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# Modelling of climate change impacts on coastal flooding/erosion, ports and coastal defence structures

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ABSTRACT

In the present work, a numerical model for the redesign of the existing and future ports and coastal defence structures is presented. The model is mainly based on the higher order Boussinesq equations and describes nonlinear wave transformation in the surf and swash zone, wave structure interaction, breaking wave induced currents and morphological changes. The existing model is adapted to describe climate change impacts on coastal flooding/erosion, ports and coastal defence structures. Model applications result to the estimation of: wave overtopping over the breakwaters crests, wave entering the harbour basin through diffraction, coastal erosion and storm surge/wave flooding. The model provides coastal engineers with a useful tool for the redesign of the existing coastal structures.

*Keywords:* Wave modelling; Climate change; Coastal structures; Coastal erosion

## 1. Introduction

The climate change is expressed on the open sea and the coastal zone through a number of impacts, such as: sea level rise, increase of the frequency of extreme wind events, change of the annual frequency of winds, more frequent storm surge events, higher waves, changes of the dominant wave direction, stronger currents on the coastal zone, etc. The above impacts, mainly due to a new sea level design and higher wave attacks, induce morphodynamic responses such as beach and dune erosion, inundation on low-lying areas leading to increased flooding risk of the coastal zone. In addition, the impacts distract the operational aims of ports and coastal defence structures.

Due to sea level rise and extreme meteorological events, the coastal structures will be exposed to larger waves which in turn will lead to greater overtopping and transmission, and greater penetration into a harbour. The problem will become more significant in shallow waters where the depth imposes the maximal amplitude because of wave breaking. Hence, the design, functionality and safety of such structures have to be re-evaluated under climate change. Wave overtopping is usually estimated by empirical and semi-empirical simplified formulas that can be implemented under certain and defined constraints. A better approach could be based on advanced numerical models of non-linear wave propagation based on the Boussinesq equations [1]. Another effect of sea level rise and extreme

*Presented at the 1st EWaS-MED International Conference on Improving Efficiency of Water Systems in a Changing Natural and Financial Environment, 11–13 April 2013, Thessaloniki, Greece* meteorological events is the increased wave run-up and flooding. To estimate this quantity, simplified empirical and semi-empirical formulas are used that can be implemented under certain and defined constraints. A better approach could be based on advanced numerical models of non-linear wave propagation [2]. The 2DH version of the above non-linear models [3] can be used for the estimation of the wave penetration into a harbour which is expected to increase due to climate effects (higher waves, changes the dominant wave direction, etc.).

Finally the response of a coastal area to future storm events is usually evaluated by means of a morphodynamical model [2,4]. The model can be used to estimate coastal erosion caused by extreme marine events and therefore it can significantly contribute to calculating potential erosion of a coastal area.

In the present paper advanced numerical models will be presented for the modelling of climate change impacts on coastal flooding/erosion, ports and coastal defence structures, aiming to contribute to the advancement of knowledge regarding the criteria, the methods and the goals of designing and upgrading coastal structures against coastal flooding and erosion under the prospect of climate change.

## 2. Coastal flooding model

The coastal flooding model consists of a storm surge and a near-shore wave propagation model. The first model estimates sea level rise due to wind effects and the second one estimates wave run-up on beaches.

### 2.1. Storm surge model

The storm surge simulation model is based on the depth-averaged wind induced circulation equations. The equations are written [5]:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial (\mathrm{Uh})}{\partial x} + \frac{\partial (\mathrm{Vh})}{\partial y} = 0$$

$$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + V\frac{\partial U}{\partial y} + g\frac{\partial \zeta}{\partial x} = \frac{1}{h}\frac{\partial}{\partial x}\left(v_h h\frac{\partial U}{\partial x}\right) + \frac{1}{h}\frac{\partial}{\partial y}\left(v_h h\frac{\partial U}{\partial y}\right) + fV + \frac{\tau_{sx}}{\rho h} - \frac{\tau_{bx}}{\rho h}$$

$$\frac{\partial V}{\partial t} + U\frac{\partial V}{\partial x} + V\frac{\partial V}{\partial y} + g\frac{\partial \zeta}{\partial y} = \frac{1}{h}\frac{\partial}{\partial x}\left(v_h h\frac{\partial V}{\partial x}\right) + \frac{1}{h}\frac{\partial}{\partial y}\left(v_h h\frac{\partial V}{\partial y}\right) - fU + \frac{\tau_{sy}}{\rho h} - \frac{\tau_{by}}{\rho h} \tag{1}$$

where $\zeta$ is the water surface elevation above the mean water level; $d$ is the water depth; $h$ is the total depth of water (i.e. $h = d + \zeta$); $U$, $V$ are the depth-averaged velocity components in $x$ and $y$ directions, respectively; $f$ is the Coriolis parameter; $v_{\mathrm{h}}$ is the horizontal eddy viscosity coefficient, $\tau_{\mathrm{sx}}$ and $\tau_{\mathrm{sy}}$ are the shear stresses at the water surface in the $x$ and $y$ directions, respectively, which represent the vertical boundary condition as follows:

$$\tau_{sx} = \rho k W_x\sqrt{W_x^2 + W_y^2} \quad \tau_{sy} = \rho k W_y\sqrt{W_x^2 + W_y^2} \tag{2}$$

where $k$ is the surface friction coefficient (kg/m$^3$) typically of the order of $10^{-6}$ (here we assume $k = 0.000001/0.000003$); $W_x$ and $W_y$ are the wind speeds in $x$ and $y$ directions (m/s), respectively.

Similarly, the bed friction terms ($\tau_{bx}$, $\tau_{by}$) are expressed by quadratic forms [6].

The horizontal eddy viscosity coefficient is given by the well-known Smagorinsky model [7].

Differential Eq. (1) are approximated by finite difference equations according to the explicit scheme developed by [5].

The total reflection boundary condition, i.e. $U = 0$, $V = 0$ $\partial\zeta/\partial n = 0$ (where $n$ is the unit vector normal to the boundary), is incorporated in the model at the land boundaries.

Steady state is achieved when the ratio between the difference in kinetic energy between time steps falls less than a certain accuracy (typically $1 \times 10^{-4}$).

Fig. 1 shows the simulation of a storm surge event during winder 2010 in the Aegean Sea. South winds with speed 24 m/s (7–8 Beaufort) induced sea level rise of order of 0.5 m at the coasts of Samos and Lesbos islands. A similar value of the storm surge is predicted by the model. The climate change effect was not the intensity of the winds, but the duration of them (more than 4 d).

### 2.2. Near-shore wave propagation model

Near-shore wave propagation is simulated by a Boussinesq nonlinear wave model. The following higher order Boussinesq-type equations for breaking and non-breaking waves are used [2,6]:

$$\zeta_t + \nabla(h\mathbf{U}) = 0 \tag{3}$$

$$\mathbf{U}_t + \frac{1}{h}\nabla\mathbf{M_u} - \frac{1}{h}\mathbf{U}\nabla(\mathbf{U}h) + g\nabla\zeta + G = \frac{1}{2}h\nabla[\nabla\cdot(d\mathbf{U}_t)] - \frac{1}{6}h^2\nabla[\nabla\cdot\mathbf{U}_t] + \frac{1}{30}d^2\nabla[\nabla\cdot(\mathbf{U}_t + g\nabla\zeta)] + \frac{1}{30}\nabla\left[\nabla\cdot\left(d^2\mathbf{U}_t + gd^2\nabla\zeta\right)\right] - d\nabla(\delta\nabla\cdot\mathbf{U})_t - \frac{\tau_b}{h} + \mathbf{E}$$

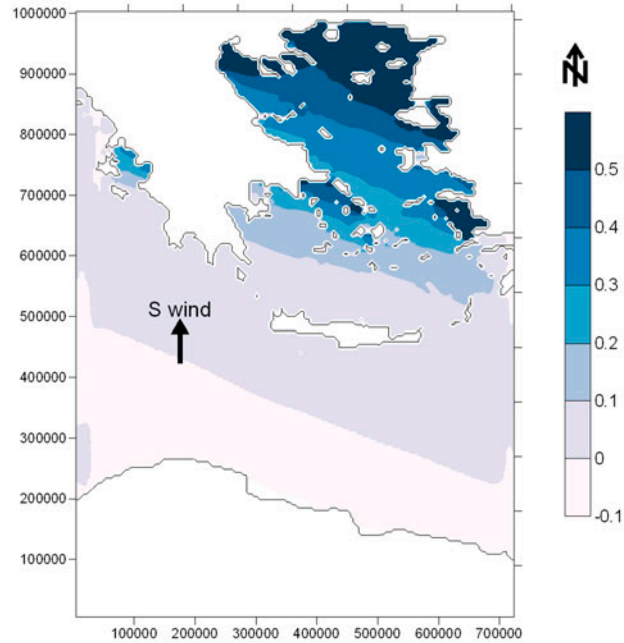


Fig. 1. Prediction of storm surge sea level rise during winder 2010 in the Aegean Sea.

where

$$G = \frac{1}{3}\nabla\left\{d^2\left[(\nabla\cdot\mathbf{U})^2 - \mathbf{U}\cdot\nabla^2\mathbf{U} - \frac{1}{10}\nabla^2(\mathbf{U}\cdot\mathbf{U})\right]\right\} - \frac{1}{2}\zeta\nabla[\nabla\cdot(d\mathbf{U}_t)]$$

where subscript $t$ denotes differentiation with respect to time; $d$ = still water depth; $U$ = horizontal velocity vector, $\mathbf{U} = (U,V)$, where $U$ and $V$ = depth-averaged horizontal velocities in directions $x$ and $y$; $\zeta$ = surface elevation, $h$ = total depth $h = d + \zeta$, $g$ = gravitational acceleration, $\tau_b = (\tau_{bx}, \tau_{by})$ = bottom friction term; $\delta$ = roller thickness (determined geometrically according to Schäffer et al. [8]), $\mathbf{E}$ = eddy viscosity term; and $\mathbf{M}_u = (d+\zeta)\ \mathbf{u_o}^2 + \delta(c^2 - \mathbf{u_o}^2)$, in which $\mathbf{u_o} = (u_o, v_o)$ is the bottom velocity.

In the swash zone, the "dry bed" boundary condition is used to simulate run-up. The condition, at a grid point $(i,j)$, is written [9]:

if $(d+\zeta)_{i,j} > h_{cr}$ and $(d+\zeta)_{i-1,j} \le h_{cr}$ and $U_{i,j} > 0$ then $U_{i,j} = 0$

if $(d+\zeta)_{i,j} > h_{cr}$ and $(d+\zeta)_{i,j-1} \le h_{cr}$ and $V_{i,j} > 0$ then $V_{i,j} = 0$

if $(d+\zeta)_{i,j} \le h_{cr}$ and $(d+\zeta)_{i-1,j} \le h_{cr}$ then $U_{i,j} = 0$

if $(d+\zeta)_{i,j} \le h_{cr}$ and $(d+\zeta)_{i,j-1} \le h_{cr}$ then $V_{i,j} = 0$

if $(d+\zeta)_{i,j} \le h_{cr}$ and $(d+\zeta)_{i-1,j} > h_{cr}$ and $U_{i,j} < 0$ then $U_{i,j} = 0$

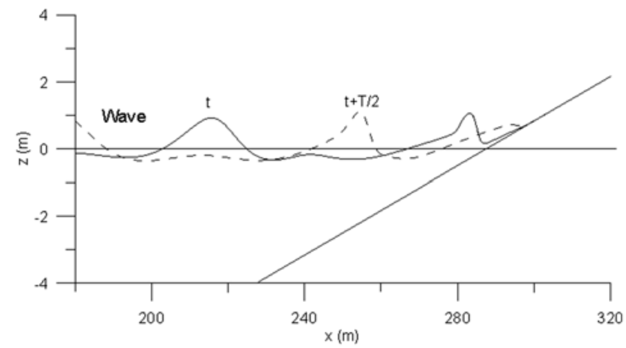


Fig. 2. Simulation of coastal flooding due to wave run-up.

if $(d+\zeta)_{i,j} \le h_{cr}$ and $(d+\zeta)_{i,j-1} > h_{cr}$ and $V_{i,j} < 0$ then $V_{i,j} = 0$ (5)

where $h_{cr}$ is a very small depth below which drying is assumed to occur. Here, we consider $h_{cr} = 0.00001$ m.

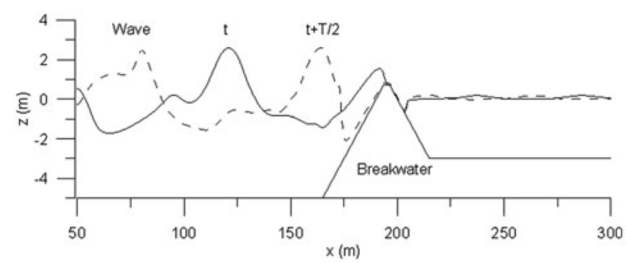


Fig. 3. Simulation of wave run-up and overtopping.

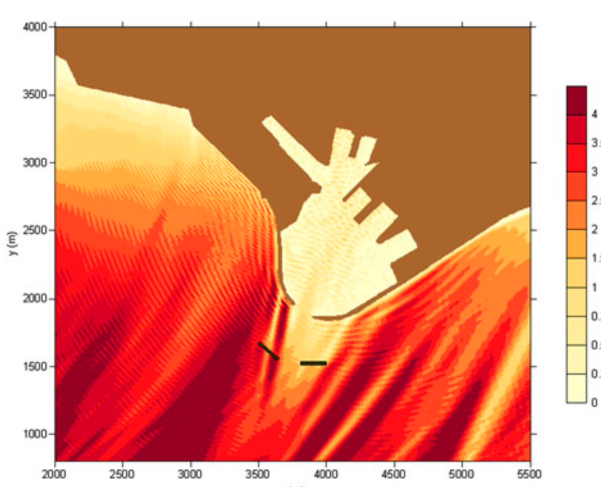


Fig. 4. Wave height distribution in a harbour where two floating breakwaters are used near the entrance to reduce wave penetration.

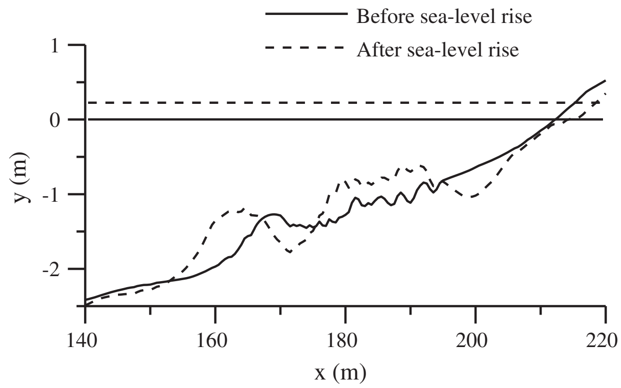


Fig. 5. Sea level rise effects on cross-shore morphology evolution.

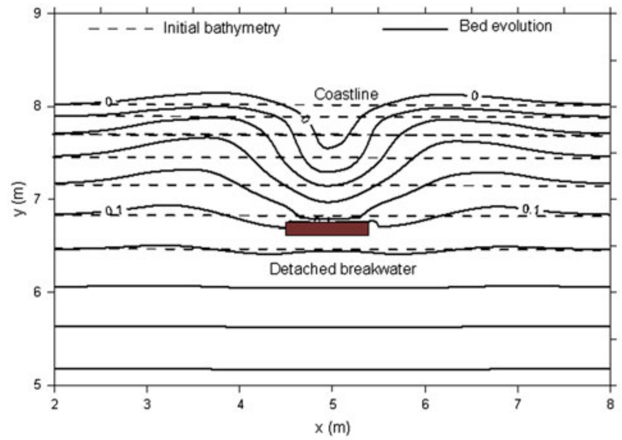


Fig. 6. Salient formation behind a detached breakwater for coastal protection.

The governing equations are finite-differenced utilizing a high-order predictor-corrector scheme that employs a third-order explicit Adams–Bashforth predictor step and a fourth-order implicit Adams–Moulton corrector step [10]. The corrector step is iterated until the desirable convergence is achieved. First-order spatial derivatives are discretized to fourth-order accuracy.

Fig. 2 shows the simulation of inundation induced by storm waves (i.e. flooding due to wave setup and run-up). The initial water depth can be taken from the storm surge model, and consequently the total sea level rise, due to synergy of waves and storm surge, can be obtained.

## 3. Wave–coastal structure interaction model

The above nonlinear wave propagation model is adapted to simulate wave overtopping. Eq. (5) together with that proposed by Lynett et al. [1] moving boundary scheme are applied for the simulation of wave run-up and overtopping over coastal structures. An overtopping example is presented in Fig. 3. Using the proposed numerical model, the coastal engineer can re-design coastal structures aiming to reduce the overtopping (by increasing crest level or/and offshore slope or by constructing a berm/submerged breakwater offshore the coastal structure).

Fig. 7. Sea wall failure due to extreme sea level rise in Eressos coast, Lesbos, Greece.


Fig. 8. Aegean Sea, Lesbos Island and case study area.

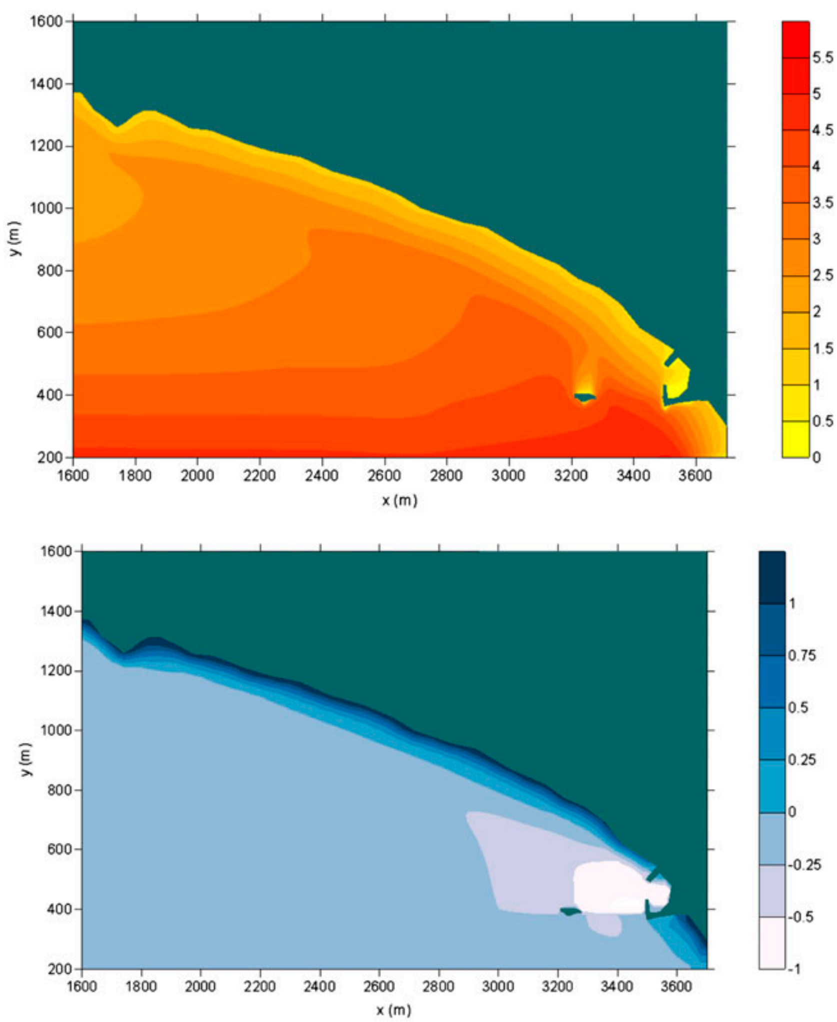


Fig. 9. Wave height distribution (top) and sea level rise due to breaking waves (bottom) in the coast of Eressos, Lesbos.

## 4. Model for wave penetration into a harbour

The near-shore wave propagation model, after the incorporation of appropriate conditions for wave–structure interactions, can be used for the re-design of ports and harbours under the future conditions (higher waves, changes of the dominant wave direction, etc.). The model is adapted to incorporate wave–breakwater interactions [3,11], wave propagation over submerged breakwaters [12] and wave floating breakwater interactions [13]. In this way the model becomes able to estimate wave penetration into a harbour and can be used for the re-design of harbour layout. In Fig. 4 the wave height distribution into a harbour is shown. In order to reduce wave penetration into the harbour two floating breakwaters are designed to be constructed near the entrance.

## 5. Coastal erosion model

The wave model of Section 2.2, describes nonlinear wave transformation in the surf and swash zone. The model is also able to describe breaking wave-induced current (simply after time integration of the horizontal velocities). By incorporating sediment transport formulae [4] the model can be applied to simulate beach morphology evolution. In Fig. 5 the model is applied to simulate the effects of future sea level rise on cross-shore beach morphology evolution. The sea level rise is supposed to be equal to 0.25 m. The wave height is taken equal to $H_o = 1.20$ m and the period $T = 5$ s. The sand used was well graded with a median diameter of $d_{50} = 0.3$ mm and the theoretical Bruun equilibrium profile was formed as initial profile. The estimated shoreline recession of the waterline is predicted to be 7.5 m.

In [4,14] the model is adapted to incorporate the effects of coastal protection structures, aiming to contribute to a proper design of them. In Fig. 6 the model is applied to design a detached breakwater for coastal protection (reproduction of Ming and Chiew [15] experiments, breakwater length 0.9 m, distance from the initial shoreline 1.2 m).

## 6. A case study

The above models are applied to re-design a seawall failure (Fig. 7) during the winter of 2010 in the

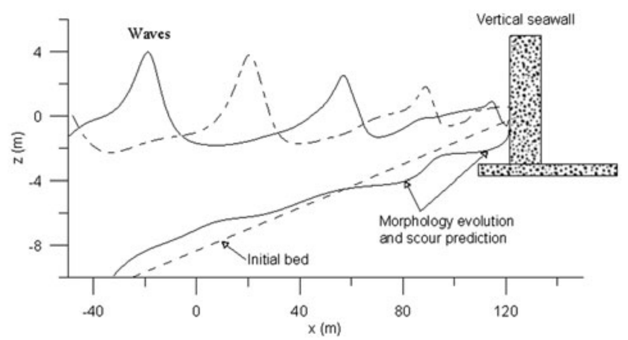


Fig. 10. Bed morphology evolution and maximum scour in front of the sea wall.

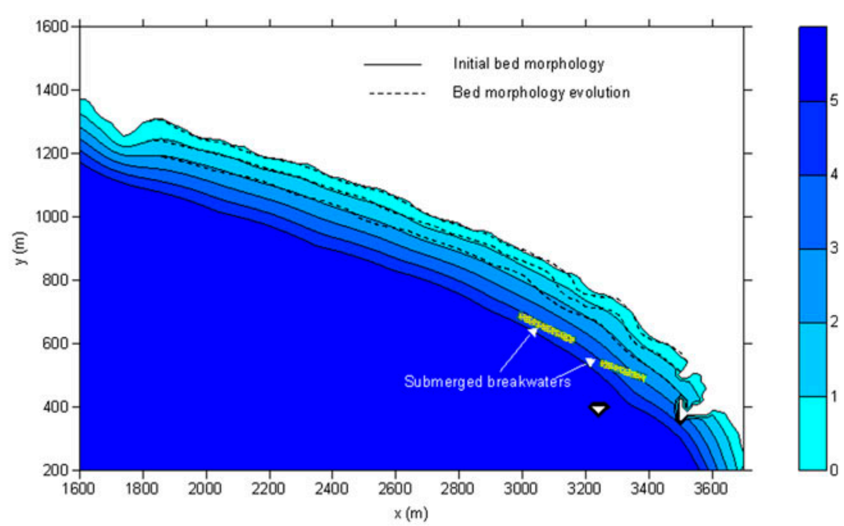


Fig. 11. Prediction of the bed morphology evolution after the construction of two submerged breakwaters for coastal protection.

coast of Eressos, Lesbos Island, Greece (Fig. 8). The failure of the seawall occurred due to wave attack in combination with the extreme storm surge sea level rise. In Fig. 9 wave height distribution and sea level rise due to breaking waves in the Eressos coast, are shown. The unfavourable wave SW direction is considered for the re-design of the seawall. The results are obtained by applying the wave model presented in Section 2.2. After applying the sediment transport and bed morphology model we are able to predict the maximum scour in front of the sea wall and design the depth of the toe of the new structure (Fig. 10). Also, another alternative should be considered: nourishment of the coast and construction of two submerged breakwaters for further protection against erosion. In Fig. 11, the prediction of the coastline evolution after the construction of two submerged breakwaters is shown. It is obvious that the breakwaters provide adequate protection to the nourished coast.

## 7. Conclusions

Due to sea level rise and extreme meteorological events, the coastal structures will be exposed to larger waves which in turn will lead to greater overtopping and transmission, and greater penetration into a harbour [16]. In addition other impacts such as the change of the annual frequency of winds, the more frequent storm surge events, the changes of the dominant wave direction etc. lead to morphodynamic changes in the coastal zone such as beach and dune erosion, inundation on low-lying areas leading to increased flooding risk of the coastal zone. Consequently the design, functionality and safety of the coastal structures have to be re-evaluated [16].

Civil Engineers need the help of advanced mathematical models, in order to confront the above problems in coastal zone. Numerical models for wind-induced circulation and near-shore wave propagation can be extended/adapted to simulate the above mentioned climate change impacts on coastal flooding/erosion, ports and coastal defence structures. The models, through the simulation of wave overtopping over the breakwaters crests and the wave entering the harbour basin through diffraction can provide Coastal Engineers the tools to harbour layout re-design and to design new structures for additional protection. Also coastal erosion and storm surge/wave flooding can be confronted by designing new coastal defence structures, again with the use of numerical models.

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## References

[1] P.J. Lynett, J.A. Melby, D.H. Kim, An application of Boussinesq modeling to hurricane wave overtopping and inundation, Ocean Eng. 37 (2010) 135–153.
[2] Th.V. Karambas, C. Koutitas, Surf and swash zone morphology evolution induced by nonlinear waves, J. Waterw. Port Coast. Ocean Eng. 128(3) (2002) 102–113.
[3] D.R. Fuhrman, H.B. Bingham, P.A. Madsen, Nonlinear wave–structure interactions with a high-order Boussinesq model, Coastal Eng. 52(8) (2005) 655–672.
[4] Th.V. Karambas, Design of detached breakwaters for coastal protection: Development and application of an advanced numerical model, International Conference on Coastal Engineering, Santander, Spain, 2012.
[5] C.G. Koutitas, Mathematical Models in Coastal Engineering, Pentech Press Limited, London, 1988.
[6] Th.V. Karambas, E.K. Karathanassi, Longshore sediment transport by nonlinear waves and currents, J. Waterw. Port Coast. Ocean Eng. 130(6) (2004) 277–286.
[7] P.A. Madsen, M. Rugbjerg, I.R. Warren, Subgrid modelling in depth integrated flows, In: International Conference on Coastal Engineering, Torremolinos, 1988, pp. 505–511.
[8] H.A. Schäffer, P.A. Madsen, R. Deigaard, A Boussinesq model for waves breaking in shallow water, Coastal Eng. 20 (1993) 185–202.
[9] A. Militello, C.W. Reed, A.K. Zundel, N.C. Kraus, Two-dimensional depth-averaged circulation model M2D: Version 2.0, Report 1 US Army Corps of Engineers, ERDC/CHL TR-04-2, 2004
[10] G. Wei, J.T. Kirby, Time-dependent numerical code for extended Boussinesq equations, J. Waterw. Port Coast. Ocean Eng. 121(5) (1995) 251–261
[11] Th.V. Karambas, E.C. Bowers, Representation of partial wave reflection and transmission for rubble mound coastal structures, Hydrosoft 96, Malaysia, 1996.
[12] E.E. Kriezi, Th.V. Karambas, Modelling wave deformation due to submerged breakwaters. Proc. ICE Maritime Eng. 163(1) (2010) 19–29.
[13] E.V. Koutandos, Th.V. Karambas, C.G. Koutitas, Floating breakwater response to waves action using a Boussinesq model coupled with a 2DV elliptic solver, J. Waterw. Port Coast. Ocean Eng. 130(5) (2004) 243–255.
[14] Th.V. Karambas, Ch. Koftis, E. Koutandos, P. Prinos, Innovative submerged structures/vegetation effects on

coastal erosion: Numerical modeling of hydro-morphological processes, Proceedings of the Twenty-second International Offshore and Polar Engineering Conference, Rhodes, Greece, 2012.
[15] D. Ming, Y.-M. Chiew, Shoreline changes behind detached breakwater, J. Waterw. Port Coast. Ocean Eng. 126(2) (2000) 63–70.
[16] Th.V. Karambas, Modeling of climate change impacts on coastal flooding/erosion, ports and coastal defence structures, 1st EWaS-MED International Conference, Improving Efficiency of Water Systems in a Changing Natural and Financial Environment, Thessaloniki, Greece, 2013.