Numerical Study on the Use of Low Crested Detached Breakwaters along the Northern Coast of Egypt

S. Abohadima[1*], R. Marmoush[2] and K. A. Rakha[3]

[1]Faculty of Engineering, Department of Mathematics and Eng. Physics, Cairo University, Egypt.
[2]Dar El- Handasah Consulting company, Egypt.
[3]Faculty of Engineering, Department of Irrigation and Hydraulics, Cairo University, Egypt.

Authors' contributions

This work was carried out in collaboration between all authors. Author RM performed the numerical model runs and the analysis for the results, and wrote the first draft of the manuscript and managed literature searches. Author SA designed the study. Authors SA and KR managed the analyses of the study and literature searches and revised the manuscript. All authors read and approved the final manuscript.

Original Research Article

ABSTRACT

Many resorts were constructed along the northern coast of Egypt for recreational purposes. High waves often occur in the used-zone leading to uncomfortable conditions. Moreover, the resulting longshore current with high speeds due to wave breaking leads to undesirable current flow. Rip currents generated in the used-zone can also be a hazard to swimmers This study aims at testing the possibility of using detached low crested submerged breakwaters to provide suitable swimming conditions along the northern coast of Egypt. Detached low crested submerged breakwaters are considered as they do not obstruct the sea view and have lower negative impacts on the environment. This study attempts to provide a numerical examination for several configurations using different breakwater and the gap lengths. Three numerical models were used; refraction diffraction model for waves, hydrodynamic model for current circulation and one line model for shoreline movement. The best breakwater configuration to provide safe swimming area is recommended with an assessment of the negative impact on the shoreline stability. This

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Corresponding author: E-mail: s@abohadima.com;

study will be useful for the recreational resorts along the northern coast of Egypt.

Keywords: Numerical modeling; rip currents; water waves; shoreline movement; detached breakwaters.

NOMENCLATURE

α *Tensor index (= 1 or 2 for x or y direction)*
Q_α *is the total flux volume*
$S_{\alpha\beta}$ *is the radiation stress*
$\mathscr{U}_\alpha$ *is the total velocity in the horizontal directions (α direction),*

$\mathscr{U}_{w\alpha}$ *is the horizontal shortwave induced particle velocity in the (α) direction,*

V_α *is the total current velocity,*
$V_{d\alpha}$ *is the depth varying current,*
ζ_t *is the elevation of the wave trough,*

τ_β^{S} *And* τ_β^{B} *are surface and bottom shear stress,*

$\tau_{\alpha\beta}$ *is the horizontal turbulent stresses in the fluid,*
t is time,
D_B *is berm height,*
D_c *is the closure depth,*
q is a source or sink in sediment
Q_s *is the longshore sand transport rate (m3/sec)*
H is the significant wave height,
C_g *is the wave group speed,*
θ_{bs} *is the angle of breaking with shoreline, the subscript "b" denotes breaker.*
A is the wave amplitude,
k_o *denotes a constant wave number*
ϕ is the velocity potential;
C is the wave celerity;
Cg is the group velocity;
k is the wave number
K_1 *and* K_2 *are empirical coefficients,*
ρ_s *and* ρ *are the densities of sand and water, respectively,*
p is the sand porosity,
y is the shoreline position
*tanβ is the average bottom slope from shoreline to closure depth,*

1. INTRODUCTION

During the summer season in Egypt, many residents spend their vacation along the northern coast (Figs. 1 & 2). The regular summer beach activities are swimming, waddling and paddling besides enjoying the sea view. Swimming is considered the most important activity within sea water. Swimming takes place in the surf zone extending offshore about 75 meters from the shoreline. This reach of sea will be denoted as the "used-zone".

Often high waves reach the used-zone leading to uncomfortable and in many cases dangerous conditions during swimming. Moreover, the resulting longshore current with high speeds due to wave breaking leads to undesirable swimming conditions. Cross currents such as rip currents generated in the surf zone can also be a hazard to swimmers.

According to coastal guard manuals and open swimming racing regulations, a value of 0.5 m wave height and 0.3 m/sec. current velocity (i.e., longshore and/or crossshore) are the maximum acceptable values for safe swimming conditions.

In Egypt, several resorts such as Nakheel, Kerir, Marabella and Marina constructed protection systems to control the used-zone hydrodynamics as shown in (Figs. 1 & 2). These systems led to negative impacts on the shoreline stability with erosion to the east of the structures constructed [1].
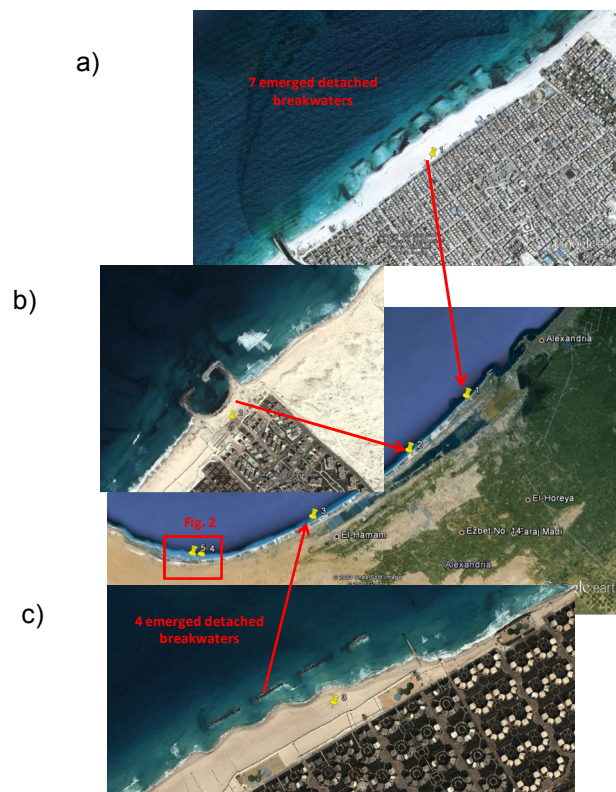
Fig. 1. Sample of Projects along Northern Coast of Egypt West of Alexandria (a, b and c are Nakheel, Kerir and Marabella respectively).

Because of the tourism value of the northern coast beaches, the proposed protection system should not obstruct sea view or reduce the water quality.

Numerous previous studies covered the mechanism of protection systems to control hydrodynamics with associated environmental impacts (specially submerged detached breakwaters).

Six selected project sites were studied by [2] with different geometric characteristics and in wide-ranging climate conditions that were monitored and analyzed during the DELOS. They investigated characteristics of the European structure protections by Low Crested Structures (LCSs). In addition, their study described the sites and prototype observation of the impact of LCSs; a) description of the site, environmental conditions and response to its construction , b) description of the ecological impacts and socioeconomic effects where available.

Fig. 2. Sheltered swimming areas about 90 km west of Alexandria (Marina zone)

The study by [3] confirmed the inability to clearly differentiate between erosive and accretive shoreline response since submerged structures can result in both shoreline erosion and accretion. The published prototype observations do not even indicate a clear relationship between the mode of shoreline response and measurable environmental and structural variables (i.e., erosion resulted for: long structures as well as short structures, structures with higher crest levels as well as lower crest levels, both broad-crested and narrow-crested structures and both steep and mild bed slopes).

It was confirmed however, in general that under oblique wave incidence the superposition of the unidirectional longshore current (which is weakened in the lee of a submerged structure) on the nearshore circulation pattern resulting from the flow over the structure appears to result in a gradient in the longshore current. At the up-drift side, the structure induced nearshore circulation would oppose the ambient longshore current, resulting in a lower net longshore current in this region. At the down-drift side, the two currents converge, and the

net longshore current is enhanced. The net result is deposition of sediment on the up-drift section of the shoreline behind the structure, and some erosion on the down-drift section. Along coastlines with substantial longshore sediment transport (e.g. the Gold Coast, Australia) this mechanism can account for the development of a salient in the lee of the structure.

A simple relationship between the breaking parameter (ɣ2) and the relative submergence (dc/Hm0) was proposed by [4]. This relation can be used in wave models for modeling wave transmission and wave energy decay over submerged breakwaters. The calculated transmission coefficients using the provided approach were compared with values calculated using the empirical equations by [5] and [6]. For this purpose, six hypothetical test cases were considered with varying submergence depth.

The study by [7] concluded that Incident and transmitted wave angles are not always similar: for rubble mound structures, the transmitted wave angle is about 80% of the incident one, whereas for smooth structures the transmitted wave angle is equal to the incident one for incident wave angles less than 45degrees and is equal to 45degrees for incident wave angles larger than 45 degrees.

In this study the usage of a series of Submerged Low Crested Detached Breakwaters (SLCDBWs) is tested to alleviate the above mentioned problems. The present study aims to investigate the efficiency of using SLCDBWs in order to provide safe swimming for the Northwestern Coast of Egypt. For this purpose numerical models were used to study several alternative layouts for SLCDBWs.

The acceptable conditions used in this study are 0.5 m for wave height and 0.3 m/s for currents.

2. NUMERICAL MODELS

Three numerical models were used to simulate the waves, hydrodynamics and shoreline changes. The REF/DIF numerical model was used to model the nearshore wave field and the radiation stresses. The SHORECIRC numerical model was used to predict the wave-induced currents. Finally the GENESIS shoreline change model was used to predict the shoreline changes due to the different alternatives. The validation of these models using published data for other areas was provided in [8].

2.1 Refraction Diffraction Model

The REF/DIF Model is a steady-state model that can simulate wave shoaling, refraction, diffraction and energy dissipation. The REF/DIF model was developed by [9]. The model solves an approximation of the mild slope equations (MSE). The parabolic mild slope equations (PMSE) [10] is solved that accounts for wave shoaling, refraction, diffraction, and breaking for monochromatic waves. Wave reflection is not included in the model.

The MSE are valid for bed slopes up to 1:3 [11] and can be written as

$$\nabla\left(CC_g\nabla\phi\right) + k^2 CC_g\phi = 0 \tag{1}$$

Assuming that the velocity potential can be divided into a forward and a backward component and neglecting the backward component (neglecting the reflected waves), results in the parabolic form of the MSE would be

$$i\,2kCC_g\,\frac{\partial A}{\partial x}\;+\;\frac{\partial}{\partial y}\left(CC_g\,\frac{\partial A}{\partial y}\right)\;+\left[2k\,(k-k_o)CC_g+i\,\frac{\partial}{\partial x}(kCC_g)\right]A=0 \tag{2}$$

Furthermore a dissipation term may be added to include wave energy dissipation due to breaking as provided by [9].

The PMSE considers wave diffraction in the direction perpendicular to the direction of propagation only (y-direction) and neglects wave diffraction in the direction of propagation (x-direction). This is a reasonable approximation only when the solution marches in the direction of wave propagation. It is found by [12] that the angle between the wave direction and the direction of propagation (x-axis) should not exceed 30 degrees.

The main input to the model is the bathymetric data and the offshore wave rose. For the bathymetry, straight and parallel contours were assumed with the orientation of the shore that same as that for the first and second projects shown in Fig. 1. A beach slope of 50:1 was used based on data from several locations in the study area.

Offshore wave data from the Western European Armaments Organization Research Cell (WEAORC 2004) was used [13]. This Atlas includes large records of predicted (hindcast) wave data at certain points over the Mediterranean. The present study used the wave data at point with (32 N and 29 E) which was selected as it is the closest location for the present study area. Fig. 3 provides a plot of the wave rose at this location where it can be seen that the predominant waves are from the NW.
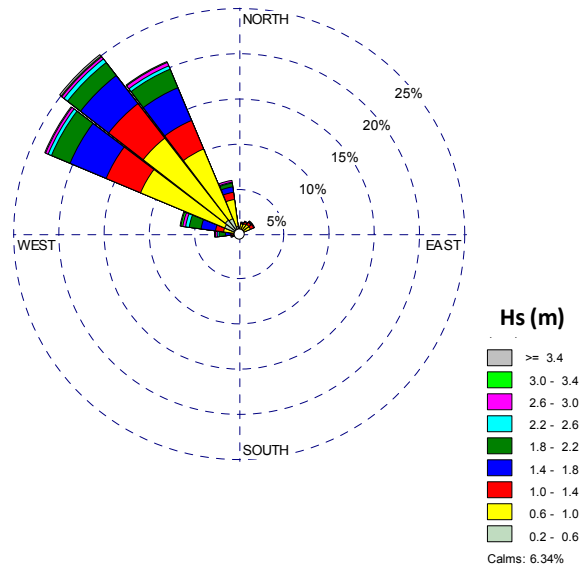

Fig. 3. Offshore wave rose

The summer wave data was extracted from the total year records to provide records for the probability of occurrence for different wave heights and the associated directions. Also it

provides records for the probability of occurrence for different wave heights and the associated peak periods. The summer season covers months of June, July and August.

These data were analyzed to determine the predominant wave conditions for the summer season. The highest probability of occurrence was found to be between 0.875 m to 1.125 m and propagates from the direction making 315 degrees with North. Based on this analysis, the predominant wave characteristics for numerical modeling were: 1.0 m as an average value of the wave heights with associated wave period 6.5 seconds from the NW direction which is equivalent to 315 degrees with north and makes 15 degrees with shoreline.

The computational domain dimensions are 900 (parallel to shore) m x 700 (cross the shore) m with grid spacing 2.5 m x 2.5 m with no sub-grids or subdivisions. Thus, the numbers of computing points are 361 and 281 respectively.

One unit of SLCDBW is installed in the middle of the domain associated with two half units at the right and left boundaries to avoid the cross boundaries effect. In all proposed alternatives, the SLCDBW are located 100 m from the shore or at 2 m depth and parallel to the shoreline.

2.2 Water Circulation Model

The SHORECIRC model is a quasi-3D finite difference model that combines the effects of vertical structure of the currents with the simplicity of a 2D-horizontal model for the nearshore circulation. This is done by using an analytical solution for the 3D current profiles combined with the numerical solution for the 2D depth-averaged horizontal equations. The model was originally developed by [14] and several improvements were included by different researchers [e.g. 15].

The SHORECIRC model is based on the depth-integrated, time-averaged equations of continuity and momentum, which in complete form and in tensor notation takes the following form:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial Q_\alpha}{\partial x_\alpha} = 0$$

(3)

For continuity and,

$$\frac{\partial Q_\beta}{\partial t} + \frac{\partial}{\partial x_\alpha}\left(\frac{Q_\alpha Q_\beta}{h}\right) + \frac{\partial}{\partial x_\alpha}\int_{-h_0}^{\overline{\zeta}} V_{d\alpha} V_{d\beta} dz + \frac{\partial}{\partial x_\alpha}\overline{\int_{h_0}^{\zeta}\left(U_{\omega\alpha}V_{d\beta} + U_{\omega\beta}V_{d\alpha}\right)dz} +$$

$$g\left(h_o + \overline{\zeta}\right)\frac{\partial \overline{\zeta}}{\partial x_\beta} - \frac{\tau_\beta^S}{\rho} + \frac{\tau_\beta^B}{\rho} + \frac{1}{\rho}\frac{\partial}{\partial x_\alpha}\left(S_{\alpha\beta} - \overline{\int_{-h_0}^{\overline{\zeta}} \tau_{\alpha\beta} dz}\right) = 0$$

(4)

For momentum where,

$$Q_\alpha = \int_{-h_0}^{\overline{\zeta}} U_\alpha dz$$

(5)

$$S_{\alpha\beta} = \overline{\int_{h_0}^{\zeta}\left(\rho\delta_{\alpha\beta} + \rho\, U_{\omega\alpha} U_{\omega\beta}\right)dz} - \delta_{\alpha\beta}\frac{\rho g h^2}{2} - \rho\frac{Q_{\omega\alpha} Q_{\omega\beta}}{h}$$

(6)

The third and fourth (integral) terms in the momentum represent the effect of the depth variation of the current on the horizontal distribution of volume flux Q_α. Noting that $S_{\alpha\beta}$ and $Q_{w\alpha}$ are outputs from the REF/DIF Model.

The same grid size and domain used for the REF/DIF model was used in this model. The radiation stresses calculated from the wave data using linear wave theory is used as the input driving forces to the model.

2.3 Shoreline Change Model

The GENEralized model for SImulating Shoreline change (GENESIS) is a finite difference one line model that assumes the beach profile shape remains constant and performs time dependent sediment budget analysis to calculate the shoreline change over an arbitrary beach. Details on the model may be found in [16, 17, 18]. The model includes a wave transformation model to calculate shoaling, refraction, and diffraction. Wave transmission at detached breakwaters and a variety of terminal and internal boundary conditions and constraints can be included.

The governing equation for the rate of change of shoreline position can be expressed as,

$$\frac{\partial y}{\partial t} + \frac{1}{(D_B + D_c)}\left[\frac{\partial Q_s}{\partial x} - q\right] = 0 \tag{7}$$

The empirical predictive formula for Qs is,

$$Q_s = (H^2 C_g)_b\left[a_1 \sin 2\theta_{bs} - a_2 \cos\theta_{bs}\frac{\partial H}{\partial x}\right]_b \tag{8}$$

The first term in Eq. (8) corresponds to the "CERC formula" described in the [19] and accounts for longshore sand transport produced by obliquely incident breaking waves.

The second term in Eq. (8) is not a part of the "CERC formula" but describes the effect of the longshore gradient in breaking wave height which is usually much smaller than that from oblique wave incidence effect unless diffracted waves exist due to vicinity structures.

The non-dimensional parameters a_1 and a_2 are given by,

$$a_1 = \frac{K_1}{16(\rho_s/\rho - 1)(1-p)(1.416)^{5/2}} \tag{9}$$

$$a_2 = \frac{K_2}{8(\rho_s/\rho - 1)(1-p)\tan\beta(1.416)^{7/2}} \tag{10}$$

These coefficients were calibrated by [20] for the GENESIS model by simulating the shoreline change for the beach at Marabella and Suez canal resorts along the Egyptian Northwestern coast using the data covering a period from 1990 to 1994. The calibration resulted in the following values; $K_1 = 0.05$ and $K_2 = 0.04$. These values were used in this study.

The computational domain dimensions are 3300 (parallel to shore) m. the extra 1000 m from left and right sides (compared to wave model) were included to avoid the boundary effects.

3. ALTERNATIVES STUDIED

One of the studied sites in [2] is Lido di Ostia. It's touristic beaches in Italy lying along Mediterranean Sea were suffering from erosion. Selection of LCS was based on aesthetics - not to obstruct sea view- and environmental -water refreshment- reasons. Function of LCS is to hold the artificial beach at a shallower slope, reducing both offshore sand losses and long-shore transport and enhancing the development of marine fauna without endangering bathing and leisure navigation.

LCS system geometry consists of:

  - 2800 m continuous -no gaps- LCS parallel to shoreline with associated submerged groins.
  - 100 distance between shoreline and LCS.
  -Water depth at LCS location is 4 m.
  - Water depth above LCS crest is 2 m.
  - LCS crest width is 15 m.
  - Beach slope 1:250.

On the other hand, the environmental conditions at the Lido di Ostia beach are:

  - Typical wave height is 1.0 m
  - Wave direction is SW
  - Spring tidal range 0.4 m.

Historical observation of beach evolution shows an obvious tendency to equilibrium after using LCS and nourishment processes.

Because of the high similarity in environmental conditions between the present study area and Lido di Ostia, most of the proposed LCS configurations were similar to those at Lido di Ostia.

A number of alternatives for protection systems with various configurations of a SLCSDBW were proposed and examined from all related aspects to determine the optimum geometry to be used along the Northwestern Coasts of Egypt.

[2] concluded that the main parameters governing the LCS efficiency are: a) LCS distance from shoreline and b) water depth above crest (negative freeboard). Moreover, [21] stated that the most important parameters controlling the detached breakwaters efficiency are: a) breakwater length, b) breakwater distance from shore and c) Surf zone width. Therefore, some parameters remain constant in all alternatives such as breakwaters side slope, orientation and crest width as they will not have a significant effect on the resulting hydrodynamics.

The present study focuses on only two main factors that can majorly affect the hydrodynamics changes which are the breakwaters lengths and gaps widths while all other variables remained constant. The study examines five cases of protection systems with different breakwater lengths and gap widths for the same numerical domain. For simplicity, each case was defined based on the blocking percentage which refers to the ratio of breakwater lengths to the total beach length. The following are the examined cases (Fig. 4):

- Case 0: 0 % blocking (Do nothing) which represents the natural condition
- Case 1: 33 % blocking with 150 m SLCDBW lengths and 300 m gap length
- Case 2: 56 % blocking with 250 m SLCDBW lengths and 200 m gap length
- Case 3: 78 % blocking with 350 m SLCDBW lengths and 100 m gap length
- Case 4: 100 % blocking with 900 m SLCDBW length with no gaps

Another classification of the proposed protection alternatives can be provided according to the ratio between the SLCDBWs length (B) to gap between SLCDBWs width (G):

- Case 0: (B/G=0) where SLCDBWs are not used
- Case 1: (B/G=0.5) where B=150 m and G= 300 m
- Case 2: (B/G=1.25) where B=250 m and G= 200 m
- Case 3: (B/G=3.5) where B=350 m and G= 100 m
- Case 4: (B/G=infinity) where continuous SLCDBW with no gaps.

Seven transversal profiles were chosen at the most critical locations for the waves and current fields to display the behavior/values in all cases (Fig. 5).
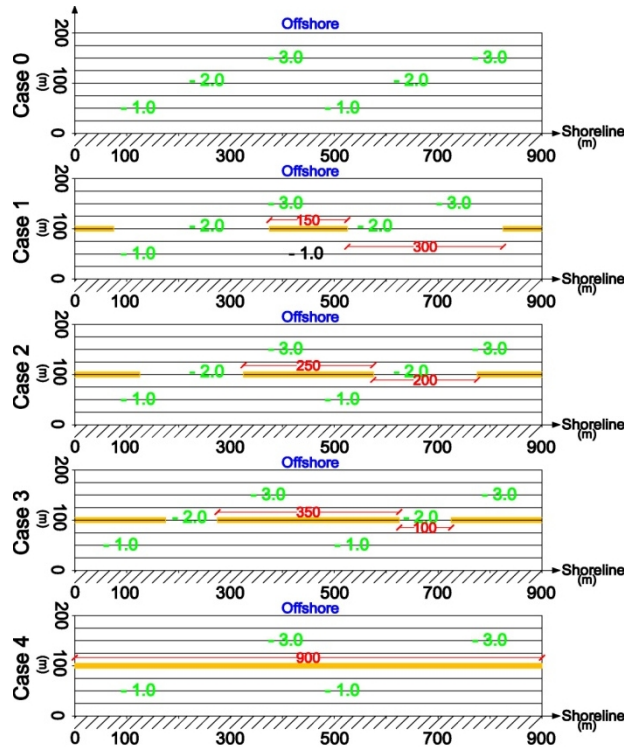
Fig. 4. Proposed cases of protection systems with SLCDBWs
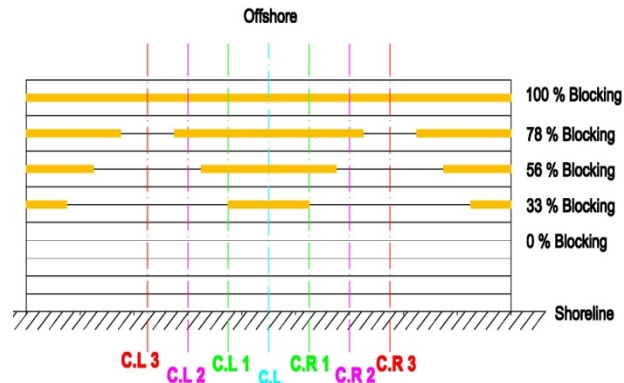
Fig. 5. Locations of transversal profiles performed in wave and current fields.

4. MODEL RESULTS

4.1 Wave Model

- In Case 0, the offshore limit of used-zone has waves with 1.1 m height which exceeds the 0.5 m (maximum safe wave height).
  High dissipation of energy at a water depth of 1.5 m -75 m offshore.
- In Case 1 and within blocked zones, the transmitted wave in lee side is 0.5 m high directly behind the SLCDBWs (Fig. 6b).
  Diffracted waves around SLCDBWs rounded heads towards the sheltered area results in 0.7 m height behind SLCDBWs directly and approach the surf zone at 0.6 m height (Fig. 6c).
  In gaps zones, waves propagate exactly as if there is no protection (similar to Case 0) (Fig. 6e).
- In Case 2 and Case 3, the previous description of Case 1 can be applied however the length of sheltered areas behind protection system is directly proportional to SLCDBW length (Fig. 6a and 6d).
- For Case 4, the resulting wave field is very similar in behavior to the blocked reaches in the previous examined Cases (1, 2 and 3) without any diffraction phenomena (Fig. 8).
  [21] used a modified relationship for a case of relative submergence equal to 1.06 that gives breaking parameter equal to 1.15 and a transmission coefficient of 0.76. Other empirical equations by [5] and [6] give 0.59 and 0.68 respectively.
  Results of model show good agreement with previous values (Transmission coefficient is 0.5 for a relative submergence of 1).
  Fig. 6a, shows slight diversion of incident waves over SLCDBWs. Value of diversion complies with the 80% reduction in wave angle for rough rubble mounds concluded in [7].
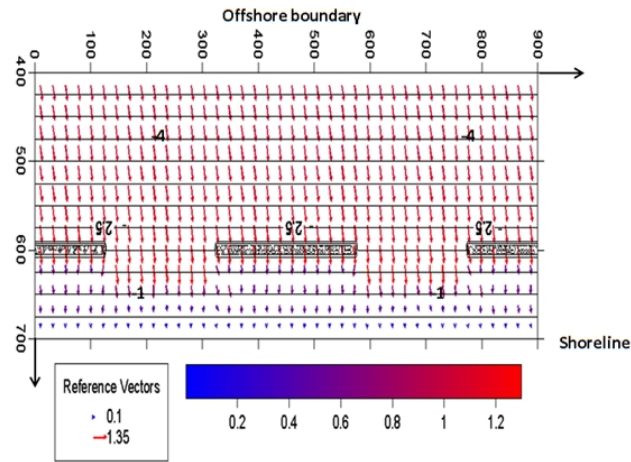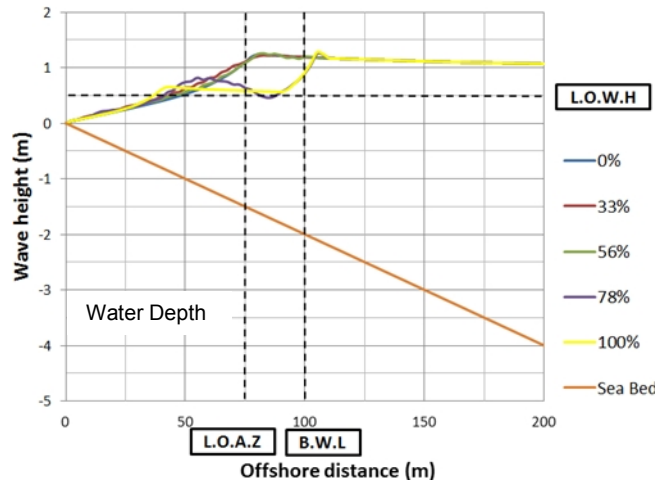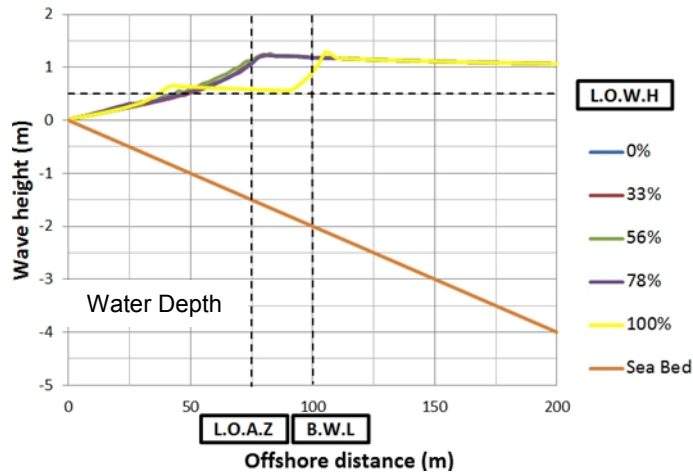
a) Wave vectors distribution of Case 3 (78% blocking)
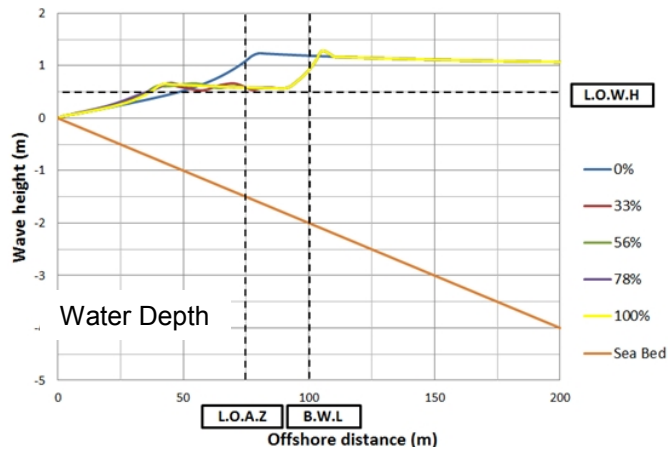
b) Wave height values for all cases with sea bed level at C.L
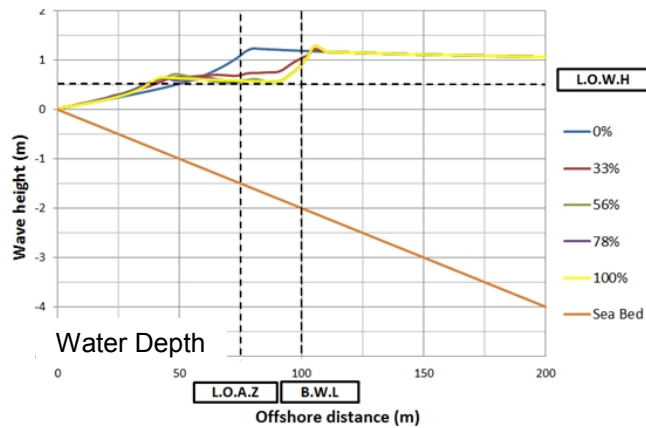
c) Wave height values for all cases with sea bed level at C.R 1

d) Wave height values for all cases with sea bed level at C.L 2

e) Wave height values for all cases with sea bed level at C.L 3

**Fig. 6. REF/DIF result and cross-shore profiles on wave field for Case 2
(56% blocking)**
*L.O.W.H is the Limit of Wave Height, L.O.A.Z is the Limit of Active Zone, B.W.L is the BreakWater
Location*

4.2 Wave Induced Currents

- In Case 0, offshore the wave breaker line the currents are very weak. Within the surf zone, the longshore currents reaches a maximum value of 0.22 m/s.
- In Case 1, at the east side of SLCDBWs, the generated longshore currents s are diverted to move to the east direction along the gap and parallel to the shoreline with a speed around 0.32 m/s just behind the breakwater head and within the used-zone. Such current speed is slightly higher than the safe swimming currents range (less than 0.3 m/s.) (Fig. 7a and 7b).

The second behavior was at the west side of SLCDBWs, where longshore currents are diverted to the west direction to interact with the longshore current from the east side of the previous SLCDBW. This interaction results in a clockwise current cell (eddy) turning around the western head center with an average speed of 0.32 m/sec. Part of current's eddy exists in the SLCDBW gap and used-zone. Finally, the generated currents are exceeding the safe swimming limit (to be 0.3 m/s.) (Fig. 7a and 7c).

Behind the SLCDBWs, the water conditions are safe for swimming (within acceptable values of safe swimming).

Moderate crossshore current with 0.2 m/s speed near the shoreline along the domain. (Fig. 7d and 7e).

- For Case 2, the results are similar to Case 1 with higher speed of the generated longshore current at the eastern side of SLCDBWs reaching 0.45 m/s. The size and extend of the current cell generated at the western head is larger and stronger with a speed reaching 0.45 m/s.

  For the protected area directly behind the SLCDBWs, the water conditions are calm (same currents speeds as in Case 1).

- For Case 3, current speeds are the worst. The generated current at the eastern head exceeds 0.5 m/sec. and interacts immediately with the current cell at the western head for the next SLCDBWs. The lee side of SLCDBWs are as Case 2.

- In Case 4 as the used-zone is totally protected, the dissipated wave energy is the highest. Thus the generated longshore is located exactly as in Case 0, but with lower speeds around 0.1 m/sec. Swimming in used-zone is totally safe (Fig. 8).

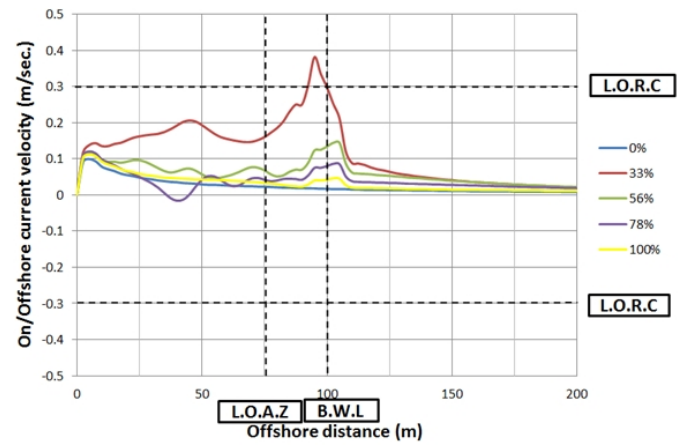 Model results are in a good agreement with currents pattern mentioned in [3].
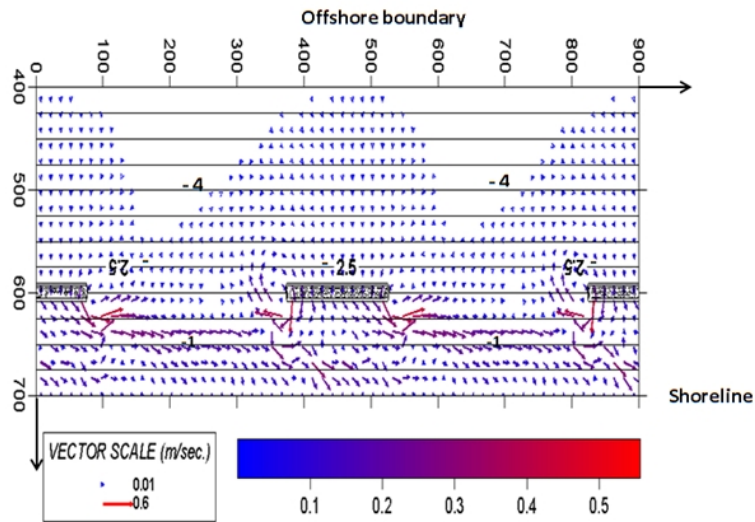
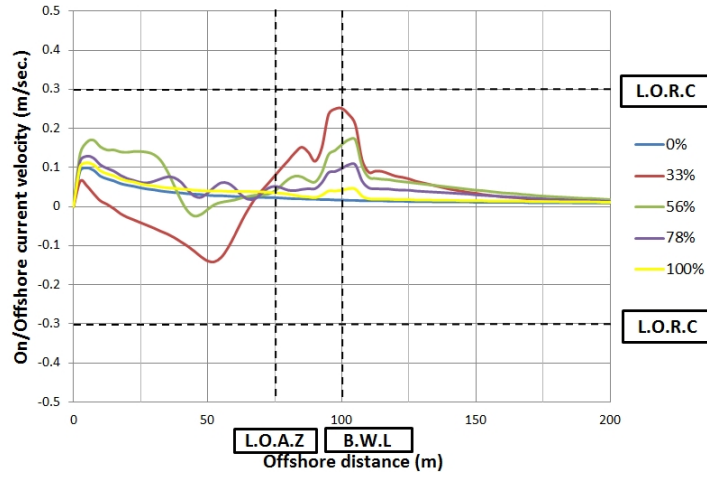**a) Velocity vectors distribution of Case 1 (33 % blocking).**
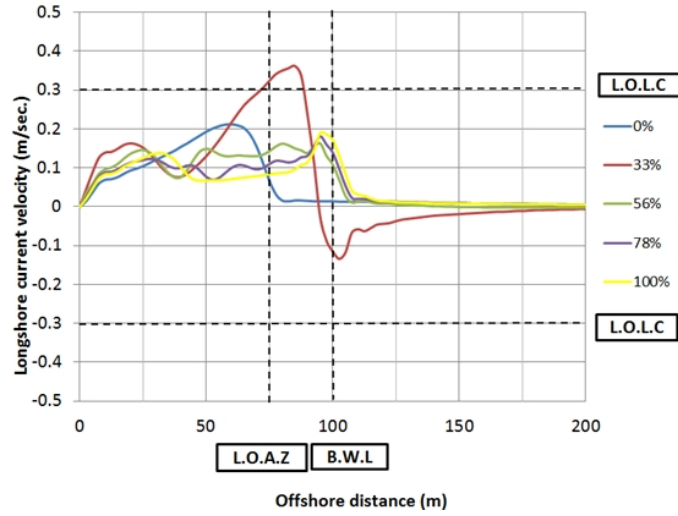
B) Longshore current velocity for all cases at C.R 1
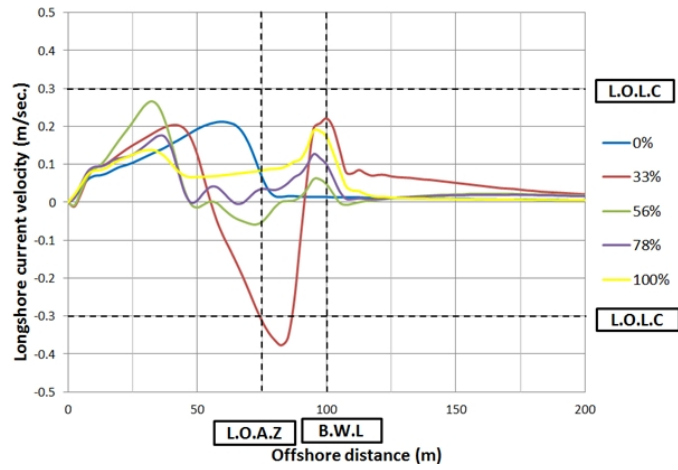
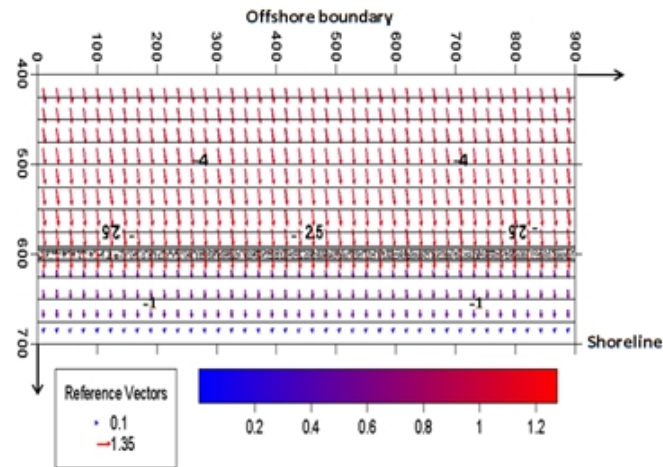**c) Longshore current velocity for all cases at C.L 1**

d) On/Offshore current velocity for all cases at C.R 1

e) On/Offshore current velocity for all cases at C.L 1

Fig. 7. SHORECIRC result and cross-shore profiles on currents' fields for Case 1 (33% blocking).
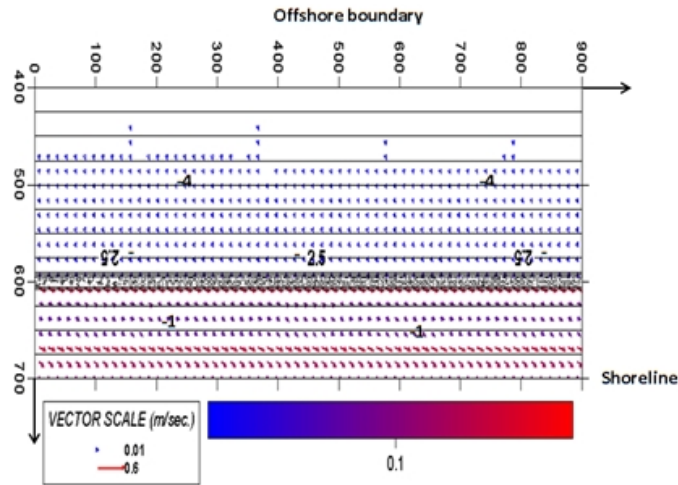*L.O.L.C is the Limit of Longshore Current, L.O.R.C is the Limit of Rip Current*

Fig. 8. Wave and currents fields for Case 4 (100% blocking) respectively.

4.3 Safe Swimming Areas

Fig. 9 shows the limits of generated safe swimming area within used-zone as follows:

- Case 1 with B/G=0.5 generates protected length for swimming behind SLCDBW of about 28% from the total breakwater length.
- Case 2 with B/G=1.25 generates protected length for swimming behind SLCDBW of about 60% from the total breakwater length.
- Case 3 with B/G=3.5 generates protected length for swimming behind SLCDBW of about 67% from the total breakwater length.
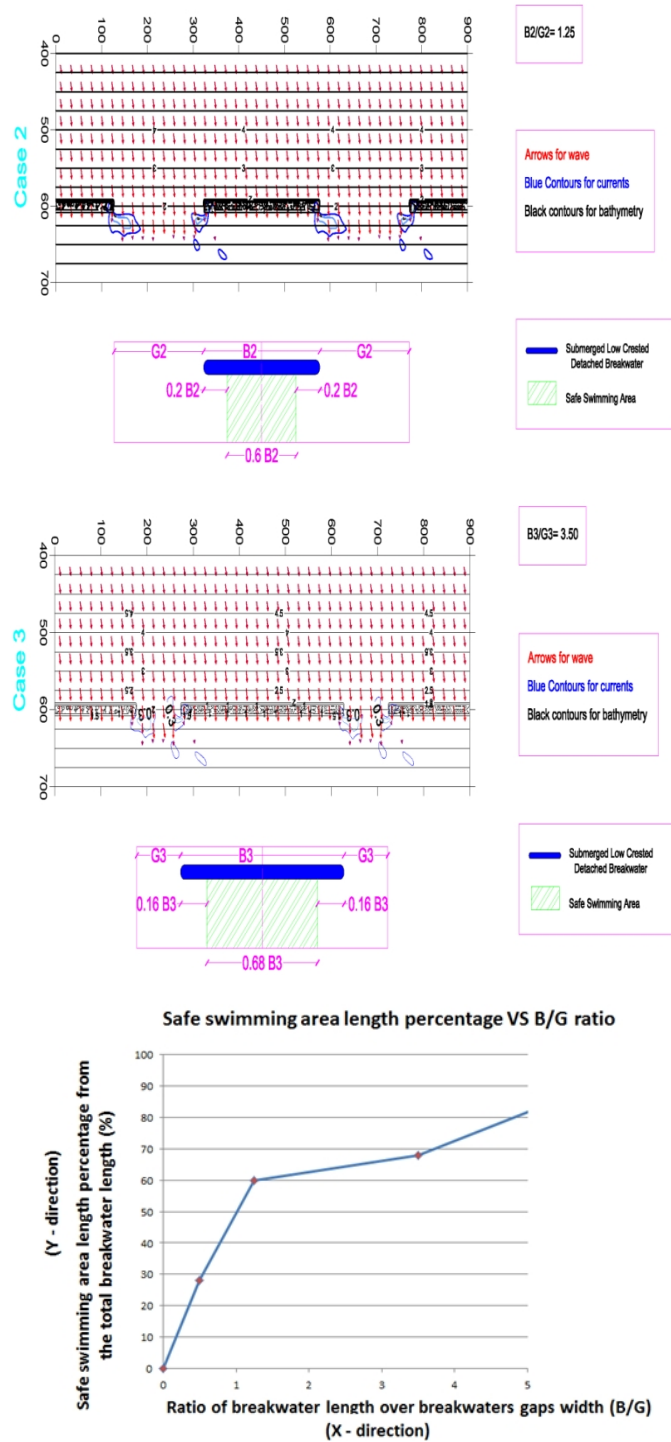 Note that Centerline of swimming area is the same centerline of breakwaters.
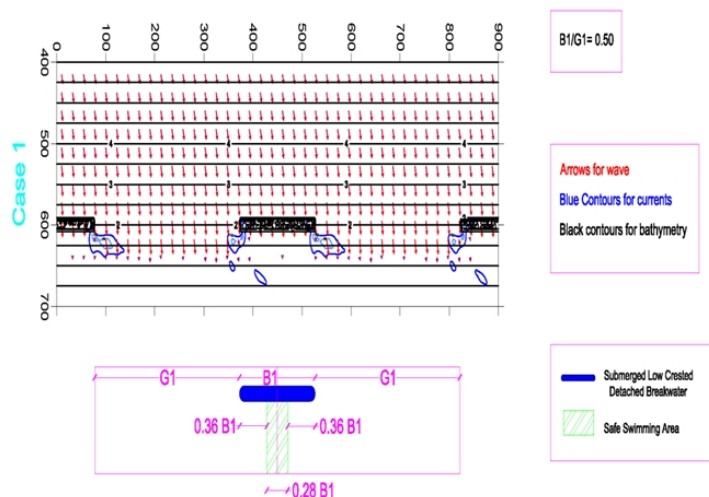
Fig. 9. Length of safe swimming zone percentage from the SLCDBW total length of Cases 1, 2 and 3 and chart showing such values.

4.4 Shoreline Changes Model

The results show that using SLCDBWs for protection will result in formation of salient's behind the breakwater and erosion down drift the SLCDBW. As the blocking percentage increases the formed salient dimensions increase and extend of erosion increases. Such impact will harm the neighboring beaches in the downdrift in addition to the loss of part of the swimming area due to the salient. Salient's and erosion dimensions (length and width) can be expressed as factors of breakwater distance from shoreline (Fig. 10).

Model results show fair agreement with shoreline line changes described in [3].
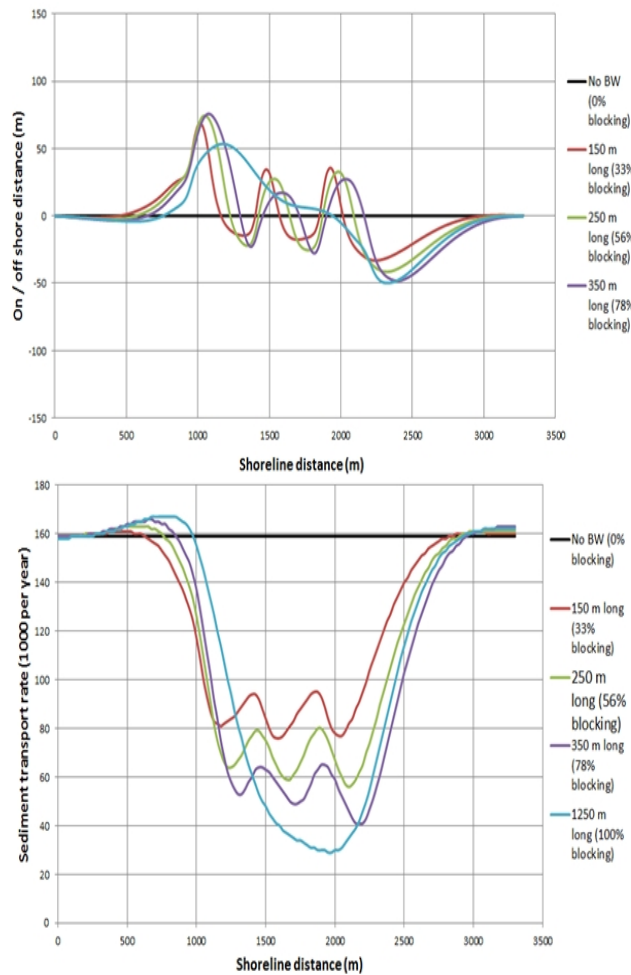
Fig. 10. Shoreline changes and sediment transport rates after 1 year for all proposed cases of protections respectively.

5. CONCLUSIONS

This paper provided a study on the use of SLCDBWs to produce suitable swimming areas along the North coast of Egypt. Three numerical models were used in this study, the

refraction diffraction model for wave transformation, the water body circulation model for hydrodynamics and the GENESIS one line shore changes model for shoreline changes.

Usage of SLCDBWs provides water exchange with no sea view obstruction. At the same time, SLCDBWs controls nearshore hydrodynamic fields.

The model results showed that in order to produce a sufficient area suitable for swimming, the SLCDBW should extend over the full length of the resort. The negative impact however on the downdrift beaches will require a detailed study to mitigate such effects.

COMPETING INTERESTS

Authors have declared that no competing interests exist.

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