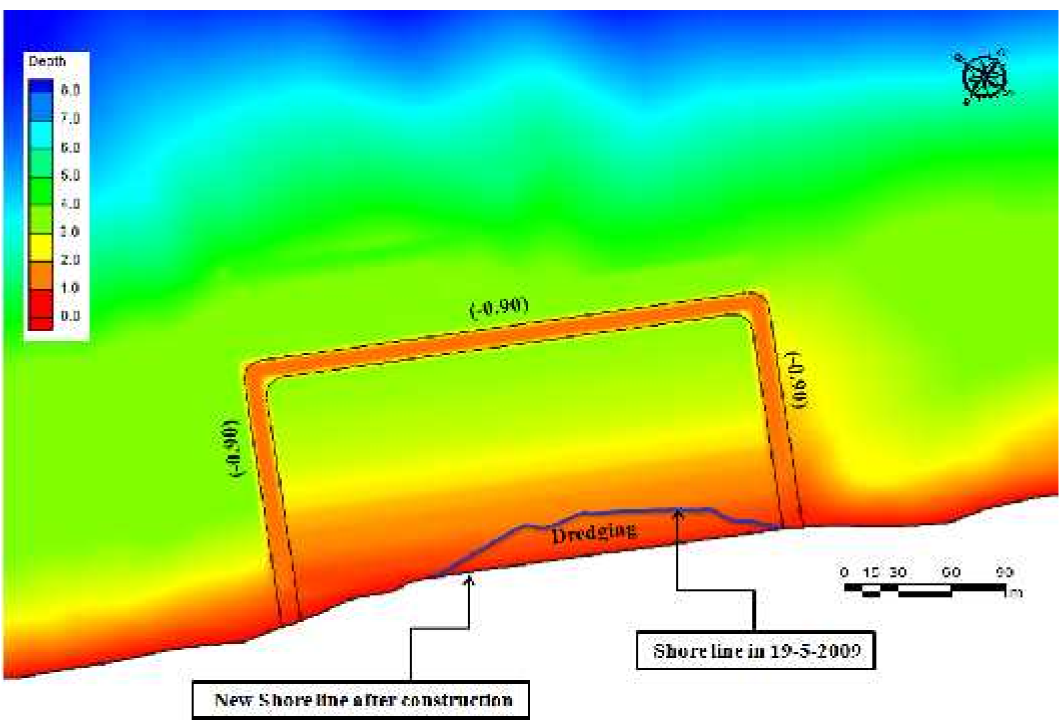

Fig. 11. Layout of structures and sea bed levels for alternative (06)

Sediment samples at the study area region were collected at different depths/locations along the coast. Sea bed soil samples are collected from the backshore and the beach face up to 6 meters water depth. Laboratory grain sizes distribution tests performed for the samples illustrate that the backshore and the onshore part of the beach face consist of well graded medium sand. Samples collected in water depths close to 2 meters are medium to coarse sand which could be attributed to the washing of fine sand during collection of samples by divers. From analysis of the sediment sample in the study area, it is cleared that the sand on the beach and in the upper part of the coastal profile is coarse, in the order of D50= (0.3-0.7) mm, while in the deeper part (below MSL-2m) the median grain size somewhat smaller, in order of D50=(0.2-0.3)mm.

The net long shore sediment transport in pilot area is very small to negligible. On the basis of computations and observations near the south-western boundary of the pilot area, the net long shore transport is estimated to be slightly north-east directed with a magnitude of 15000m^3/year approximately. Near the north-eastern boundary of the pilot area the net long shore transport is estimated to be negligible.

Another set of data regarding the waves were collected from the available sources such as the Coastal Research Institute (CoRI) and the Hydraulics Research Institute (HRI).

2.3 SMS Model

SMS is a finite difference model originally developed by Brigham Young University 1985 in cooperation with the U.S. Army Corps of Engineers, Engineer Research and Development Center (ERDC), and the U.S. Federal Highway Administration (FHWA). Wave models in SMS include CMS-Wave, and BOUSS-2D and include both spectral and wave transformational models. The SMS model adopts GENESIS based on the one-line theory for sediment transport calculations [5]. Numerical wave models BOUSS-2D and CMS-Wave may be used together (coupled) to evaluate potential alternatives of coastal planning for various coastal conditions.

CMS-Wave [6,7],8] is part of the Coastal Modeling System (CMS) for simulating combined waves, currents, sediment transport, and morphology change at coastal inlets, estuaries, and river mouths [9]. The CMS-Wave is a spectral wave model belonging to the phase-averaged class. It is commonly based on Energy Balance equation. It performs steady-state spectral transformation of directional random waves co-existing with ambient currents in the coastal zone. The model simulates half-plane and full-plane wave propagation, so that wave generation, wave reflection and bottom frictional dissipation of multi-directional waves can be considered. BOUSS-2D is a two dimensional (2-D) phase-resolving wave model that employs a time-domain solution of fully nonlinear Boussinesq-type equations for waves propagating in water of variable depth. The theoretical background and user manuals for BOUSS-2D are available in CMS technical reports and CHETNs [10,11].

Based on the bathymetric survey, rectilinear grids have been generated and the associated depth files have been prepared for the wave models (CMS-WAVE & BOUSS-2D). The model extends 3000m along the shoreline and 1500m offshore. A high grid resolution has been applied in the area of the proposed structure (perched beach) in front of the tourist resort, and a low resolution has been implemented far away of the area of their influence. The grid cell size (5x5m) is fine inside the structure area and the adjacent regions while it is course (20x20m) in the far field till the open boundary (240 cells along the shoreline and 150 cells offshore). The model grid meets criteria for smoothness (adjacent cells do not differ much in size) and orthogonally (the angles of the comers of the cells are close to 90º), in order to avoid that small disturbances due to irregularities in the grid grow to governing features during the computation. The average effective grain size of the soil on-site is found to be 0.3mm, the average berm height is computed and found to be 2m and the closure depth is considered to be 8m. Based on the measured shorelines of 2007, 2009 and 2012 the model was calibrated. A comparison was prepared between the computed and measured shoreline changes for different cases of different calibration coefficients (K1, K2) in the longshore sediment transport formula. It has been found that the best values of the calibration coefficients for the study site are K1=0.45 and K2=0.10.

2.4 Base Line Condition

(Fig. 12) shows the wave direction and height as waves approach the shoreline for moderate and severe wave conditions. It can be observed that waves approach the shore normal to it due to refraction. The variation of wave heights along the centerline of the project area for the case before constructing the perched beach (Shoreline is located at distance=0) is presented in (Fig. 13). It can be observed that the waves break at distances of 160m, 120m and 40m measured from the shoreline for deep water wave heights of 4.5m, 2.5m and 1.77m, respectively. Also, it can be observed that only a narrow strip of less than 30m, having a wave height of less than 0.60m for H=1.77m, exists. Thus, the beach is unsuitable for swimming even during the occurrence of moderate wave heights. Also, it can be observed that for a wave height of 4.50m, the wave height decreases rapidly when compared with incident waves of 1.77m and 2.50m.

The location of the breaker line for the dominant high waves during the summer season, i.e., H=1.77m, is very close to the shoreline hence limiting the surf zone against the convenience of swimmers. It can be observed that there are two breaker lines, i.e., in deep and shallow water. Moreover, a strong rip current can be noticed on a rip head, which is formed offshore. Although the breaker line is shifted offshore and the surf zone becomes wider than the case of H=1.77m, considerably large waves exist near the shoreline and a secondary breaker line is formed. It can be concluded that swimmers should abandon the sea for a long period during the summer season due to the occurrence of H=1.77 and 2.5m waves.

(Fig. 14) shows the flow field in the pilot area. Five transects along the shoreline of the resort are shown, namely as C.S.1~C.S.5, and C.S.3 is at the resort centerline. Furthermore, the cross shore and the long shore currents are also computed as shown in (Figs. 15-a and 15-b). It is clear that the transect C.S.3 shows the largest long shore velocity and has a maximum of 0.68m/s for H=2.5m. The rip current reaches a maximum value of about 0.75m/s occurs at C.S.4 for H=2.5m during the summer season. It can be confirmed that the rip currents are generally large and dangerous for more than a month every year during the summer season (see Table 1). It can be concluded that the presence of the hazardous rip currents, circulation zones and the adverse effect of the steep profile along the coast of the pilot area, in combination with the breaking waves, makes that only a very narrow strip (20m approx.) of coastline usable by most swimmers. So, there is an essential need to construct a suitable coastal structure in the pilot area in order to create safe conditions for swimmers with minimum shoreline change.

3. IMPACTS OF THE PERCHED BEACH

Numerical simulations have been made for the case of the actual dimensions of the structures and site specific conditions [Alternative (04), (Fig. 9)]. It can be observed that two openings exist in the west and east groins, and dredging works have been applied to provide better shape of the beach and nearly uniform bed slope within the perched beach.

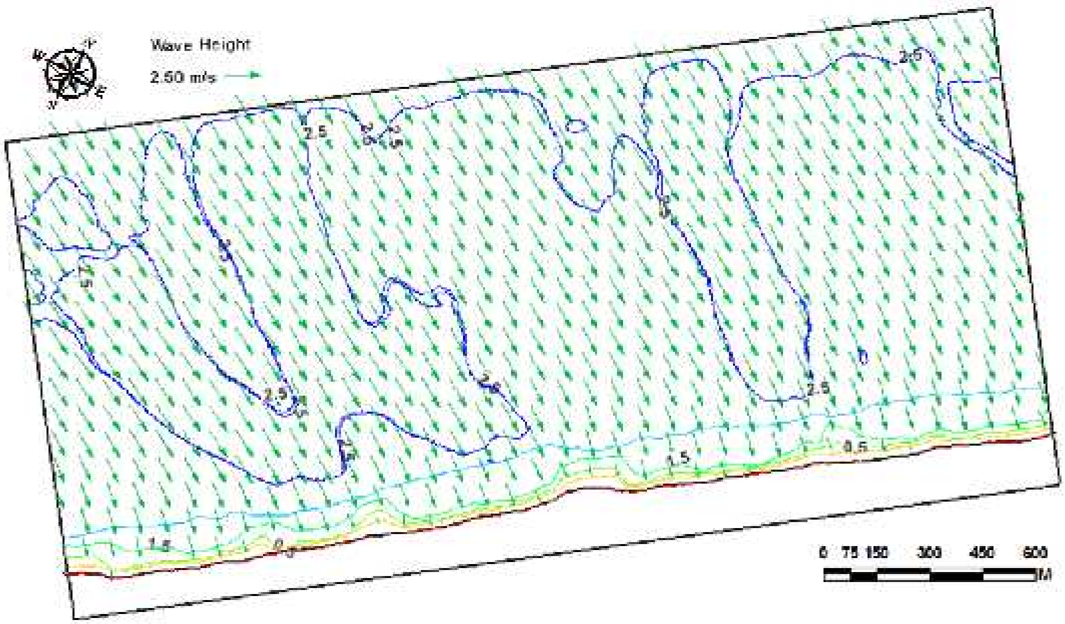

Fig. 12. Computed wave height and direction in the study area (H=2.5m, T=7.5sec and direction is NW) for the baseline case

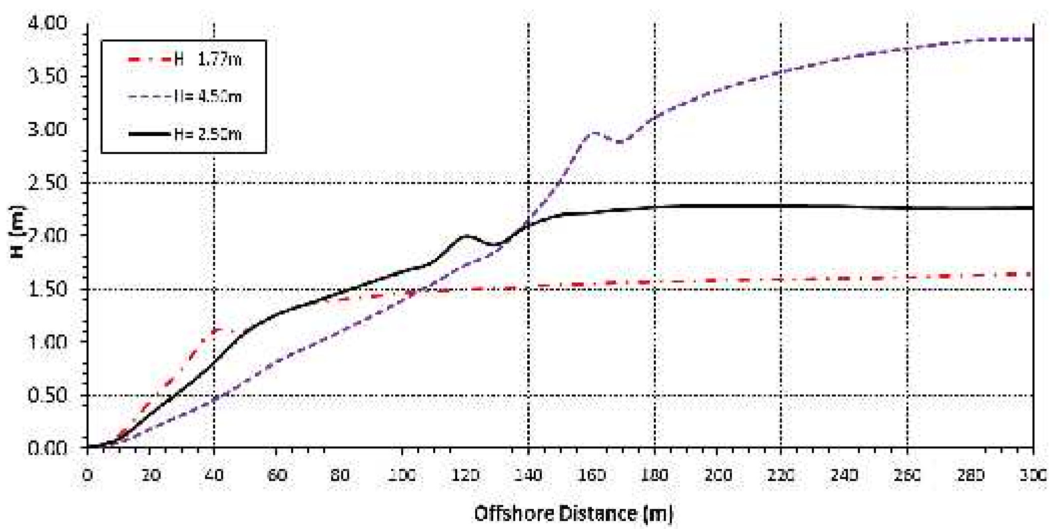

Fig. 13. Computed wave height along the centerline of the beach

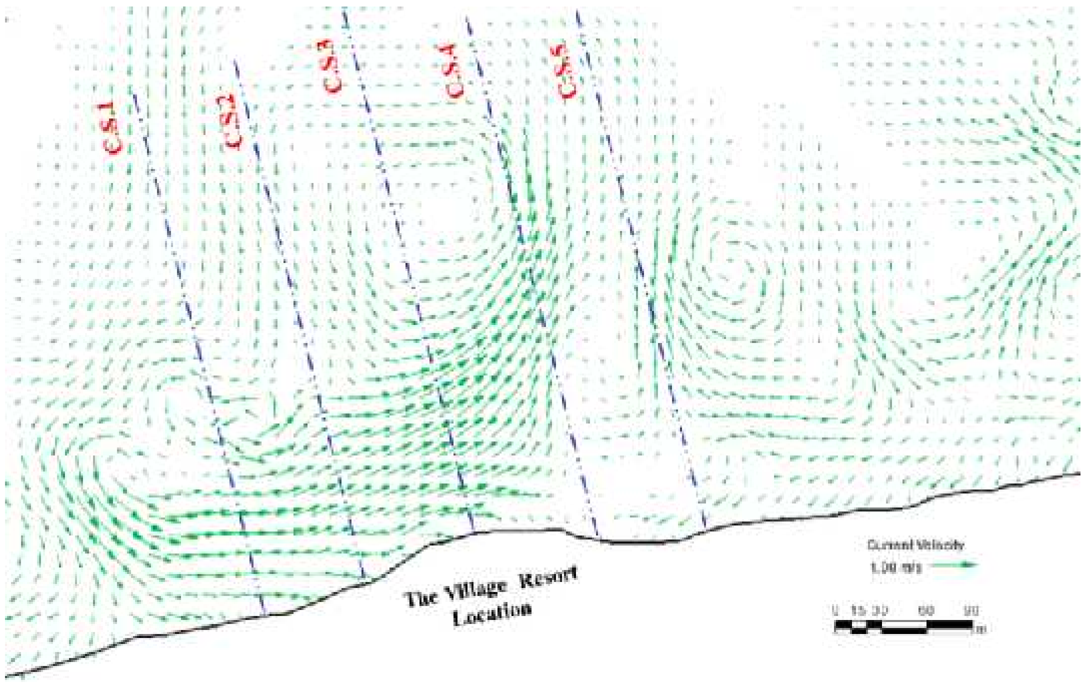

Fig. 14. Current velocity in the beach for H=2.5m, T=7.5s and direction is NW

(Figs. 16a and 16b) show the wave direction and height as the waves approach the perched beach from the prevailing wind direction for H=1.77m and 2.5m. It can be observed that the incident wave is partially reflected at the submerged breakwater and small waves are transmitted to the protected area over and through the permeable submerged structures. Over the submerged breakwater, the combined effects of wave shoaling and damping occur due to the change in the water depth and the dynamic interaction with the flow inside the submerged breakwater. The gap allows some wave energy into the perched beach and high waves are formed at its offshore face. Thus, it is recommended to prohibit swimmers from accessing the opening, and a strong plastic wire mesh should be installed in the openings to save swimmers from any possible movement through them.

It is also evident that wave breaking takes place over the submerged breakwater, and small waves are transmitted to the protected area (Fig. 17). It can be remarked that the transmitted wave height is always less than 0.75m even for the extreme wave height of 4.5m, but it is less than 0.5m for all other incident wave height conditions. Comparisons between the wave heights for the cases before and after construction of the perched beach are evident in (Fig. 17). It is worthwhile that before the construction of the perched beach, large waves could approach the shoreline at a height of more than 1.0m for the prevailing deep water summer waves of 1.77m in height. The latter waves break at about 40m from the existing shoreline and the broken wave height is less than 0.5m within the last 20m from the shoreline. This causes inconvenience to swimmers due to the limited width of the surf zone and possibly large induced rip currents.

The virtue of the perched beach is well pronounced as shown in (Fig. 18) as the current velocities in the protected area become very small due to the construction of the perched beach. The velocity inside the perched beach is less than 0.5m/sec which is very convenient for swimmers even during the extreme incident wave conditions. It can be observed that no rip current or circulation occur inside the perched beach, which is safe for swimmers. On the other hand, rip currents and circulation zones are evident outside the perched beach.

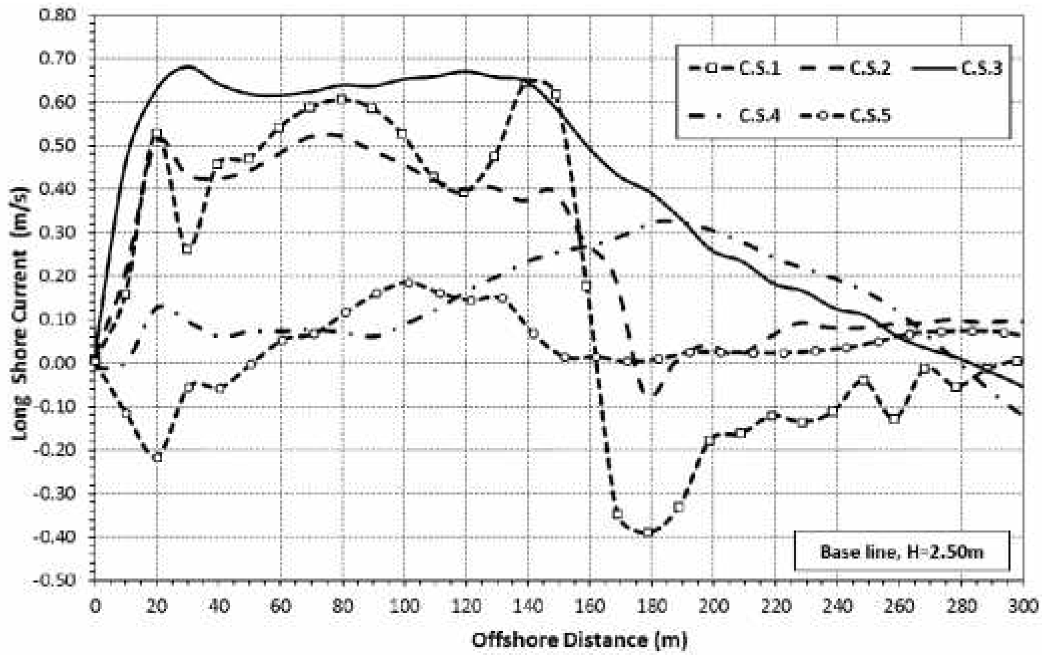

Fig. 15a. Long shore current at the profiles for H=2.50m

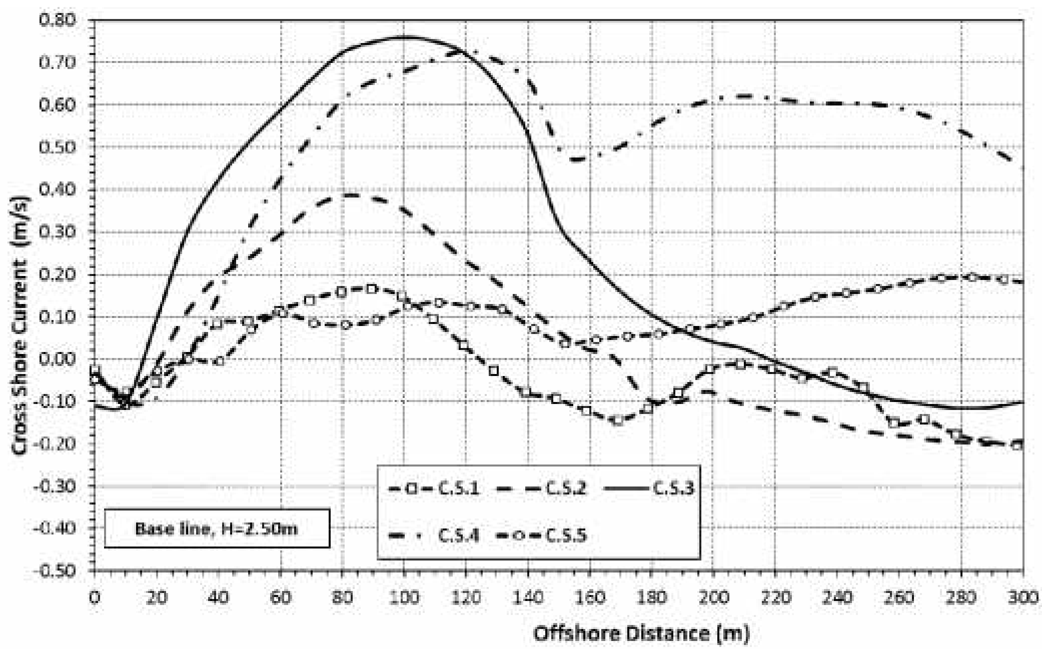

Fig. 15b. Cross shore current at the profiles for H=2.50m

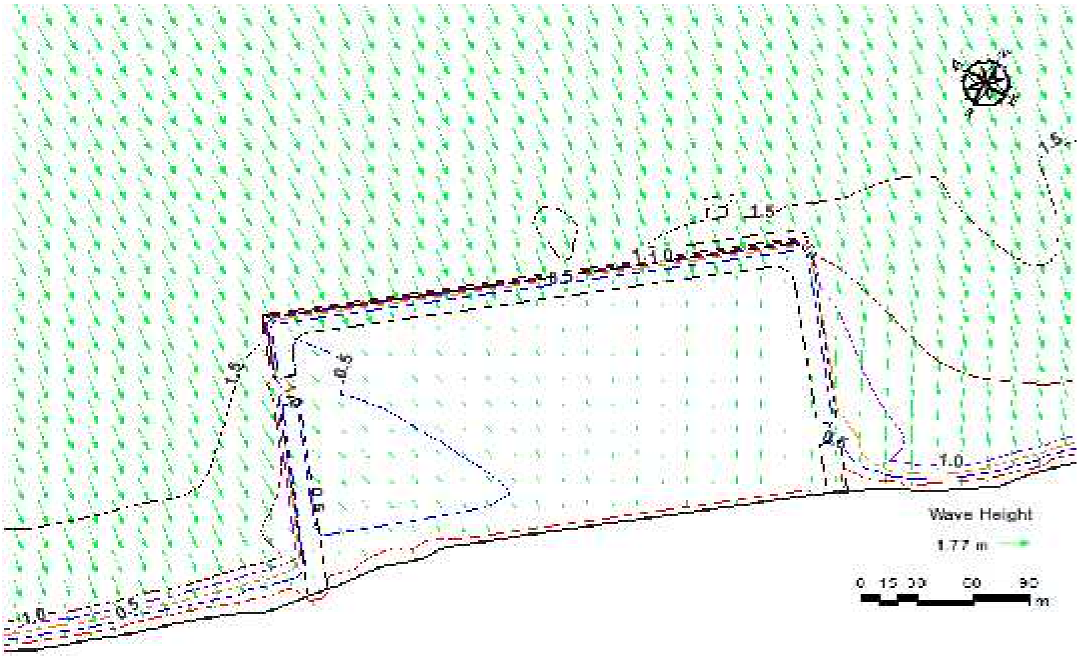

Fig. 16a. Computed wave height and direction in the vicinity of the structure for H=1.77m

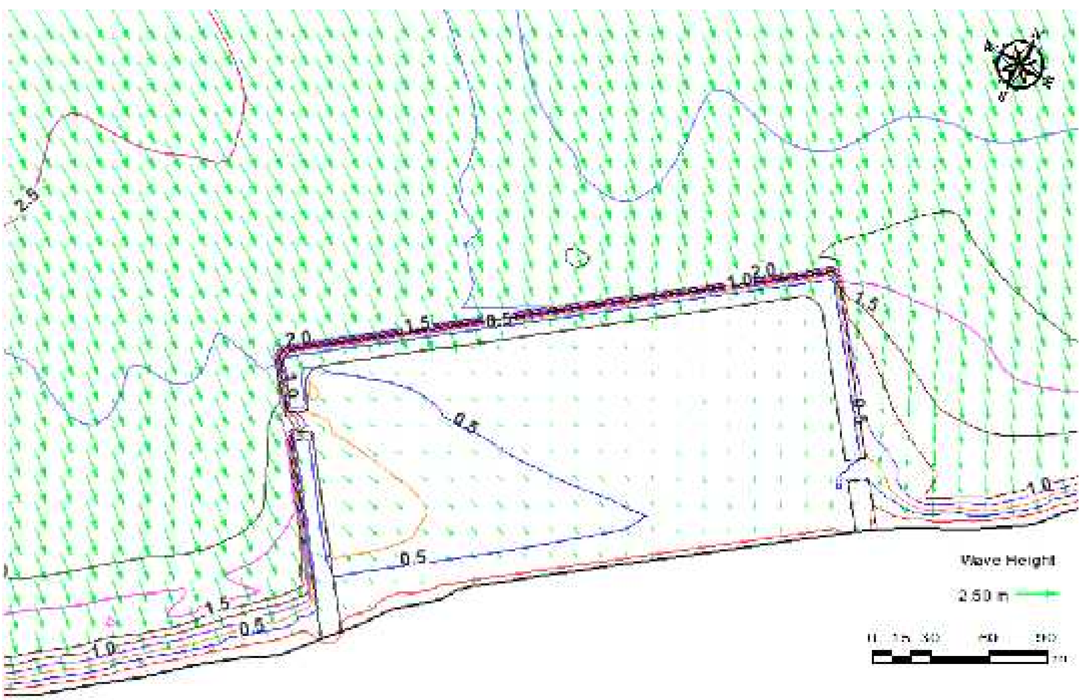

Fig. 16b. Computed wave height and direction in the vicinity of the structure for H=2.50m

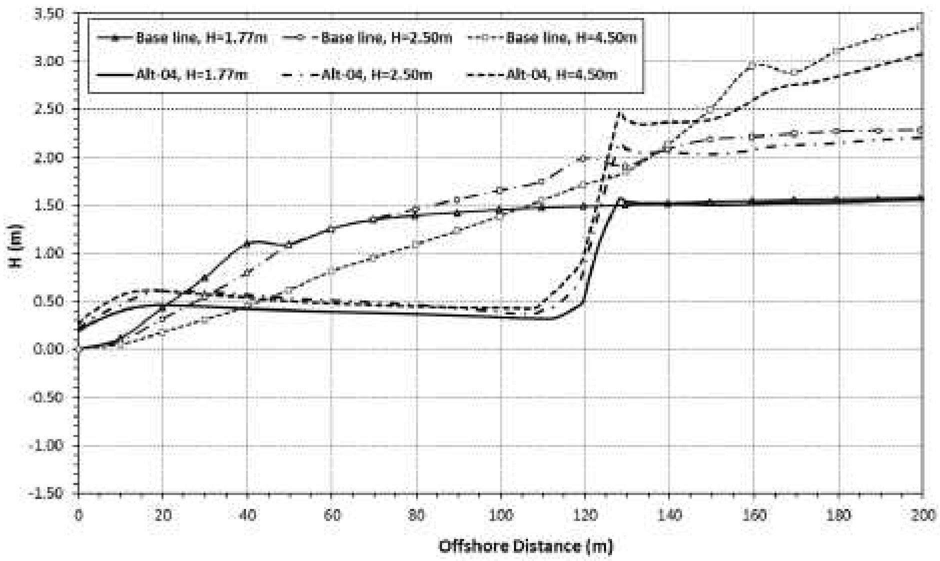

Fig. 17. Computed wave height at the centerline of the perched beach (at the profile C.S.3)

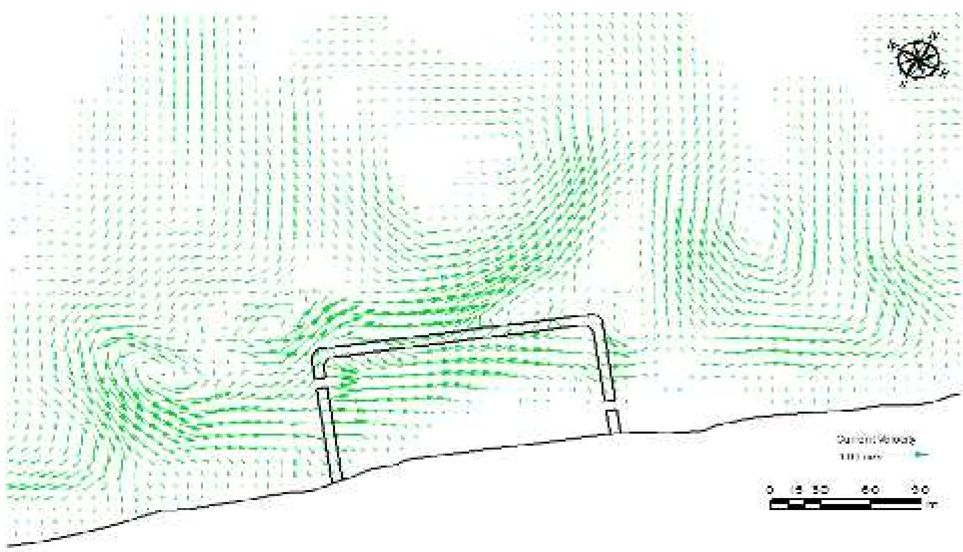

Fig. 18. The current velocity in the vicinity of the structure for H=2.50m

The cross and long shore currents along the centerline of the perched beach for various incident wave conditions are presented in (Figs. 19 and 20). It can be noticed that the rip currents are almost null during moderate summer waves, i.e., H=1.77m, and are less 0.2m/s during high summer waves, i.e., H=2.50m. On the other hand, the long shore current inside the perched beach is less than 0.5m/sec during the summer waves. The maximum long shore current velocity reaches 0.70m/s and 0.60m/s during summer for H=2.50m and H=4.50m, respectively, and occur at the offshore toe of the submerged breakwater outside the perched beach. It is also evident that eddies are sometimes generated at the west corner of the submerged breakwater and large current velocities are induced at the offshore toe of the submerged breakwater (Fig. 19) for the highest summer and winter waves, i.e., H=2.5 and 4.5m. Thus, additional toe protection has been made during construction at the latter locations to save the breakwater from toe scour and possible failure. The currents also cross the opening in the west groin to the perched beach at relatively slow velocity before they reach/pass the openings in the east groin. (Fig. 18) shows that two large eddies are generated during the highest summer wave, i.e., H=2.5m, and are located at the west and east sides of the perched beach. Observations have been made after the complete construction of the perched beach and confirmed the aforementioned results of the numerical model to a great extent.

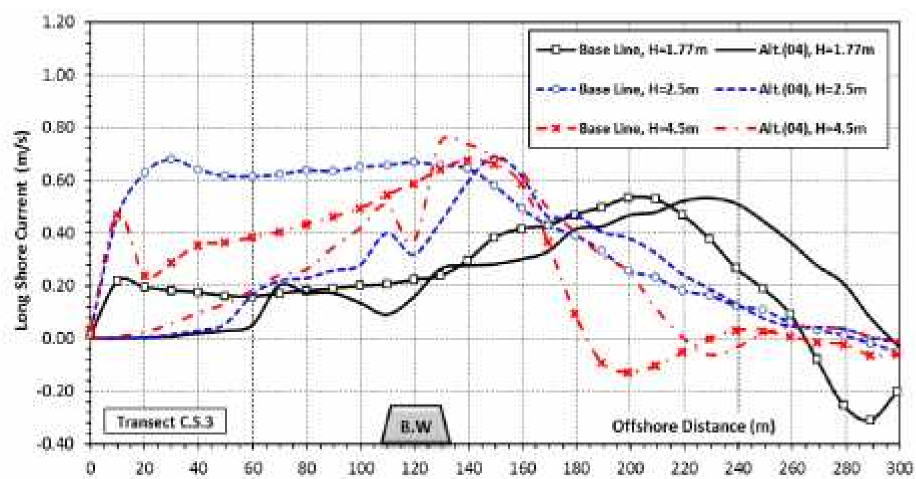

Fig. 19. Long shore current velocity along C.S.3 before/after construction of the perched beach

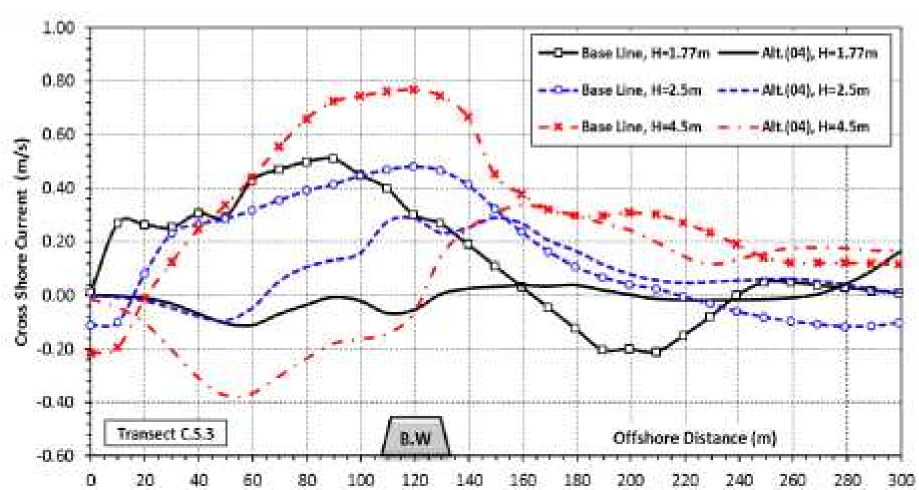

Fig. 20. Rip current velocity along C.S.3 before/after construction of the perched beach

4. ALTERNATIVES DISCUSSION

The proposed alternatives have various configurations including: submergence ratio of the breakwater, groin with/without gap and emerged/submerged groin. The alternatives have been compared from the point of view of wave height, currents velocities, flushing and shoreline changes. In the current discussions, it has been focused on the case of maximum deep water wave in summer, i.e., H=2.5m.

4.1 Wave Height

Comparisons between the wave heights for the cases before and after construction of the perched beach are evident in (Fig. 21). The computations have been made for the different alternatives, as well, for various values of d/h (crest height/water depth), as shown in (Fig. 22). It is observed that wave breaking takes place over the submerged breakwater and small waves are transmitted to the protected area. These figures confirm that a partial standing wave is formed in front of the breakwater and nonlinear wave damping takes place over it with a smaller wave being transmitted to the onshore side. The results also suggest that wave energy dissipation depends greatly on breakwater height. It is shown that the higher the breakwater is, the lower the transmitted, but the slightly higher the reflected wave energies are. It has been found that the transmitted wave height is less than 0.60m for the alternatives: Alt. (01), Alt. (02), Alt. (03), and Alt. (04) which have the same submergence ratio (d/h=0.84) , which means that the site can be used for recreational purposes for most of the summer season. On the other hand, the transmitted wave height is more than 0.6m for the alternatives: Alt. (05), and Alt. (06). (Fig. 23) shows that there is no significant effect on the wave height at the centerline of the perched beach due to the gaps in groin and the emerged/submerged groins. So, these wave conditions are considered as non-comfortable swimming conditions, as stated by [2] and hence these alternatives do not provide safe swimming conditions.

Computations have also been made for the transmission coefficient (Kt) using various well-known formulae available in literature. The values of the coefficient of transmission either computed by the empirical formulae or the results of the SMS model have been presented in (Fig. 24). It has been found that the value of Kt for the actual dimensions of the perched beach (Alt.04) is 24% as per the SMS model, but it is 22%, 38%, 45%, 52% and 68% as computed by [12,13,14,15,16], respectively. It can be concluded that the SMS model results agree reasonably with those computed by [12,13], but [15,14,16] show much higher values. Thus, only the results of [12,13] are considered applicable to the current perched beach. Generally, it can be stated that the maximum transmitted wave height in summer varies from 0.55m to 0.95m as computed by the SMS model, [12,13] formulae.

4.2 Current Velocity

The cross and long shore currents along the centerline of the perched beach for the different alternatives are presented in (Figs. 25-28) during high summer waves, i.e., H=2.50m. It can be noticed that the rip currents before construction of the perched beach are more than 0.7m/s, while they decrease after construction to 0.2m/s inside the perched beach. Also, it can be noticed that the long shore currents are less than 0.2m/s for a distance 60m offshore, but after this distance they increase rapidly to reach 0.46ms/, 0.68m/s and 0.77m/s in case of d/h=0.84, d/h=0.72 and d/h=0.62, respectively, and occur at the offshore toe of the submerged breakwater outside the perched beach. So, it can be confirmed that the higher the breakwater is, the lower offshore currents are. Also, attention should be given to design of the breakwater toe. Furthermore, swimming close to the offshore side of the breakwater is forbidden.

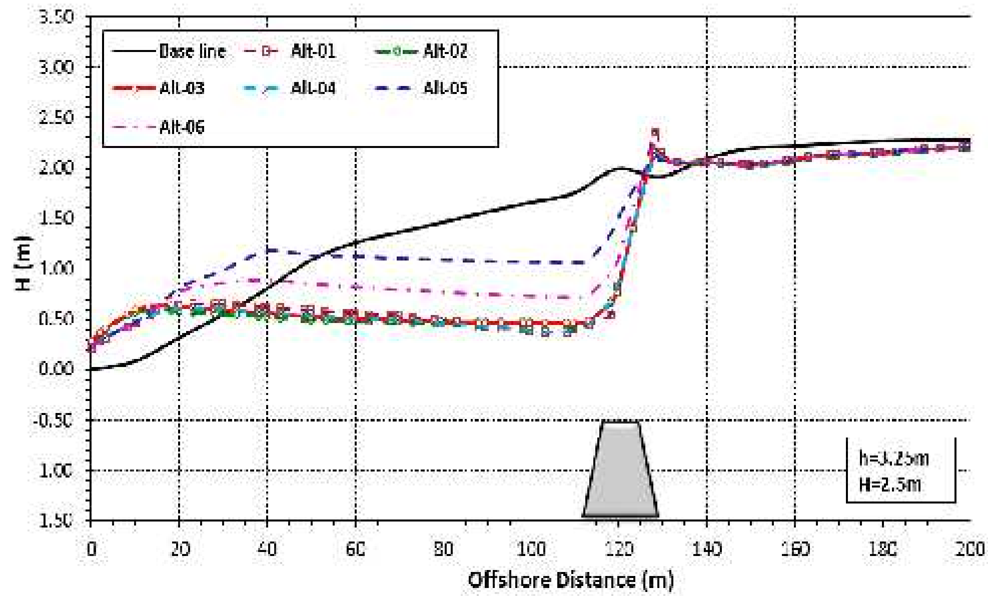

Fig. 21. Computed wave height along the centerline of the perched beach (H=2.50m)

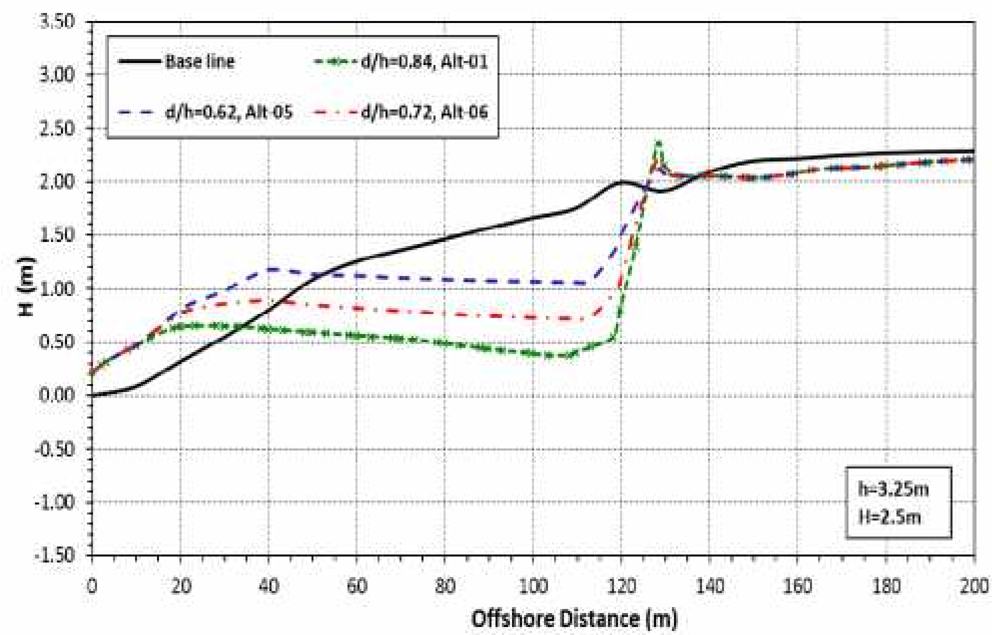

Fig. 22. Effect of d/h on the wave height at the centerline of the perched beach (H=2.50m)

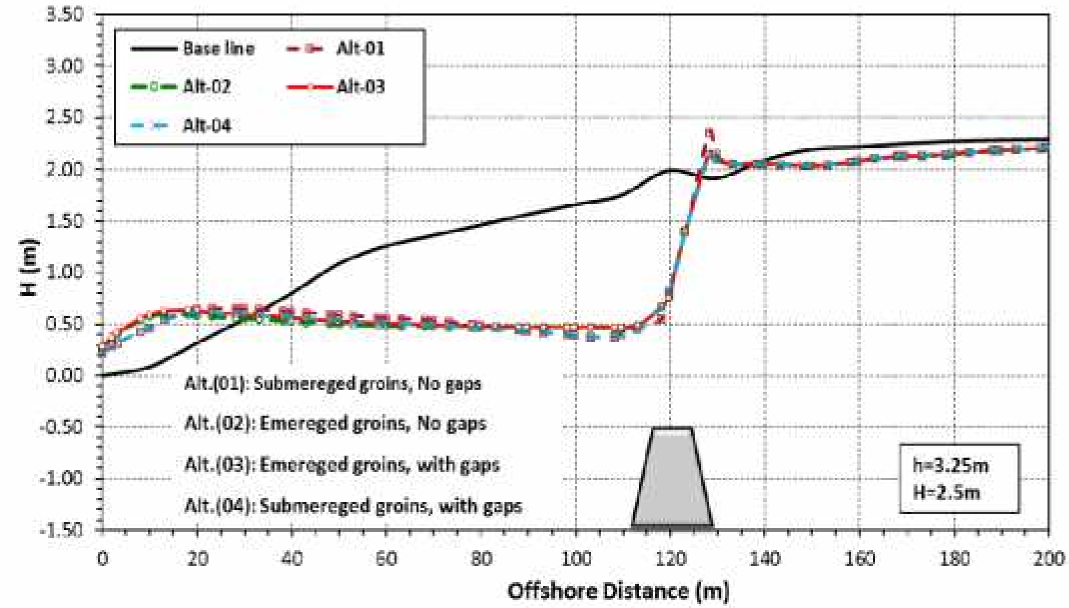

Fig. 23. Effect of the gaps in groin and the emerged/submerged groins on the wave height (H=2.50m)

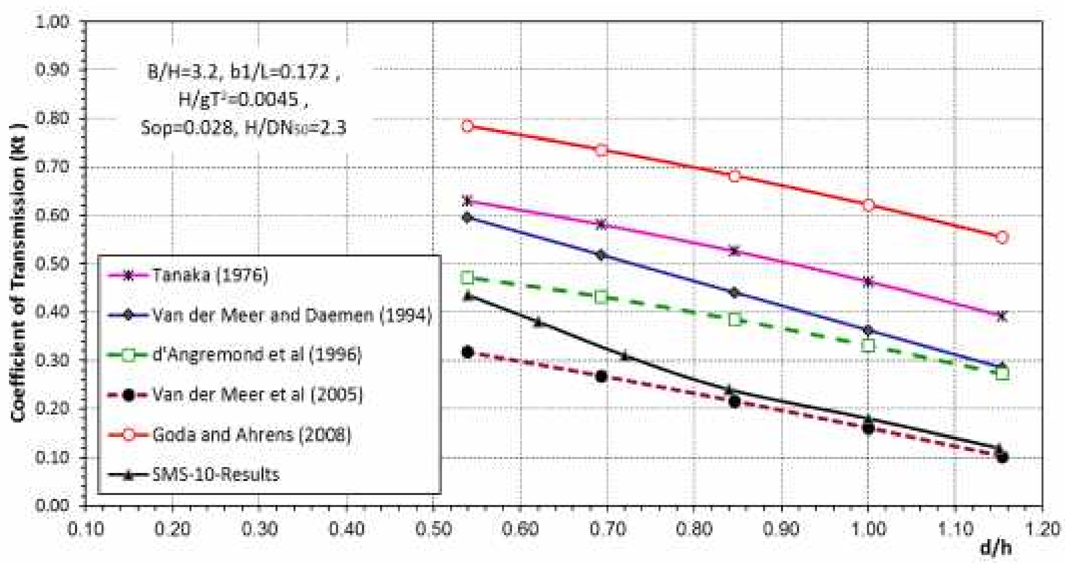

Fig. 24. Computed values of the transmitted wave height using various empirical formulae and SMS

Moreover, it can be observed that the effect of the submergence ratio (d/h) on the rip current is small compared to its effect on the long shore currents. The emerged/submerged groins have significant effect on the cross and long shore currents inside the perched beach. On the other hand, the gaps in the groin have no effect on the cross and long shore currents inside the perched beach, as shown in (Figs. 27-28).

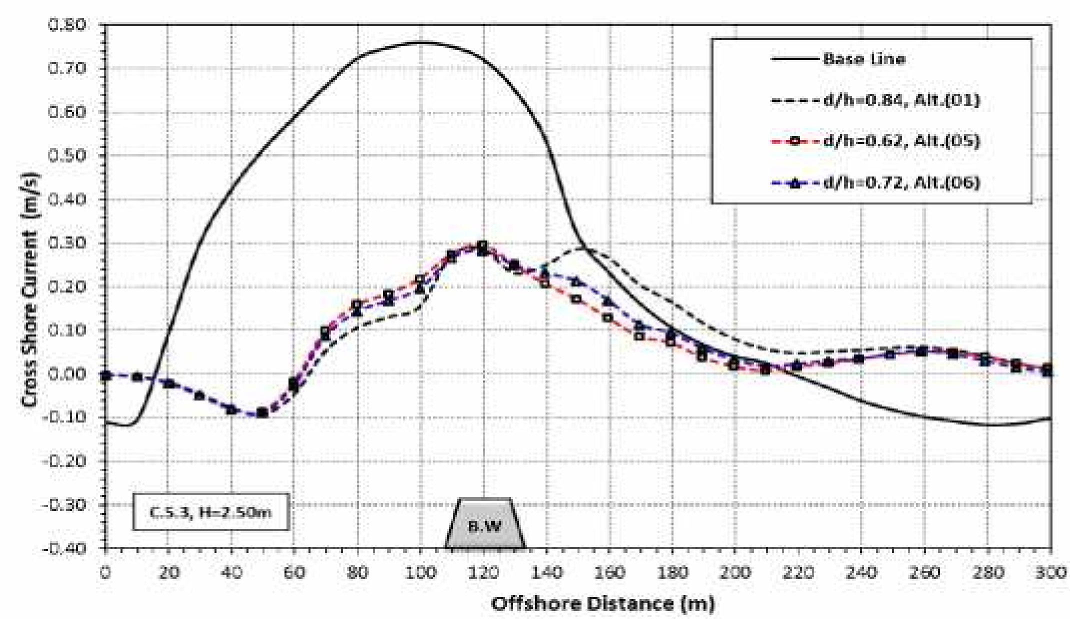

Fig. 25. Rip current velocity along the perched beach for various values of d/h

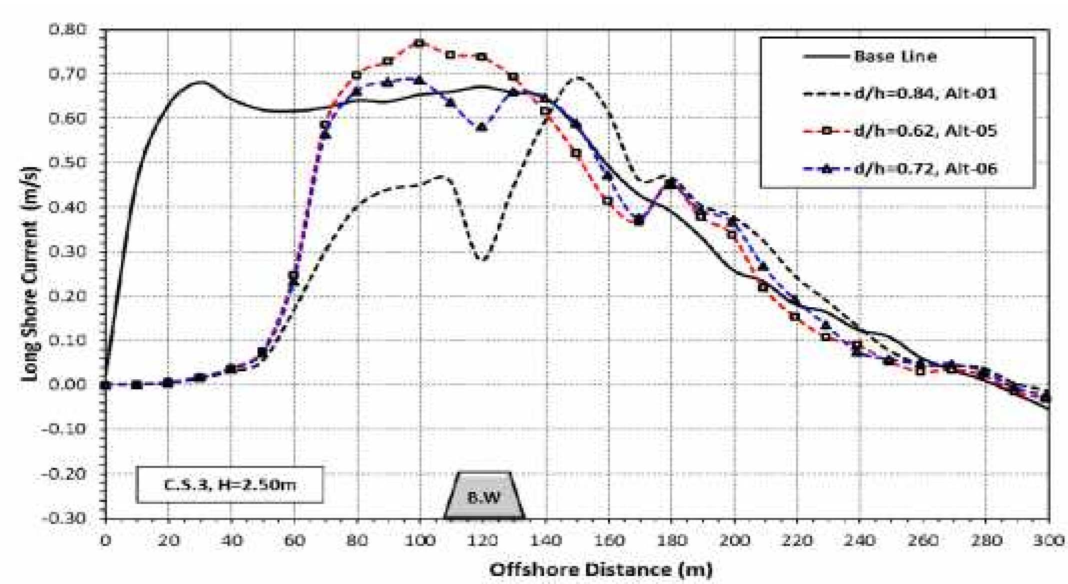

Fig. 26. Long shore current velocity along the perched beach for various values of d/h

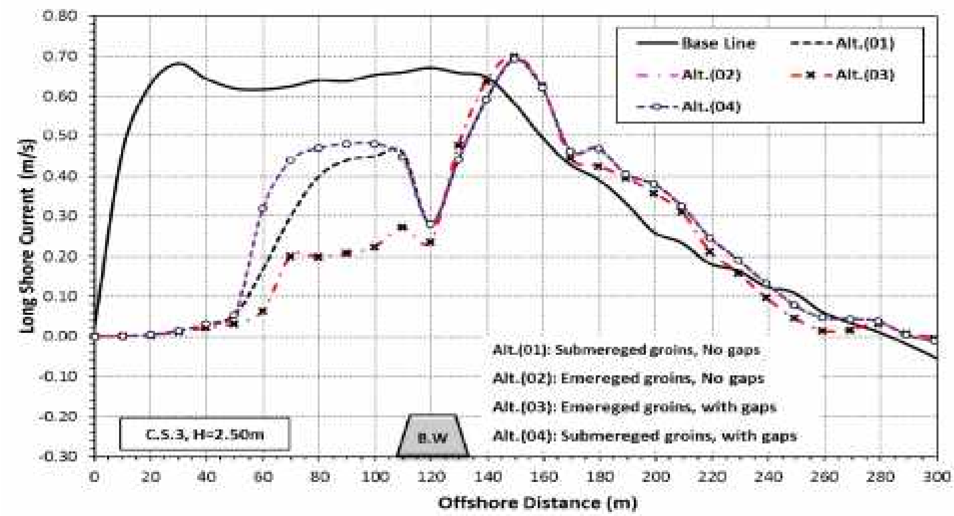

Fig. 27. Effect of the gaps in groin and the emerged/submerged groins on the long shore current velocity along the perched beach (H=2.50m)

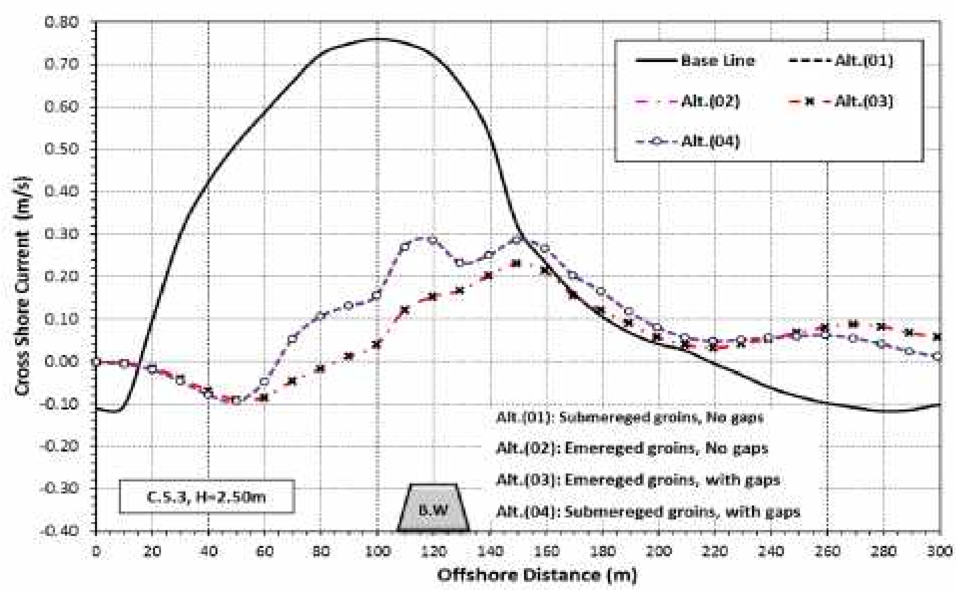

Fig. 28. Effect of the gaps in groin and the emerged/submerged groins on the rip current velocity along the perched beach (H=2.50m)

4.3 Shoreline Changes and Flushing

Computations have been made using the GENESIS [5] program within the package of the SMS model for shoreline locations. Calibration has been made for the sediment transport model (GENESIS) using the bathymetric survey data conducted in 2009 and 2012, before commencement of construction of the perched beach, and the measured in 2013. (Fig. 29) shows the computed and measured shoreline changes after one year from the perched beach construction. A comparison was made between the computed and measured shoreline changes for different cases of different calibration coefficients (K1, K2) in the longshore sediment transport formula. The correlation shown in (Fig. 30) illustrate that the best values of the calibration coefficients for the study site are K1=0.40 and K2=0.08. According to the proposed, the long shore sand transport calibration coefficients, it can be observed that there are small differences between the computed and measured shoreline changes along the up drift and down drift of the structure.

Moreover, computations have also been made for predicting the shoreline changes after complete construction of the perched beach being in January 2013 with the results being presented in (Fig. 31) after 3 years later, i.e., 2016. The longshore sediment transport has been calculated based on the Ozasa and Brampton formula in the SMS model. The net transport has been computed over three years of simulation, including seabed update. The computed net sediment transport rate that corresponds to the period 2009-2012, i.e., before construction of the perched beach, has been found to be 15000m^3/year and its direction is eastward. (Fig. 31) shows the shoreline at the west side of the perched beach progresses along the groin during the period 2013-2016.

The shoreline at the up drift side has been anticipated to progress for 13.3m/year seaward in case of no gap (Alt. 01) and for 11.7m/year if the gap is kept clear (Alt.04) during the period 2013-2016 (Figs. 32-33). Thus accretion is slightly slower in case of the gap than in case of no gap. It can be stated that the gap allows some sediment to cross the groin into the perched beach and hence less accretion occurs at the up drift side. On the other hand, the down drift side has been found to erode at a slow rate. The shoreline at the down drift side has been found to retreat for 4.5m/year in case of no gap (Alt. 01) and for 2.8m/year if the gap is kept clear (Alt.04). As well, erosion is slightly slower in case of the gap than in case of no gap. The accretion rate has been estimated by the numerical model and found to be 9600 to 7200m^3/year at the up drift side. The corresponding erosion rate has been found to be 6850 and 5100m^3/year for the cases of no gap and gap, respectively. Although the erosion is generally within acceptable limits, the gap seems to considerably reduce the erosion rate and shoreline retreat.

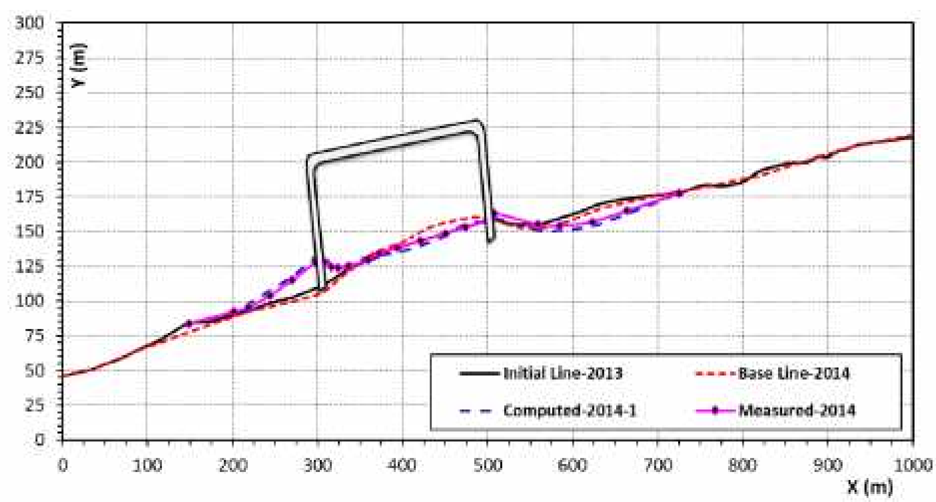

Fig. 29. Computed and measured shoreline changes around the perched beach

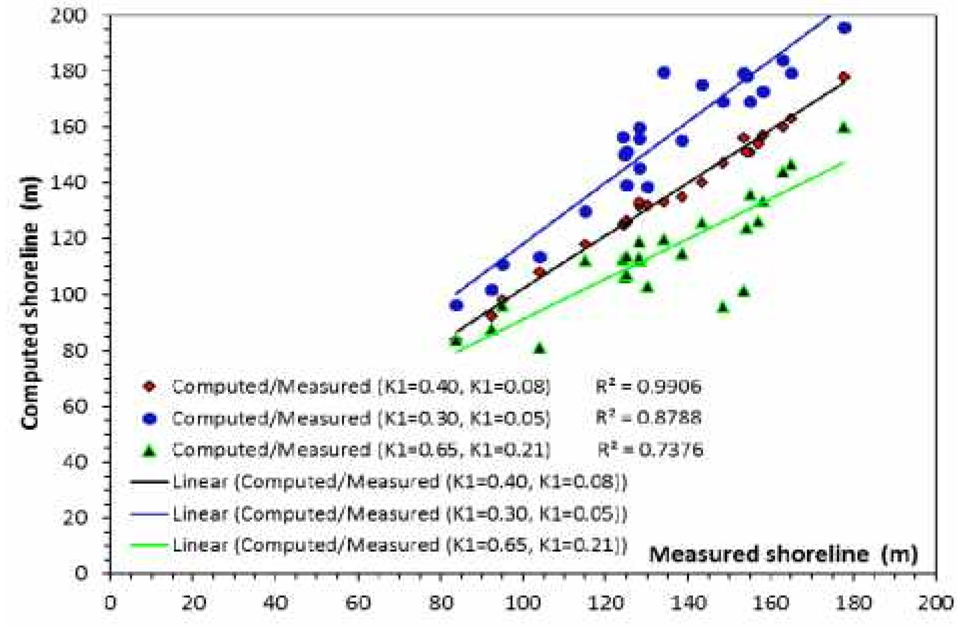

Fig. 30. Correlation of the computed and measured shoreline changes

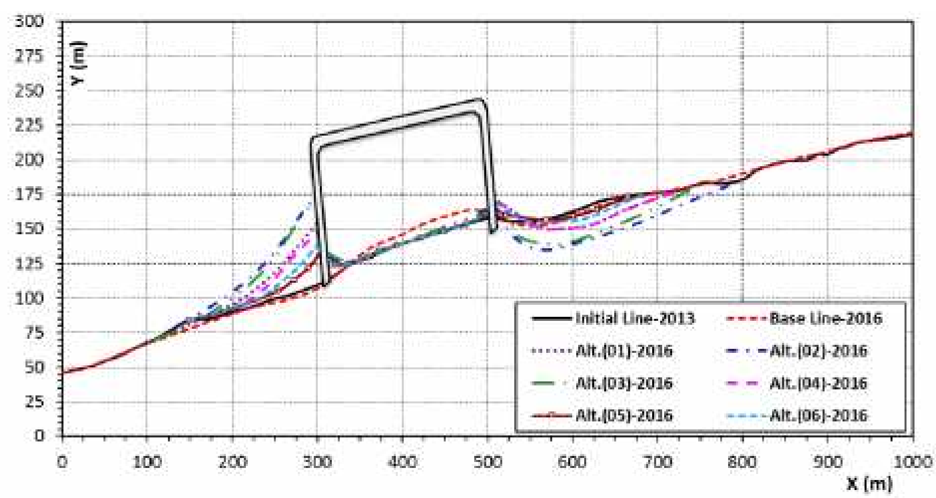

Fig. 31. Shoreline changes around the perched beach for various alternatives after 3 years of construction as compared with the case without a perched beach

Moreover, the submergence ratio has significant effect on the shoreline changes. The shoreline at the up drift side has been found to progress for 13.3m/year, 9.3m/year and 6.7m/year toward the sea in case of d/h=0.84, d/h=0.72 and d/h=0.62, respectively. While, the shoreline at the down drift side has been predicted to retreat for 4.5m/year, 2.5m/year and 1.3m/year toward the shore in case of d/h=0.84, d/h=0.72 and d/h=0.62, respectively. The accretion rate has been found to be 9600m^3/year, 4800m^3/year and 3600m3/year at the up drift side. The corresponding erosion rate has been found to be 6850m^3/year, 2400m3/year and 2050m^3/year for the cases of d/h=0.84, d/h=0.72 and d/h=0.62, respectively (Figs. 32-33).

It can be observed that the emerged/submerged groins have significant effect on the shoreline changes along the up drift and down drift of the perched beach, as well. In case of emerged groins (Alt. 02) the shoreline at the up drift side progresses 20.0m/year and the shoreline at the down drift side retreats 7.2m/year. In case of submerged groins (Alt. 01), the shoreline at the up drift side progresses 13.3m/year toward the sea and the shoreline at the down drift side retreats 4.5m/year toward the shore. The accretion rate has been found to be 20160m^3/year and 9600m^3/year at the up drift side for Alt. 02 and Alt.01, respectively. The corresponding erosion rate has been found to be 15900m^3/year and 6850m^3/year for the cases of emerged groins and submerged groins, respectively. Although the erosion is generally within acceptable limits, the submerged groins/breakwater seems to considerably reduce the erosion rate and shoreline retreat.

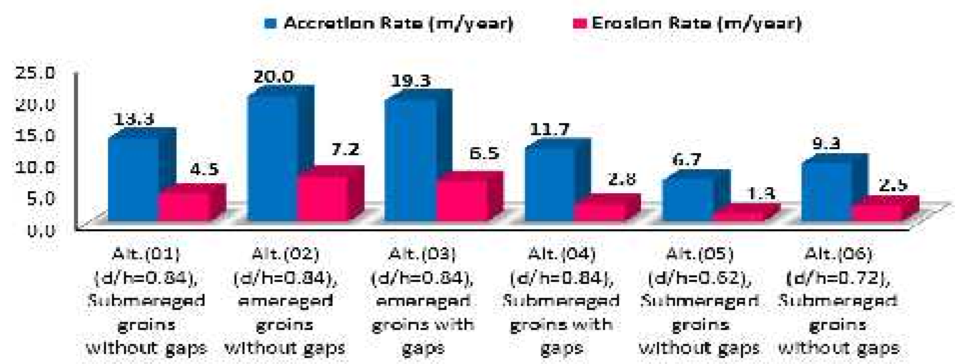

Fig. 32. Accretion and erosion rates around the perched beach for various alternatives

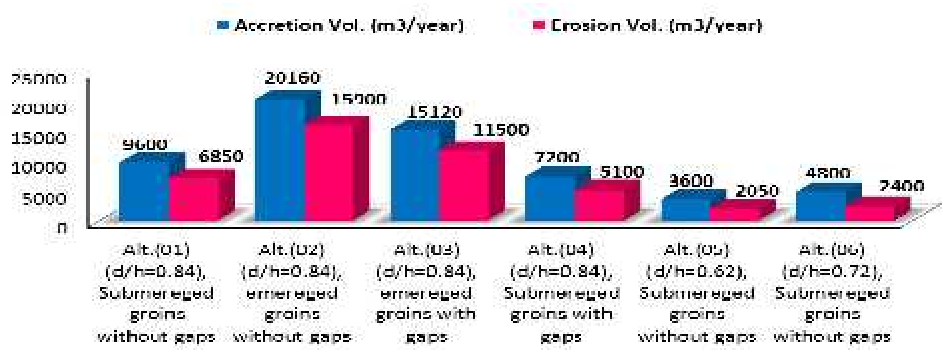

Fig. 33. Accretion and erosion volume rates around the perched beach for various alternatives

To minimize and mitigate erosion at the down drift, sand nourishment is considered to be the most straightforward soft approach. Thus, it is possible to move the sand from the accretion zone in the up drift to nourish the erosion area in the down drift. This will allow coarse sand particles deposited at the up-drift to nourish the erosion zone in the down drift instead of the finer particles, and hence erosion rates would decrease over time.

For the constructed alternative in the pilot area (Alt.04), it is recommended to perform the sand transport once every 3 years to limit the maximum shoreline retreat to a maximum of 10m while keeping the gap in good working condition. It is also evident that the shoreline at the up-drift side is almost unaffected by the perched beach after 100m away from the west groin within the period of 2013-2016, which is less than the groin length. On the other hand, the extent of erosion at the down drift side is limited to about 200m away from the east groin during the period 2013-2016, which is less than twice the groin length. The shoreline in the protected area has been found to be almost unaffected during the period 2013-2016 due to the low currents. It is noteworthy that the spacing between the groins in the protected area in 2013 and 2016 has been artificially made by dredging early in 2013 to adjust the beach profile in the protected area. The annual nourishment to mitigate the erosion at the down drift, is estimated to be 6300m^3. The grain size and schedules for the nourishment should be carefully prepared taking into account the social impact.

Further computations have been made for the flushing rate based on the relation between the tidal range and the structure height. It has been found that the flushing rate inside the perched beach ranges from 2 to 3 days, during the frequent occurrence of summer storms (H=2.5 and 1.77m) and is always less than 6 days most of the time. These values are in a good agreement with the required flushing time in hot climate, which is between 4 and 6 days. It is known that the long flushing time will lead to deterioration in the water quality.

4.4 Assessment of Alternatives

[2] Conclude that the comfortable swimming conditions exist when breaker heights are smaller than 0.6m and current velocities smaller than 0.2m/s. Furthermore, the required flushing time in hot climate is between 4 and 6 days. Based on the stated comfortable swimming conditions and the results data, it can be stated that the proposed alternatives are acceptable from a flushing point of view. It can be observed that the transmitted wave height and the current velocities are acceptable for the alternatives: Alt. (01), Alt. (02), Alt. (03), and Alt. (04) but are unacceptable for the alternatives: Alt. (05) and Alt. (06).

Although, the shoreline changes are minimal in case of alternatives: Alt. (05) and Alt. (06), nonetheless, these alternatives are unacceptable for the transmitted wave height and current velocities. Also, the flushing rate inside the perched beach is always less than 6 days in all alternatives. Moreover, the shoreline changes in case of alternatives: Alt. (05) and Alt. (06) are larger than the other alternatives.

It can be judged that the configurations of alternatives: Alt. (01) and Alt. (04) are the most suitable, but still Alt. (04) is the better due to the groin gaps effectiveness. Thus, it can be considered that the configurations of alternative (04) provide guidelines for the design of a perched beach within the Al-Arab Bay zone in Cell 5 of the North West coast of Egypt.

5. CONCLUSION

This study has been conducted to investigate the actual case of a perched beach as a pilot project along the North-West coast of Egypt. The main purpose of the perched beach is to provide safe swimming during the summer seasons for visitors of a coastal resort while minimizing the possible negative impacts on the environment. Investigations have been made for the case before and after construction of the perched beach to clarify the impacts of the pilot project using the SMS numerical model. The results of the study can be used for the application of new coastal protection systems. The following points have been concluded:

- Large current velocities and eddies are generated offshore the submerged breakwater and offshore the west groin. The need for extra toe protection at these locations is far more evident than other locations. Attention should be given to bed scour and toe design.
- The wave energy dissipation depends greatly on breakwater height. It is shown that the higher the breakwater is, the lower the transmitted but the slightly higher the reflected wave energies are.
- The combined use of empirical formulae and the results of the numerical models to estimate the transmitted wave height and design of a submerged breakwater are useful tools in coastal studies especially along the North-West coast of Egypt. It has been found that the value of Kt for the actual dimensions of the perched beach is 24% as per the SMS model.
- The higher the breakwater is, the lower offshore currents are. It should be taken into consideration to pay close attention to the breakwater toe and to swim close to the breakwater due to large currents occurring around it.
- The effect of the submergence ratio (d/h) on the rip current is small compared to their effect on the long shore currents.
- The emerged/submerged groins have significant effect on the cross and long shore currents inside the perched beach. On the other hand, the gap in groin has no effect on the cross and long shore currents inside the perched beach.
- The accretion is slightly slower in case of the gap than in case of no gap. It can be stated that the gap allows some sediment to cross the groin into the perched beach and hence less accretion occurs at the up drift side. On the other hand, the down drift side has been found to erode at a slow rate.
- The submergence ratio has significant effect on the shoreline changes. Also, the emerged/submerged groins have significant effect on the shoreline changes along the up drift and down drift of the perched beach, as well.
- To minimize and mitigate the erosion at the down drift, sand nourishment is considered to be the most straightforward soft approach to beach erosion. This will allow coarse sand particles deposited at the up-drift to nourish the erosion zone in the down drift instead of the finer particles and hence erosion rate would become less over the time.
- The configurations of alternatives: Alt. (01) and Alt. (04) are the most suitable, but still Alt. (04) is preferred due to the groin gaps effectiveness. So, it can be considered that the configurations of Alternative (04) provide guidelines for the design of a perched beach within the Al-Arab Bay zone in Cell 5 of the North West coast of Egypt.
- For the constructed perched peach (Alt.04), it has been found that:
  - The maximum length of erosion in the down drift (east of the jetties) is less than twice the jetty length and the shoreline retreats at less than 3m/year, but the maximum accretion length is less than the jetty length and the shoreline progresses is less than 12m/year. The rates of sediment transport up drift and down drift the perched beach are 7200 and 5100m3/year, respectively, and sand nourishment is recommended once every three years. The residual adverse impacts could be generally alleviated through a routine sand nourishment program of the shoreline.
  - The flushing rate inside the perched beach has been found to range from 2 to 3 days, during the frequent occurrence of summer storms (H=2.5 and 1.77m) and

is always less than 6 days most of the time. These values are in a good agreement with the required flushing time in hot climate, which ranges from 4 and 6 days.

COMPETING INTERESTS

Authors declare that there are no competing interests.

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