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COASTAL SUBMERGED STRUCTURES ADAPTATION TO SEA LEVEL RISE OVER DIFFERENT BEACH PROFILES

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EXTENDED ABSTRACT

La presente memoria tratta gli effetti che l'innalzamento del livello del mare, dovuto ai cambiamenti climatici e ai sovralzi ondosi in condizioni di mareggiata, induce sulla stabilità delle spiagge di sedimenti naturali o artificiali (provenienti da interventi di ripascimento) difese da strutture sommerse. Secondo quanto evidenziato nel Rapporto Speciale IPCC 2019 (Brown *et alii*, 2018), i cambiamenti climatici provocheranno un aumento del livello medio marino. Questo fenomeno, combinato con il previsto incremento dell'intensità e della frequenza di accadimento delle mareggiate estreme, avrà delle ripercussioni negative sulla conservazione delle spiagge, soprattutto quelle difese da ripascimento o da strutture sommerse. L'adeguamento delle opere di protezione costiera esistenti diventa dunque indispensabile per contenere le perdite di sedimenti verso il largo e la conseguente erosione delle spiagge. I possibili effetti indotti sulle spiagge sono stati analizzati per diverse condizioni ambientali e per diversi scenari di sovralzo, con lo scopo di valutare quali interventi di adattamento su un'esistente opera di difesa sommersa potrebbero essere intrapresi per garantire la resilienza della spiaggia. L'innalzamento del livello medio marino, combinato con le condizioni di attacco ondoso delle mareggiate, produce, con l'allagamento della spiaggia emersa, un arretramento della linea di riva che viene qui stimato tramite un metodo basato sull'adattamento del profilo di equilibrio in condizioni di trasporto trasversale. Tale metodo, proposto da Dean (1991), è stato qui generalizzato al caso di spiagge con diversi profili del fondo ed è stato esteso il campo di applicazione mettendo in conto l'effetto indotto da una struttura sommersa. La presenza dell'opera produce lo smorzamento dell'energia ondosa trasmessa ed un ulteriore aumento del livello marino dovuto al piling-up. Sia l'ampliamento della berma di sommità della struttura che la riduzione della sua sommergenza, ottenuta innalzando la quota di sommità dell'opera, hanno effetti benefici nel contrastare l'erosione della spiaggia. In particolare, sono messi a confronto due diversi tipi di intervento, ottenuti impiegando lo stesso volume di materiale: nella prima configurazione la struttura di riferimento viene alzata dalla quota di - 2.0 m fino alla quota di - 0.5 m sotto il livello medio del mare, mantenendo invariata la larghezza della berma di sommità; viceversa, nella seconda configurazione, la sommergenza rimane invariata rispetto alla struttura iniziale ma la berma viene allargata da 8 m fino a 24 m. I risultati mostrano che la soluzione migliore di adattamento della struttura dipende, a parità di altri fattori, dall'entità del sovralzo. Per piccole variazioni di livello, risulta essere più efficace l'allargamento della berma, mentre, per incrementi particolarmente significativi, l'innalzamento della struttura è la procedura più vantaggiosa. Lo scopo del presente studio è quello di fornire indicazioni utili per il progetto di adeguamento di strutture sommerse esistenti, andando ad individuare, per ogni condizione ambientale, il miglior intervento di adattamento della struttura in grado di produrre il minor arretramento della spiaggia.

ABSTRACT

The paper focuses on the analysis of the sea level rise and storms effects on natural or nourished beach profiles protected by submerged breakwaters. The increase in terms of intensity and frequency of extreme sea storms, and water levels produced by climate change, could lead to a deviation from the original trend of the beach. Typical Adriatic beaches will be considered as realistic study cases and a submerged structure for coastal protection of a natural or artificial (nourishment) beach is analysed in order to identify its resilience and its design adaptation. Different scenarios are taken into account, according to the 2019 special report of IPCC on climate change. A numerical model is here provided to evaluate beach response to wave set-up and sea level rise in the case of any beach profile protected by submerged breakwaters. The present model can provide useful information for the adaptation design of an existing defence submerged breakwater against coastal flooding and beach erosion. The shoreline position change, Δy, is evaluated by a large number of simulations varying both wave parameters (wave height H_i, wave length L) and the geometry of breakwater (submergence $|R_c|$ berm width B, structure height h_c). Different scenarios of sea level rises, as a consequence of climate change, and of storm conditions are considered in performing numerical simulations.

KEYWORDS: submerged structure adaptation, sea level rise, equilibrium profile, climate change, storm surge.

INTRODUCTION

Beach erosion is one of the most important and challenging problems in the management of coastal zones.

One of the main process that shapes shorelines is the variation of the water level as a consequence of different factors, such as storm surges, sea level rise due to global warming, subsidence and tides. In addition to these, other factors are the wave action, longshore gradients, onshore-offshore sand transports, sand sources and sinks.

Nearshore sediment movement is generally divided into two components, longshore and cross-shore transport. It may be considered appropriate to neglect longshore transport for beaches far from structures, inlets and river mouths. The transversal sediment transport becomes prevailing also when longshore fluxes are balanced or in the case of a milder beach, typical along the Italian Adriatic coast, which induces the rotation of incident wave fronts until they become almost parallel to the shoreline.

During the years several cross-shore models have been developed in order to evaluate beach profile evolution under different hydrodynamic conditions. The first method relating shoreline retreat to water level rise is the geometric approach of Bruun (1954, 1962, 1988). It assumes that the active portion of an offshore profile rises with rising sea level and moves landward to offset the loss of sand. Dean (1991) applied this approach, including the effect of wave-induced set-up, to the equilibrium profile (Dean, 1977) obtained by assuming uniform energy dissipation per unit volume in the surf zone. Kriebel *et alii* (1991) evaluated the profile recession due to a water level rise for different geometrical cases, e.g., profile with linear sloping beach face and profile with dune. More recently, Dean & Houston (2016) modified the Bruun rule with terms representing all phenomena affecting shoreline change including onshore sand transport, sand sources and sinks and longshore transport gradients.

Over the years, cross-shore numerical models, such as SBeach (Larson & Krauss), XBeach (Roelvink *et alii*, 2009) and CSHORE (Kobayashi, 2016), have been developed as useful tools to predict beach response to storm events. These models allow to consider the hydrodynamic processes of short and long wave transformation, wave set-up and unsteady currents and the morphodynamics due to bed load and suspended sediment transport. On the contrary, they request high computational time and the calibration of the involved parameters with field data, often not available.

Various measures have been attempted to protect beaches. Coastal structures can be divided into hard structures, like seawalls, revetments, offshore breakwaters and groins; and soft structures such as beach nourishments and submerged berms. Construction of submerged breakwaters, with natural rocks or geotextile sand containers, has intensified in recent years and, nowadays, there are several examples of this protection system along the northern and central Italian coast of the Adriatic Sea. Depending on the structure freeboard, R_c, they can act both as breakwaters, causing damping of the waves, and as sills, preventing natural or nourishment material to move seaward during storms; being $|R_c|$ in the range of 0.50 - 1.00 m under the mean water level in the first case, and increasing up to 2.00-3.00 m in the second case.

In 2019 Special Report of IPCC (Brown *et alii*, 2018) different scenarios of sea level rise are presented depending on the mean increase of air temperature. According to the report, the mean sea level could rise up to 78 cm in 2100, while the minimum attended increase is 40 cm in the best mitigation scenario. This effect, in concert with the expected increase in intensity and frequency of extreme storm events, could cause shoreline recession on both free and protected beaches. Thus, a Low Crested Structure (LCS) could become less effective due to climate changes (higher sea level and waves) and adaptation interventions could be required.

The present paper aims on the analysis of the sea level rise and storms effects on beaches protected by submerged breakwaters. The influence of the sea level rise on the shoreline recession is analysed over different profiles: equilibrium profile, linear profile and two measured natural profiles as realistic study cases of central Italian Adriatic coast. A numerical model for the prediction of the shoreline erosion due to the transversal sediment transport is here proposed. Such model is an extension of that proposed by Dean (1991) for the evaluation of beach recession in an equilibrium profile subjected to both wave set-up and sea level rise. The present model allows to evaluate the response of any beach profile type to the increase of the water level and includes the effects induced by the presence of a submerged structure: i) reduction of the incident wave height and ii) increase of the water level with its associated piling up. Submerged structures induce locally currents due to the wave-structure interaction. Indeed, waves passing over a LCS result in a net transport of water across the structure inducing a higher mean water level in the lee of the structure. This level rise is balanced mainly by outgoing currents at the gaps between contiguous barriers and at the head of the structure system, inducing vortices in the lee zone (Soldini *et alii*, 2005; Lorenzoni *et alii*, 2009) which cannot be considered by a transversal model.

The shoreline position change is estimated in a large number of simulations by varying the wave parameters (wave height H) and the breakwater's geometry (freeboard R_c, berm width B, water depth at the structure toe h_e) and by considering different scenarios of sea level rise S, as a consequence of climate change and storms.

The purpose of the study is to provide useful information for the design of a submerged structure or for the adaptation of an existing one to more severe sea conditions, finding in each case the best intervention in terms of minimum beach recession.

ANALYTICAL MODEL

The earliest relationship between increased water level S and beach profile response was presented by Bruun (1962) and is known as the "Bruun rule". Bruun's method does not require any specific form of the profile but the new equilibrium profile relative to the increased water level must have the same form as the original one, hence, the profile shape does not change with respect to the water level. Assuming that the sand volume eroded is equal to the volume deposited, Bruun obtained:

$$\frac{\Delta y}{W_*} = -\frac{S}{h_* + Z} \tag{1}$$

Here, h_* and W_* represent, respectively, the depth of breaking and its distance from the shoreline, Z is the berm height of the shoreline and S is the variation in water level due to climate change and storm surge (see Fig. 1).

Dean (1991) modified the Bruun rule considering the effect of the set-up, $\overline{\eta}$, due to wave breaking over the specific equilibrium beach profile of the type $h(y) = A\, y^{2/3}$, where h is the water depth at a seaward distance, y, and A is a scale parameter which depends primarily on sediment characteristics.

An equilibrium beach profile represents a balance of destructive and constructive forces acting on the beach. Several approaches have been pursued in the attempt to characterize equilibrium beach profiles. The first equilibrium profile theory is based on the assumption that the turbulence in the surf zone, created by the breaking process, is the dominant destructive force. By neglecting the contributions of the non-linear interactions between waves and the beach reflection, the amount of turbulence is the amount of energy dissipated per unit water volume by the breaking waves. The uniform energy dissipation per unit volume within the breaking zone, D_*, written in terms of the energy conservation is:

$$\frac{1}{h}\frac{\partial\left(EC_G\right)}{\partial y} = D_*\left(d\right) \tag{2}$$

where E and C_G are the wave energy density and group velocity, respectively, and d is the sediment particle diameter. The wave energy flux is $F=EC_G=1/8\, \rho g \kappa^2\, h^2\, (gh)^{1/2}$ in which ρ is the water mass density, g is gravity acceleration and κ is the breaking index. Taking the derivative and simplifying, Eq. (2) leads to $D_*(d)=5/16\, \rho g^{(3/2)}\, \kappa^2\, h^{1/2}\, dh/dy$ which can be integrated for h obtaining:

$$h = \left(\frac{24 D_*(d)}{5\rho g\sqrt{g}k^2}\right)^{2/3} y^{2/3} = A(d)\, y^{2/3} \tag{3}$$

In Eq. (3), the dimensional parameter A is the profile scale factor and it is a function of the energy dissipation and of the sediment grain size of the beach.

Both the aforementioned approaches (Bruun and Dean) are based on the hypothesis that coastal erosion is only due to the transversal sediment transport.

To include the wave set-up $\overline{\eta}$ and the sea level variation S in the previous derivation of the equilibrium profile, Dean (1991) expressed in Eq. (2) the local water depth not by $h(y)$ but by $h(y) + S + \overline{\eta}\,(y)$, finding the following result:

$$\left(h + S + \overline{\eta}\right) = Ay^{2/3} \tag{4}$$

where the y-origin is now the location where $h + S + \overline{\eta} = 0$.

This equation for the equilibrium profile can be combined with the set-up equation across the surf zone to find the water depth $h\,(y)$ and $\overline{\eta}\,(y)$. The set-up in the surf zone is due to the transfer of momentum from the organized wave motion to the surf zone, and, according to the linear shallow water wave theory, it is described by the relationship of Bowen *et alii* (1968):

$$\overline{\eta}\left(y\right) = \overline{\eta}_b + J\left[h_b - h\left(y\right)\right] \tag{5}$$

in which h_b is the breaking depth ($h_b=h^*-\overline{\eta}_b-S$), $\overline{\eta}_b$ is the set-down (negative) at breaking and J is a constant involving the breaking index κ in the form $J=(3\kappa^2/8)/(1+3\kappa^2/8)$. At the breaker line the set-down reaches a maximum that can be approximated as $\overline{\eta}_b \approx -H_b/20$, where H_b is the wave height at the breaking point (Dean & Dalrymple, 2004).

The main characteristics of the phenomenon are represented in Fig. 1.

Following procedures similar to those used for evaluating recession with a uniform water level across the surf zone, the volume balance is:

$$\int_{\Delta y}^{0}\left[Z-S-\overline{\eta}(y)\right]dy+\int_{\Delta y}^{W_*+\Delta y}A\left(y-\Delta y\right)^{\frac{2}{3}}dy= \\ =\int_{0}^{W_*+\Delta y}Ay^{2/3}dy+\int_{0}^{W_*+\Delta y}\left[S+\overline{\eta}(y)\right]dy \tag{6}$$

Eq. (6) can be simplified for the case of small relative shoreline change ($|\Delta y/W_*|<<1$) and for $\kappa = 0.78$ obtaining:

$$\frac{\Delta y}{W_*}\approx-\frac{0.068H_b+S}{Z+1.28H_b} \tag{7}$$

As shown in Eq. (7), the dimensionless shoreline change is much more strongly related to the sea level variation than wave height, with the sea level variation being approximately 15 times as effective. However, the coastal erosion, in dimensional form, strongly depends on the breaking zone width thus, indirectly, on the breaking wave height, accordingly to the relation $W_*= (H_b/\kappa A)^{3/2}$ for the equilibrium profile.

The relative role of breaking waves and sea level variation has been investigated in this paper also for different profile forms. The linear profile has been investigated too, even if a planar beach may at first appear to be an unrealistic shape. In fact, in some instances, such beaches occur, both artificially and naturally. For example, many laboratory studies are carried out over artificial linear profiles. Beach nourishment is often placed on a beach with a nearly planar offshore slope as well as some natural sandy beaches with small sediment sizes have nearly planar foreshores with slopes ranging 1/80 - 1/100. The assumption about the maintenance of the beach profile shape could be even stronger, especially after storm surges when the beach would tend to an equilibrium profile. For planar beaches, where the water depth is given by $h(y)=my$ with m the beach slope, Eq. (6) can be still solved analytically and simplified in the form:

$$\frac{\Delta y}{W_*}\approx-\frac{0.096H_b+S}{Z+1.28H_b} \tag{8}$$

In Eq. (8), the coefficient associated to the wave set-up in the evaluation of the beach erosion is about 40% larger with respect to that of the equilibrium profile calculated in Eq. (7).

In the present paper Dean's method is applied in presence of a submerged breakwater (Fig. 2) in order to provide a design guide for the best adaptation of existing structures to the expected sea level rise and the more severe sea storms.

The structure is considered to act by reducing the incident wave height at structure toe, H_i, through a transmission coefficient, K_t, and by increasing the water level with its associated piling-up, P_0. In Fig. 2, S_0 refers to the variation in water level seaward of the structure as the combined effect of storm surges and sea level rise due to climate change, while S is the landward water level variation obtained by adding the piling-up to S_0.

The transmission coefficient, defined as the ratio of the transmitted over the incident wave height, $K_t = H_t/H_i$, has been derived through the following expressions suggested by Briganti *et alii* (2003) for permeable breakwaters:

$$K_t=-0.4\frac{R_c}{H_i}+0.64\left(\frac{B}{H_i}\right)^{-0.31}\left(1-e^{-0.5\xi}\right) \tag{9}$$

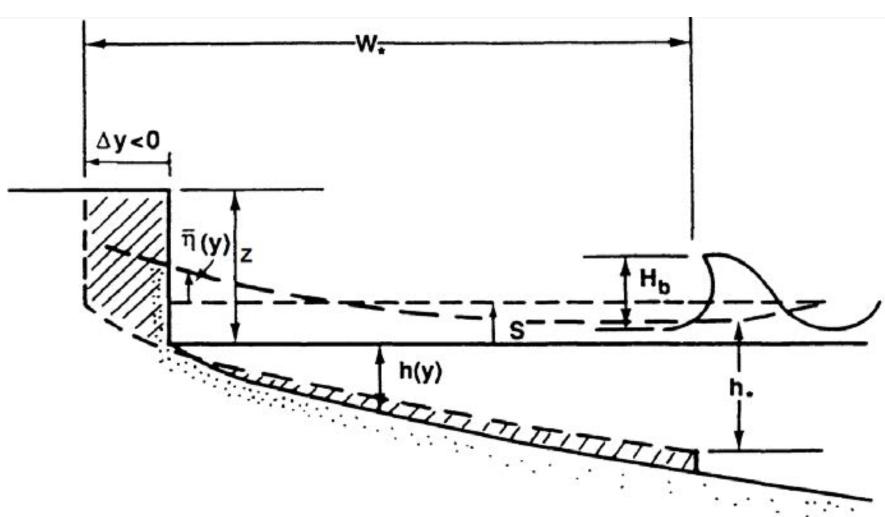

Fig. 1 - Profile geometry and notation for shoreline recession due to waves and increased water level (adapted from Dean & Dalrymple, 2004).

$$K_t = -0.35\frac{R_c}{H_i} + 0.51\left(\frac{B}{H_i}\right)^{-0.65}\left(1 - e^{-0.41\xi}\right) \qquad (10)$$

where ξ is the Iribarren parameter defined as tan$\alpha/(H_i/L)^{1/2}$, R_c is the freeboard (negative for submerged structures), B is the crest width.

Eq. (9) is the original formula of D'ANGREMOND *et alii* (1996), valid for relatively narrow crested structures ($B/H_i \leq 8$) while Eq. (10) is derived for wider crests ($B/H_i \geq 12$). Both formulae are limited by lower and upper bounds: $0.075<K_t<0.80$ for narrow crests and $0.05<K_t<-0.006B/H_i+0.93$ for wide crests.

The piling-up for zero net inshore discharge has been determined by the CVB method, described in CALABRESE *et alii* (2003, 2005). Under the simplifying hypothesis of «static» piling-up, the CVB method provides the following expression:

$$P_0 = \frac{\left(H_i^2 - H_t^2\right)}{16h_m}\left(\frac{1}{2} + G\right) \qquad (11)$$

where $G = (2kh/\sinh(2kh'))$, h_m is the average water depth from the breaking point to the breaking end. Defined h_{m0} the average water depth in absence of piling - up, in presence of piling-up the average depth is increased by $P_0/2$; when the breaking ends near the berm inshore edge the depth is:

$$h_m = h_{m0} + P_0/2 = h' - \left\{h_c - \frac{\left(h_b + R_c\right)}{2\left(B + x_b\right)}x_b\right\} + P_0/2 \qquad (12)$$

where h_c is the structure height, h' is the water depth at structure toe (h_e+S_0 for this study), h_b is the breaking depth and x_b is the distance between the breaking point and the seaward crest edge. As suggested in CALABRESE *et alii* (2005), the water depth at the breaking point has been roughly estimated by coupling the linear shallow water shoaling theory and the KAMPHUIS (1991) breaking criterion:

$$h_b = \left[\frac{H_{m0i}}{0.56\exp\left(3.5\tan\alpha\right)}\right]^{4/5}\left(h'\right)^{0.2} \qquad (13)$$

where H_{m0i} is the incident significant wave height calculated integrating the incident power spectrum for frequencies larger than 0.5 times the offshore peak frequency.

Solving by iteration Eq. (11) and (12), it is possible to derive the value of the piling-up which is added to the presumed sea level variation S_0.

ANALYTICAL MODEL RESULTS

The performed analyses consider an equilibrium profile of the form $h = Ay^{2/3}$, where the factor A has been chosen equal to 0.09. This value is representative of sand sediment with a mean diameter d = 0.17 mm, according to DEAN (1987).

The sea level variation S_0 is assumed to be equal to 0.5, 1.0 and 1.5 m. The results are presented in Fig. 3 where the beach recession Δy is plotted with respect to the submergence R_c for two different values of the offshore wave height (H = 2 m, 3 m). The wave height has a significant influence on the beach erosion since an increase of the wave height is responsible of a larger active zone W_* and, as a consequence, of a larger shoreline recession.

Moreover, higher values of the submergence ($|R_c|$) induce larger beach recessions until, for large ratios $|R_c|/H_i$, the transmission coefficient reaches the upper bound in Eq. (9). A further increase of $|R_c|$ has no longer influence over the wave transmission but reduces the effect of the piling-up. This trend is particularly appreciable for smaller waves (left panel of Fig. 3).

An existing Low Crested Structure (LCS) could become less efficient in terms of beach protection due to climate change (higher sea level and waves) and a structure adaptation could be needed. A purpose of this paper is to identify the best adaptation scenario for some possible combinations of sea level rise and wave height. The structure could be adjusted either by reducing the freeboard R_c or by increasing the berm width B.

The results for this comparison are shown in Fig. 4. The analyses are performed with a wave height H = 3 m, a wave

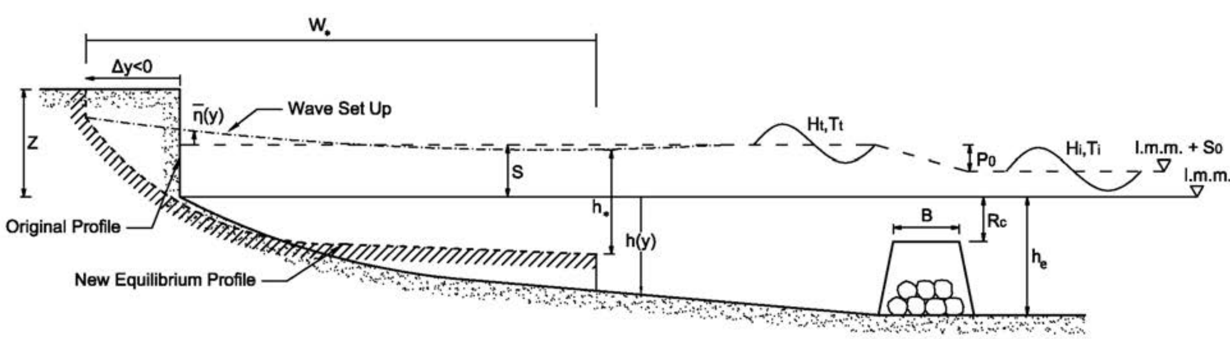

Fig. 2 - Profile geometry and notation for beach recession including wave set-up in presence of a submerged structure.

period T = 8 s and two scenarios of sea level increment (S_0=0.5, 1.5 m). The submerged structure is located at a water depth of h_e = 4 m and five different berm widths B are tested. In such case the incident wave height H_i corresponds to the offshore wave height H (no breaking occurs at the offshore side of the structure being the breaking threshold given by $\kappa(h_e+S_0)$ always larger than H). Each line Δy-R_c in Fig. 4 is plotted for a different berm width B. Three points along these lines are highlighted and are related to different submerged structures, as shown in Fig. 5. LCS_1 is assumed as a reference structure with a berm width of 8 m and a submergence of 2.0 m, LCS_2 and LCS_3 are referring to two alternative adaptation scenarios which can be performed with the same amount of material and, consequently, with about the same cost: LCS_2 consists in a reduction of the freeboard up to a value of - 0.50 m keeping constant the berm width of 8 m, while LCS_3 represents the increasing of the berm width to a value of 24 m maintaining the same submergence of 2.0 m.

For a smaller value of the sea level variation (S_0=0.5 m, left panel), the structure adaptation with a wider berm (from LCS_1 to LCS_3) provides a larger reduction of the shoreline recession. Indeed, the intervention LCS_2 reduces the beach erosion by 11.6% with respect to the reference configuration LCS_1, while the structure LCS_3 leads to a 14% smaller erosion of the shoreline with respect to that obtained with LCS_1. On the contrary, for significant values of the sea level increase (S_0=1.5m, right panel), a higher structure (from LCS_1 to LCS_2) is more efficient than a wider one (the shoreline erosion reduction is 21.4% for LCS_2 and 12.3% for LCS_3).

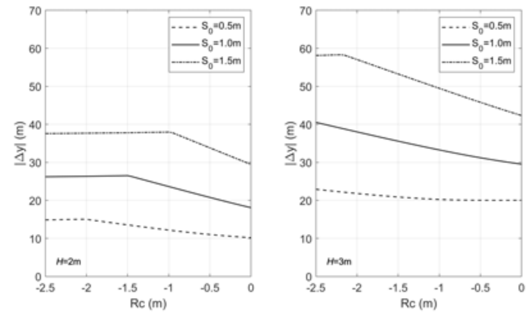

Fig. 3 - Results of the analytical model applications showing the beach erosion Δy over the freeboard R_c for a structure with a berm width B = 8 m located at a water depth h_e = 4 m. Different scenarios of sea level variation (S_0 = 0.5, 1.0, 1.5 m) and two incident wave heights H = 2 m (left panel) and H = 3 m (right panel) are considered.

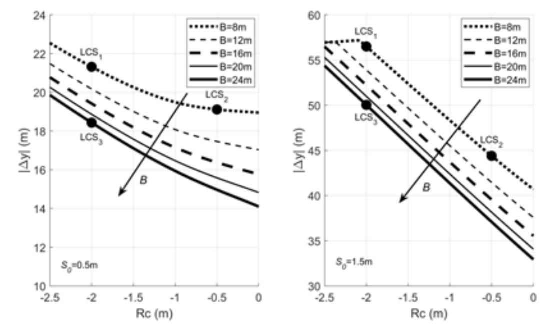

Fig. 4 - Results of the analytical model applications for the comparison between different adaptation interventions over a submerged structure located at a water depth h_e = 4 m. Two scenarios of sea level variation S_0 = 0.50 m (left panel) and S_0 = 1.50 m (right panel) with H = 3.0 m and T = 8.0 s are tested.

NUMERICAL MODEL

Dean's procedure for the evaluation of beach recession including the effect of waves has been applied to measured beach profiles in order to verify its applicability on any beach profile shape. The numerical model provides an approximate solution for the sediment volume balance equation now written in the form:

$$\int_{\Delta y}^{0}\left[Z-S-\bar{\eta}(y)\right]dy+\int_{\Delta y}^{W_*+\Delta y}h(y-\Delta y)\,dy= \\ =\int_{0}^{W_*+\Delta y}h(y)\,dy+\int_{0}^{W_*+\Delta y}\left[S+\bar{\eta}(y)\right]dy \qquad (14)$$

The followed procedure is the same of the analytical model but in this case the integral of Eq. (14) is solved numerically over the generic beach profile. In these analyses the integration of Eq. (14) has been performed assuming that the beach profile translates landward and upward due to the new water level without change in form. This hypothesis, which is the same of that adopted by Bruun and Dean, is clearly a simplification of the phenomenon but it allows to easily execute comparative analyses between the different types of structure adaptations.

The emerged beach face has been approximated with a "square-berm", as in the equilibrium beach profile applications of Dean (1991). Profiles with sloping beach face could be adopted too; in the case of a berm with a linear form, the associated recession is higher than the value predicted using the square-berm profile because the total shoreline retreat includes a term due to the inundation effect, roughly equal to S/m_b with m_b the berm slope.

The presence of the structure is taken into account since it modifies the input conditions (H and S_0). As mentioned above, the offshore wave height is reduced while an increase in terms of sea level is generated by the piling-up that is added to the initial condition S_0. The breaking threshold is calculated by multiplying the total water depth (h_e increased by S_0) and the breaking index κ. If the offshore incident wave reaches the structure in non-breaking conditions (H smaller than the breaking threshold), the input wave height H is directly used in the computation as H_i. Otherwise, the offshore wave breaks inducing a set-up seaward of the structure and, therefore, a further increment of the water level. In this situation, the offshore wave height is reduced to

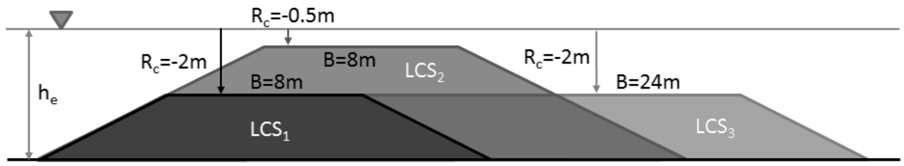

Fig. 5 - Characteristics of three different compared structures. LCS_1 is the "reference" structure with B = 8 m and R_c = - 2 m, LCS_2 is the adaptation performed by reducing the submergence up to - 0.5 m and LCS_3 is the structure with same submergence of LCS_1 but larger berm width (B = 24 m).

the maximum admissible value, proportional to the total water depth. The wave height that arrives at the structure toe, H_i, is then reduced to H_t due to the transmission over the structure. By neglecting the shoaling effect, H_t is the maximum wave that breaks over the beach profile and it is used to identify the breaking depth h_* and its associated distance from the shoreline W_*. In fact, the breaking that occurs on the sea side of the structure is taken into account only as a further increment of the water level while only the transmitted wave becomes responsible of the active beach profile. Therefore, in the computations, the volume balance has been performed considering the sediment motion at the inshore side of the structure.

From the breaking point of the transmitted wave, the water level is increased by an internal set-up, which varies with the depth along the cross-shore distance y. Known the total sea level variation S, the wave set-up $\overline{\eta}(y)$, and the width of the breaking zone W_*, Eq (14) is solved numerically and the beach recession Δy is evaluated.

NUMERICAL MODEL RESULTS

The numerical procedure has been applied to four different beach profiles: equilibrium profile, linear profile and two real beach profiles.

The analysed equilibrium profile is of the form $h=Ay^{2/3}$, with A = 0.09, while for the linear profile, a slope m equal to 1/100 has been assumed. The real beach profiles have been chosen along the Italian coast of the Adriatic Sea in Senigallia and Rimini. They are free beach profiles, without coastal protections, and are characterized by a monotone beach profile (Fig. 6). Beaches with bar formation are not considered in these analyses because in this condition the integration method does not work well due to the high variation of W_* for small variations of h_*.

Several numerical analyses have been performed for three different submerged structures, as considered for the analytical study (LCS_1, LCS_2 and LCS_3 in Fig. 5). Offshore waves with wave period T = 9 s and different wave heights (H varies from 2.0 m to 5.0 m with increments of 0.5 m) are simulated, while S_0 is assumed to vary in the range of 0.0 - 2.4 m with increments of 0.2 m. The wave period has been kept constant and its value has been chosen in order to be representative of the site wave condition. The water depth h_e at the structure toe (considered at the onshore side) has been changed in the range 2.0 m - 5.0 m with increments of 0.5 m. Combination of all these parameters (H, T, h_e, S_0) leads to 637 numerical simulations for each submerged structure (LCS_1, LCS_2 and LCS_3) and each beach profile (Equilibrium, Linear, Senigallia and Rimini profiles). The total amount of numerical simulations with the defences was equal to 7644. The summary of the numerical simulations and the range of the numerical results are reported in Table 1. Moreover, other numerical simulations have been run for all the different beach profiles with no defences.

At first, the numerical method has been applied over the four different beach profiles, to evaluate how the shoreline shape affects the response. The efficiency of the alternative adaptations is expressed with the ratio $\Delta y/\Delta y_0$, which measures the beach recession induced in the presence of the structure with respect to that obtained with no defences (free beach profile). As expected, for all wave and level conditions and for all the examined beach profiles, shorelines without any coastal protection are more vulnerable to erosion, as shown in Fig. 7. However, the use of submerged structures to counter the recession is not equally advantageous depending on the beach profile shape. The response of the planar beach is much less affected by the defences compared to those of the other profiles, being larger the values of $\Delta y/\Delta y_0$. The influence of the different structure adaptations on the shoreline recession is similar for all the beach profiles. Indeed, by comparing the different adaptation strategies, the reference submerged structure (LCS_1) induces, in almost each condition, a larger shoreline recession with respect to those obtained in presence of the adapted structures (LCS_2 and LCS_3). Therefore, both adaptation interventions are efficient in terms of beach protection. In all the panels of Fig. 7, an intersection point between the continuous and dashed lines representing the patterns of the shoreline recession for the adapted structures LCS_2 and LCS_3, respectively, is observed. Indeed, the enlargement of the structure (LCS_3) is more efficient with respect to the reduction of the submergence (LCS_2) when the sea level variation is small ($S_0 \approx$ 0.5 m), while for larger sea level variations the adapted structure performance changes, becoming more efficient the higher structure (LCS_2). Mostly for small wave heights a particular behaviour is found when the sea level variation is very small: the higher structure (LCS_2) is disadvantageous even with respect to the original structure (LCS_1) and such adaptation

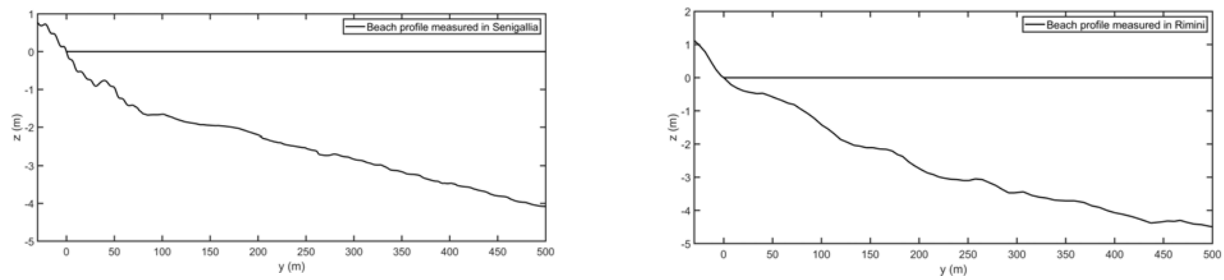

Fig. 6 - Real beach profiles measured along the Adriatic coast of the Mediterranean Sea in Senigallia (left panel) and Rimini (right panel).

intervention results ineffective. Although a higher structure acts reducing the incident wave height, it induces also a larger piling-up which is more relevant when the sea level increase is small. This effect is found for all the profile shapes but it is particularly relevant for the linear and Rimini profiles. Therefore, the structure adaptation strategy might be chosen in relation to the specific cross-shore profile. The panels of Fig. 7 show the results for three different values of *H*, proving, as expected, that the wave height influences the performance of the submerged structures. Fig. 7 shows that for larger wave heights the intersection point of the adapted structure patterns moves leftward: the adapted structure LCS_2 becomes more effective even for lower S_0. A borderline case is observed for the Senigallia profile, in which, for H = 5 m, it becomes always more efficient to adapt the structure by reducing its freeboard. Such result suggests that when the wave height increases and the sea level variation is significant, the effects of the wave transmission become dominant in terms of beach recession with respect to those of the piling-up, hence, the enlargement of the structure induces lower benefits in terms of shoreline recession. The adaptation interventions should be chosen with care according with the sea level rise prediction and the specific-site conditions.

Since the qualitative beach response seems to be similar among the different profiles, the influence of the wave height and

Low crested structure	h_e (m)	S_0 (m)	H (m)	T (s)	$\|\Delta y\|$ (m)			
					Equilibrium profile	Linear profile	Senigallia profile	Rimini profile
LCS_1 R_c = - 2.0 m B = 8.0 m	2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0	0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4	2.0	9.0	2.5-59.8	6.7-119.8	2.9-93.7	5.0-82.7
			2.5	9.0	3.0-77.6	7.6-140.2	3.4-125.2	5.6-102.1
			3.0	9.0	5.0-87.6	11.4-145.1	6.6-144.8	8.0-108.9
			3.5	9.0	7.5-104.0	15.9-160.5	10.8-172.7	11.6-132.4
			4.0	9.0	10.6-105.1	21.1-161.0	15.7-173.8	15.7-137.9
			4.5	9.0	14.2-111.9	26.8-164.4	21.8-183.0	19.9-147.6
			5.0	9.0	18.3-110.0	33.5-161.7	28.4-179.6	24.8-144.5
LCS_2 R_c = - 0.5 m B = 8.0 m	2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0	0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4	2.0	9.0	2.6-60.2	8.1-120.5	2.6-94.2	6.8-83.3
			2.5	9.0	3.9-77.5	11.6-140.6	5.6-125.2	9.6-102.6
			3.0	9.0	5.6-77.2	15.6-136.8	7.4-123.8	12.6-99.1
			3.5	9.0	7.6-86.9	20.2-148.5	9.3-144.8	15.9-107.4
			4.0	9.0	9.9-88.2	25.2-149.6	11.6-146.3	19.4-108.3
			4.5	9.0	12.6-88.2	30.7-145.5	13.9-145.2	23.0-115.1
			5.0	9.0	15.6-95.1	36.7-153.5	17.3-158.1	26.9-122.2
LCS_3 R_c = - 2.0 m B = 24.0 m	2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0	0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4	2.0	9.0	1.6-59.8	4.8-119.8	2.0-93.7	3.8-82.7
			2.5	9.0	2.1-77.6	6.0-140.2	2.5-125.2	4.6-102.1
			3.0	9.0	4.0-87.0	10.0-144.5	4.5-144.2	7.3-107.3
			3.5	9.0	6.4-93.9	14.6-152.3	7.1-154.6	10.2-121.1
			4.0	9.0	8.7-93.7	19.3-152.2	9.9-154.3	13.2-121.0
			4.5	9.0	11.4-98.7	24.5-156.3	12.3-162	17.2-124.4
			5.0	9.0	14.9-96.5	30.6-156.1	16.2-160.6	22.7-124.2

Tab. 1 - Shoreline recession range (minimum and maximum) for each low crested structure (LCS_1, LCS_2, LCS_3) and beach profile (Equilibrium, Linear, Senigallia and Rimini).

of the structure location on the shoreline recession Δy is analysed only for the equilibrium profile (A = 0.09) and the results are reported in Fig. 8. As expected, the increase of the wave height induced a larger shoreline recession (upper panels of Fig. 8). The influence of the wave height is almost equal for the two adapted structures (LCS_2 and LCS_3) while a larger shoreline recession is obtained for the reference structure LCS_1.

In all the previous analyses, the water depth at the structure toe was fixed (h_e = 3 m). As above reported, other numerical simulations have been performed by changing also the water depths h_e at the structure toe (onshore side), in order to evaluate the effect of the structure location on the shoreline recession. In such analyses the wave height is fixed (H = 4m) and the structures are the same of Fig. 5. The results are shown in bottom panels of Fig. 8, where the shoreline recession is plotted over S_0 for different water depths. Each panel refers to a structure configuration identified by a berm width and a submergence. This means that in each panel, by varying h_e, the compared structures have different volumes. By moving the structure landward at lower water depths h_e, a smaller volume of material is needed to maintain the same R_c. The location of the defence structure influences the erosion in a different way depending on S_0. The shoreline recession depends on the water depth at the structure toe and on the breaking position that moves landward for increasing sea level. For all the structures, the behaviour is similar: a region with overlapped data is observed for S_0 about equal to 1m. In particular, when S_0 is small (about S_0 < 0.8m), the largest erosions are obtained if the structure is located at larger water depths, approximately larger than the breaking depth ($h_e+S_0 \approx H/\kappa$). On the contrary, for large values of S_0 (about S_0 > 1.4m), larger shoreline recessions are found when the structure is moved landward of the breaking point. Note in Fig. 8 that for large wave heights and for small water depths, the data are not reported for larger values of S_0 because the sea level at the shoreline exceeded the berm height, hence, the volume balance became meaningless and the numerical simulations have been stopped.

To better understand the combined influence of both the structure location (water depth at the structure toe) and the wave height on the shoreline erosion, for each analysed beach profile, the case of the reference structure (LCS_1) has been selected and the results (Fig. 9) are expressed through the following ratio:

$$\Delta y'_{i,j} = \frac{\Delta y\left(h_e = i\right)}{\Delta y\left(h_e = j\right)}\% \tag{15}$$

which is the percentage ratio between the shoreline recessions obtained with the structure located at the water depths i and j,

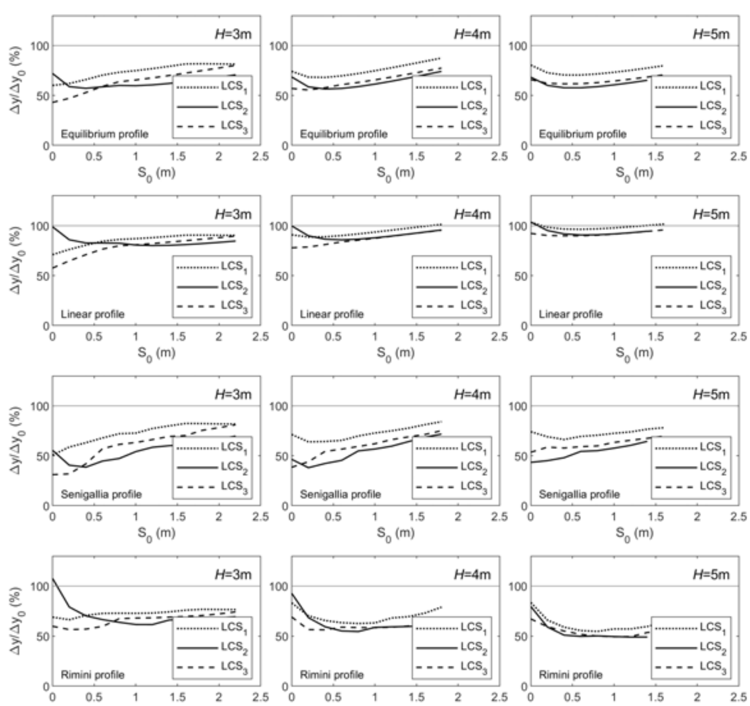

Fig. 7 - Comparison of the nondimensional beach erosion, $\Delta y/\Delta y_0$, defined as the ratio between the beach recession induced in the presence of the structure (reference, higher and larger structures) with respect to that obtained with no defences. Water depth h_e = 3 m, wave height H = 3 m (left panels), H = 4 m (middle panels), H = 5 m (right panels). Beach profiles: equilibrium profile; linear profile; beach profile of Senigallia; beach profile of Rimini.

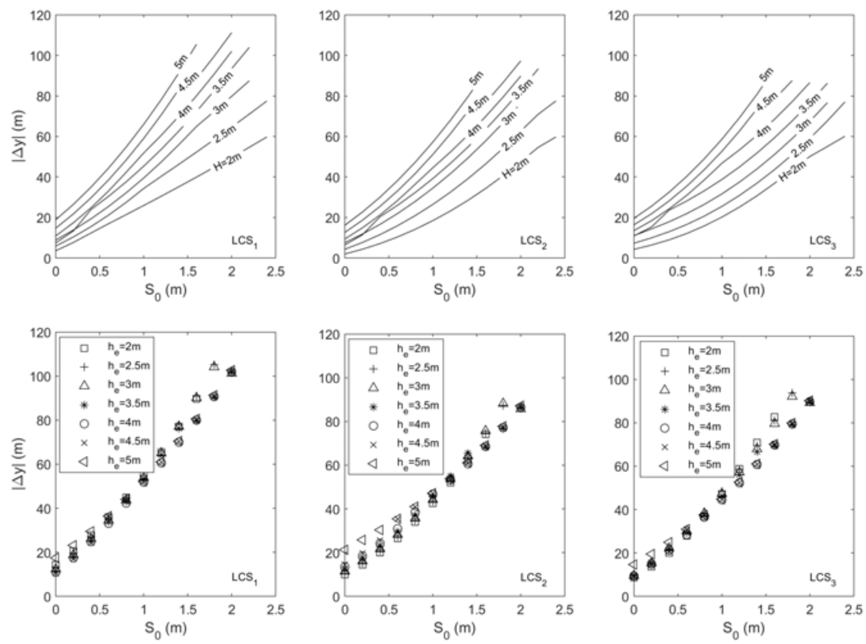

Fig. 8 - Shoreline recession Δy vs sea-level increment S_0 for h_e = 3 m and different wave heights (H = 2-5 m) (top panels); Δy vs S_0 for H = 4 m and different water depths at the base of the structure (h_e = 2 - 5 m) (bottom panels). Reference structure LCS_1 (left panels), higher structure LCS_2 (middle panels) and larger structure LCS_3 (right panels). Equilibrium profile, Z = 3 m, T = 9 s.

respectively. This ratio has been evaluated for three different incident wave heights (H_i = 2, 3, 4 m). For examples, panels a - b - c show the percentage reduction in terms of beach erosion estimated when the structure, initially located at a water depth h_e of 5 m, is moved to a water depth he of 4 m. The incident wave height H is 2 m (panels a - d - g), 3 m (panels b - e - h), 4 m (panels c - f - i). The trend is almost the same for all the beach profiles, unless small variations due to the particular shapes of the natural beaches, so the choice of the optimal position of the structure might be considered profile independent.

For small wave heights with respect to the water depth (Fig. 9, panels a-b-d), the landward moving of the structure has negligible effects in reducing the recession. The major advantages in terms of beach protection occur when the structure is located at a water depth almost equal to the wave height ($h_e \approx H$). Indeed, increase of defence efficiency (lower ratios $\Delta y'_{i,j}$) is obtained for the wave heights: H = 4 m when moving the structure from a water depth h_e = 5 m to h_e = 4 m (panel c); H = 3 m when moving the structure from h_e = 4 m to h_e = 3 m (panel e); and H = 2 m when moving the structure from h_e = 3 m to h_e = 2 m (panel g).

Moving onshore the structure towards smaller water depths ($h_e < H$) seems to be even disadvantageous since it produces an increase of beach erosion as observed in panels f - h - i. This is due to the higher water level increment induced by the external set-up of the offshore breaking wave at the structure toe. Furthermore, the landward moving of the structure induces the formation of salient/tombolo and the water quality becomes worse.

CONCLUSIONS

The present study provides a practical method for order-of-magnitude projections of shoreline recession due to the effect of sea level rise, storm surge and waves for different configurations of submerged structures over any beach profile type. A parametric analysis has been performed in order to take into account different hydrodynamic conditions, profile shapes and submerged breakwater geometry.

The analytical approach of Dean (1991) has been applied in here to a linear profile for the evaluation of the beach erosion and, hence, a new analytical solution for the beach recession of a planar beach has been provided.

The results obtained by the application of the analytical solution for the linear profile showed that, with respect to the equilibrium profile case, the wave contribution to the erosion is larger of about 40%.

The presence of a submerged breakwater induces some changes in the hydrodynamic conditions: a disadvantageous effect is represented by the piling-up that increases the water level in the inner zone while a favourable effect is the reduction of the incident wave height through the application of a transmission coefficient.

The influence of submerged structures is evaluated with both analytic and numerical approaches. The analytical model

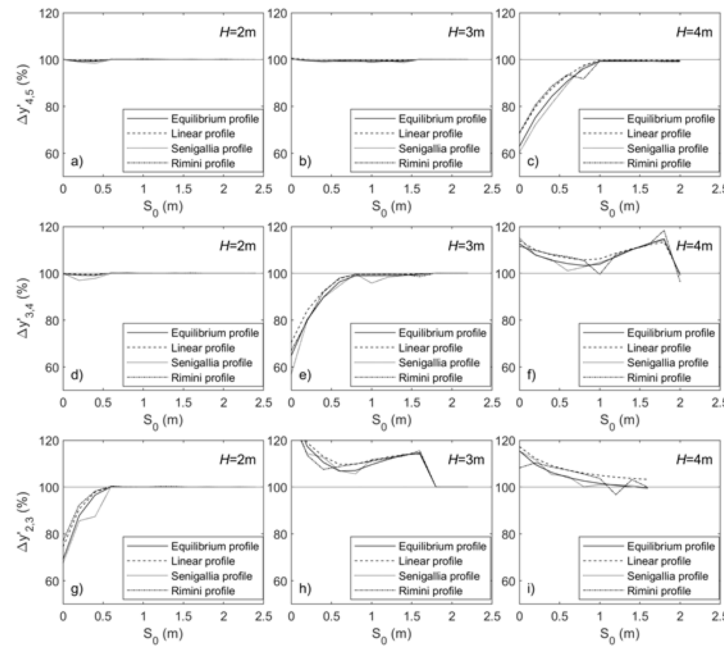

Fig. 9 - Percentage ratio $\Delta y'_{i\text{-}j}$ between the shoreline recessions at the water depths i and j, respectively, vs S_0 for H = 2 m (left panels), H = 3 m (middle panels) and H = 4 m (right panels). Structure LCS_1, Z = 3 m, T = 9 s.

is based on the equation of Dean for an equilibrium profile and on the analytical solution for the linear profile here proposed. The numerical model solves the sand volume integration over different beach profiles, both theoretical and experimental.

The climate change expected in the next years will lead to an increase of the mean sea level and of the frequency and intensity of storm events. Therefore, a submerged structure could become less efficient in terms of beach protection and must be adapted to mitigate these effects. Two alternative adaptation strategies of an existing "reference" structure are analysed: the height of the structure is increased to reduce the submergence or the berm width is enlarged keeping the same submergence of the reference structure. The two adapted structures make use of the same amount of material and, hence, involving about the same cost.

The results of the analytical model highlighted how the efficiency of the enlargement of the structure is the better intervention to perform for smaller values of sea level variation (S_0 = 0.5 m). Conversely, when the sea level increases (S_0 = 1.5 m) a higher structure ensures a better beach protection. This is confirmed also by the results obtained by the application of the numerical approach. Even if the results depend on the profile shape and on the wave height, a threshold value of S_0 about equal to 0.5m is found to identify the conditions for which it is recommended to enlarge the structure (S_0 < 0.5m) from those for which a higher structure is more efficient on the beach protection (S_0>0.5m).

The influence of the location of the structure, water depth at the structure toe h_e, has also been studied. The landward movement of the structure from larger water depth has not any appreciable effect until the breakwater is placed at a water depth almost equal to the incident wave height. In this condition, a larger reduction of the erosion is obtained, especially for small sea levels. A further approach of the structure to the coastline will induce the disadvantageous condition of a higher water level on the structure due to the external set-up and, as a consequence, a more significant erosion. The optimal position of the structure is obtained at a water depth $h_e \approx H$. Note that such results are almost profile-independent and, thus, they can be considered representative of the analysed structure. When longshore effects influence the morpho-dynamic processes of the shoreline (e.g. oblique waves attack), transversal models cannot be applied. In these cases, the effectiveness of adaptation interventions needs to be evaluated by using 2D models.

Despite the limits of this model that computes the shoreline recession only by performing a sediment volume balance, its application can be useful to give preliminary information for the design of a new submerged breakwater or for the choice of the best adaptation of an existing one. Since it was not possible to validate the model with experimental data, not available for the real study cases, the specific values of erosion can be considered as a qualitative reference for the beach response to climate change according to the physics of the phenomenon.

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