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Random wave run-up with a physically-based Lagrangian shoreline model

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Abstract

In the present paper the run-up of random waves was calculated by means of a numerical method. In situ measurements based on a video imaging technique have been used for the validation of the present numerical model. The on-site run-up measurements have been carried out at *Lido Signorino* beach, near *Marsala*, Italy,along a transect, normal to the shore. A video camera and a linear array of rods have been used to obtain field data. Numerical simulations with a 1DH Boussinesq-type of model for breaking waves which takes into account the wave run-up by means of a Lagrangian shoreline model have been carried out. In such simulations random waves of given spectrum have been propagated in a numerical flume having the same beach slope of the measured transect. The comparison between registered and estimated run-up underlined an acceptable agreement. Indeed, the numerical model tends to underestimate the actual $R_{2\%}$, with the maximum underestimate being less than 24%, which is a reasonable error in many cases of engineering interest.

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Keywords: irregular wave run-up; Boussinesq numerical model; shoreline.

1. Introduction

In the field of coastal engineering, the investigation of the swash zone process has become recently a very hot topic, due to its relevance in the context of coastal flooding and erosion phenomena.

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For example, Baldock and Holmes (1999) found in numerical simulations that incident band swash saturation was related to bore-driven swash, which also scales with wave period and beach slope. Moreover, Bellotti and Brocchini (2005) present simulations of the shoreline displacement based on bore run-up theory which give excellent agreement with experimental data for regular waves, wave groups and random waves.

Moreover, a recent numerical investigation concerning the simulation of swash zone fluid accelerations, concluded that given the poor correlation between local acceleration and pressure gradient, it is not likely that local acceleration can be used as a surrogate for pressure gradient (Puleo et al., 2007).

Notwithstanding the advances of the research, numerical models are often calibrated by using datasets produced during laboratory experiments (Xuan et al. 2013, Ramirez et al. 2013, Li et al. 2012). Although the analysis of laboratory results can provide some insights on what happens in Nature, the processes occurring in the field may be quite different from lab experiments due to many causes, e.g. the use of idealized wave conditions, such as regular waves, or turbulence-related scale effects which can be hardly eliminated in the lab.

To become a practical tool of engineering interest, the performance of a numerical model should be compared to field data conditions, where the effects of phenomena such as the actual irregularity of wave trains, presence of turbulence, infiltration and ex-filtration processes are included.

To contribute to fill such a gap, in the present work a comparison between the numerical results obtained by the Lagrangian shoreline model coupled with a Boussinesq type model proposed by Lo Re et al. (2012) and the results of a field campaign on wave run up on a mild slope sandy beach is discussed.

The paper is organized as follows: the next Section provides a description of the numerical model; then Section 3 describes the field campaign; Section 4 presents the procedures used to input the data in the numerical models, based on field data, and to analyze the results; Section 5 illustrates and discuss the obtained results; finally Section 6 draws the conclusions of the work.

2. Methods

The Boussinesq-type model considered in the present paper is able to propagate the waves from relatively small water depth (kh=0.7 where k is the wave number and h is the local water depth) up to the shoreline. Therefore, in order to compare the results of the numerical model with the field run-up measurements, a cascade of models (see Fig. 1) has been implemented to propagate the data on the waves gathered offshore, up to the initial water of the Boussinesq model.

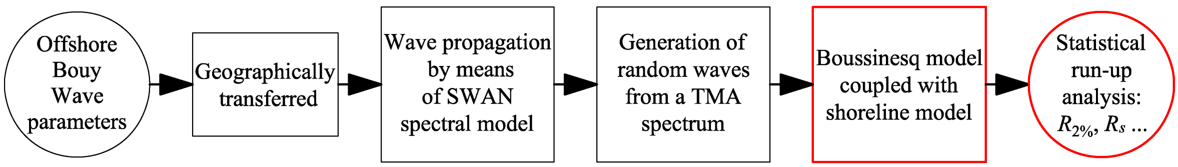

Fig. 1. Flow chart of the used methods

More in details, first the wave data recorded by the buoy of the Italian Wavemeter Network which was nearest to the study area, were geographically transferred to a point offshore of the invesitgated beach by using the model of Vincent (1984). Once the offshore wave characteristics were known, the latter were propagated from deep to shallow water using the well-known SWAN model (Simulating Waves Nearshore model) for the spectral propagation of the wave motion (Boij et al. 1999, Holthuijsen et al. 1993, Ris et al. 1999). The results of the SWAN model, in the nearshore region were then used as input waves for estimating run-up and run-down my means of the Boussinesq-type model proposed by Lo Re et al. (2012). In particular the geographically transferred offshore wave data were propagated by means of the spectral model SWAN up to the water depth h=5 m in front of the beach, in order to determine the significant wave height and the peak period of the attacking waves. Assuming a TMA spectrum, the info on significant wave height and wave period have been converted into the time series of an energetically equivalent irregular wave train, which has been propagated using the shoreline

Lagrangian numerical model of Lo Re et al. (2012) in a numerical flume having the same beach slope of the measured transect (Fig.s 2).

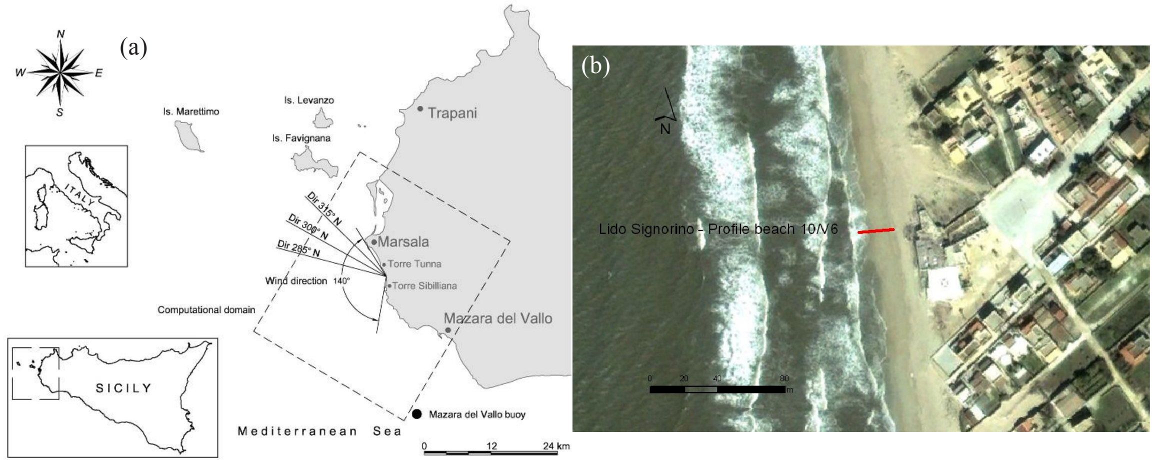

Fig. 2.(a) Geographic position of the Lido Signorino beach . The computational SWAN domain of the study area is showed in dashed line; wind and principal wave directions together with buoy position are also showed. (b)Aerial photographs and profile of the studied transect

It is widely acknowledged that numerical simulation of shoreline oscillations with a Boussinesq type of model is a difficult task, because such kind of models cannot discriminate well between the wet and dry region.
In the shoreline model of Lo Re et al., 2012, a Boussinesq type model for breaking waves with the governing equations solved in the $\zeta - u$ form was implemented, where ζ is the free surface elevation and u is the depth-averaged horizontal velocity. The values of the variables ζ e u were calculated inside the wet domain, whereas the shoreline position (defined by means of its horizontal coordinate $\xi(t)$ perpendicular to the coast) and its velocity u_s were calculated by means of the Lagrangian shoreline equations. Fig. 3 shows a sketch with the definition of the variables and of the numerical scheme adopted at the shoreline.

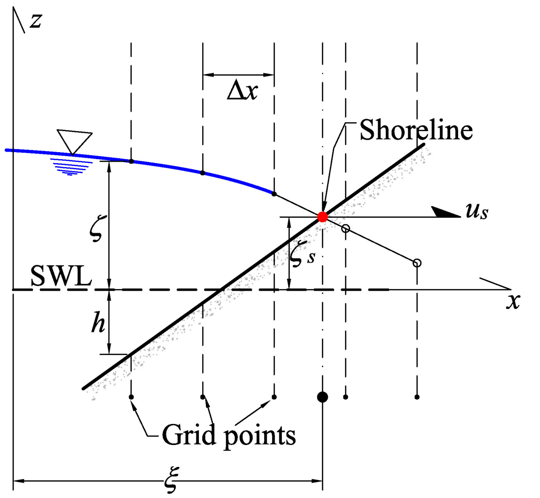

Fig. 3. Sketch of the variables used by the shoreline model of Lo Re et al. (2012)

In the case of an orthogonal wave attack as the one we considered here, the variable ξ is only function of time, i.e. $\xi = \xi(t)$ and the kinematic condition at the shoreline is the following:

$$\frac{d\xi}{dt} = u_s \tag{1}$$

Such a relation states that the fluid particles at the shoreline remain along the shoreline (Prasad and Svendsen 2001). Moreover the momentum equation at the shoreline must be also considered in order to close the problem; in dimensional form such a shoreline equation reads:

$$\frac{du_s}{dt} = -g\left.\frac{\partial\zeta}{\partial x}\right|_s + F_{fric} \tag{2}$$

where $\partial\zeta/\partial x|_s$ is the derivative of the surface elevation evaluated at the shoreline, F_{fric} is the bottom friction force evaluates as follow:

$$F_{fric} = -\frac{f}{h+\zeta}\cdot u\cdot|u| \tag{3}$$

in which h is the local depth, f is the bottom friction coefficient. When the value of F_{fric} becomes too large, due to the small value of the total water depth, a threshold is used. In such a case, the dependency on the water depth has been eliminated and the bottom friction is assumed to be only a quadratic function of the depth-averaged velocity:

$$F_{fric} = -C_f\cdot u\cdot|u| \tag{4}$$

where C_f is a coefficient that was assumed equal to 5.0 m^{-1} in the present work, such a value is based on the work of Lo Re et al. (2012).

3. Study area and field measurements

The beach considered in the present work is known as *Lido Signorino* and is located in the western part of Sicily (Fig. 2a). The Lido Signorino beach has a mild slope, being essentially a dissipative beach with beach face slopes varying from 1.5 to 10.8°. It extends in the N-S direction, for about 3.5 km, between the two headlands called *Torre Tunna* (325°N - 37°45'32.26"N; 12°27'40.00"E) and *Torre Sibilliana* (185°N - 37°43'36.31"N; 12°28'11.23"E). The sector from which waves can arrive has an amplitude of 140°. Note that, because of the presence of the *Egadi archipelago*, the beach is screened marginally by the Favignana Island, located along the 320°N direction.

The beach is made up by very fine Holocene sand with sub-rounded grains constituted by lithic and fossil shell fragments with a carbonate composition. The granulometric analysis gave a mean value of $D_{50} \sim 0.55$ mm and mean granulometric fractions of 0.4% of silt, 0.6% of clay and 99% of sand.

The dominant wind diagram (Fig. 2a), obtained from measurements of the nearby meteorological station of Trapani in the period 2004-2008, shows that the winds which can mainly model the beach have NW-SE and W-E directions. The beach tends toward erosion, with much of the natural dune ridge having been overtaken by urban development. The dunes remain only in the southern part, where the human activity is less intense. At such a location, the dunes are only 2.5 m high on average.

3.1. Wave and topographic data

Offshore wave data were provided by a wave rider buoy offshore Mazara del Vallo, deployed offshore the beach, less than 30 km far from field site and at about 100 m water depth. Such a buoy is managed by the ISPRA - Institute for Environmental Protection and Research of the Italian Government (www.idromare.it).

The beach topographic survey, necessary for the subsequent processing, was focused on measuring the following beach morphological features: the dune scarp line, the berm above sea level and the transect showed in Fig. 4. The slope was determined and then used for assessing the effects of wave motion on the position of the instantaneous land-sea boundary. Bathymetric information 5 m below mean water level were obtained by a nautical map and measurements provided by the *Istituto Idrografico della Marina Militare Italiana* (Hydrographic Institute of the Italian Navy). The sediment analysis allowed the topographic survey slopes to be compared with those usually indicated in the literature for the granulometric data collected.

3.2. Run-up measurements

The run-up on beaches may be measured in different ways depending on the general aim and on the required precision. Records of the water line positions can in principle be obtained by resistance run-up meters or by video-cameras. The technique applied in the present study is based on a monitoring video system. In particular, positions of the swash are measured on transects across the beach, normal to the shore. In each transect a line was built using stakes at 0.5 m intervals (Fig. 4). The first stake was a piezometer and it was close to the beach step. The second stake of the line was placed at a distance of 5 m onshore of the piezometer. A digital video camera Canon Dp10 was placed at a distance of 10 m from the line of stakes (orthogonally) and it was used to record 240 minutes continuously at 16 frame per seconds. The shot videos were recorded considering time windows of thirty minutes and were digitalized in order to extract the wave run-up of each wave.

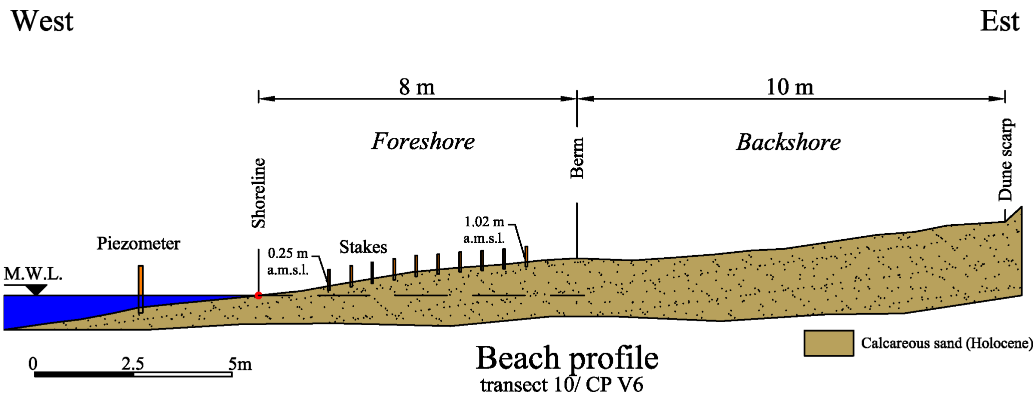

Fig. 4. Cross section of the reference transect for the run-up measurements.

4. Statistical run-up analysis

The time series of the data on wave run-up R, extracted from the optical measurements, where statistically analyzed by assuming a Rayleigh CDF (Cumulative Distribution Function) (Nielsen and Hanslow, 1991):

$$F(R) = 1 - exp\left\{-\frac{(R - R_{100})^2}{L_{zwm}^2}\right\} \tag{5}$$

in which R_{100} is the value exceeded by 100% of the waves, and L_{zwm} is the vertical scale of the distribution. The CDF parameters were estimated from the measured and simulated run-up by means of a least mean square method (LSM). The run-up was estimated from the distribution, for a given exceedance probability, as follows:

$$R(P_s) = L_{zwm}\sqrt{-ln(P_s)} + R_{100} \tag{6}$$

where P_s is the exceedance probability of wave run-up (e.g for $R_{2\%}$ $P_s = 0.02$).

5. Results

In the present work, we present a comparison of the numerical model results with the field measurements of a video monitoring station installed on a tripod near the reference transect (Fig. 4), which acquired coastal zone imagery from 11:30 to 15:30 of 29th march 2011. For this study the measured data was grouped every 30 minutes in order to calculate the statistical run-up parameters. The simulated irregular wave train (30 min) was extracted from a TMA spectrum with significant wave height and peak period equal to those obtained from the SWAN model. Fig. 5 shows an example of the spectrum of random waves extracted from a TMA spectrum with H_s=0.86 and T_p=6.16. The spatial domain of the Boussinesq model was discretized by using $\Delta x = 1$ m and $\Delta t = T/300$ s and the numerical internal wavemaker was located 300 m far from beach. For both the measurements and the numerical results the run-up parameters were calculated using equation (6). Whereas the significant wave run-up was calculated using the following expression:

$$R_s = 4 \cdot \sqrt{V(R)} + R_{100} \tag{7}$$

where $V(R)$ is the variance of run-up.

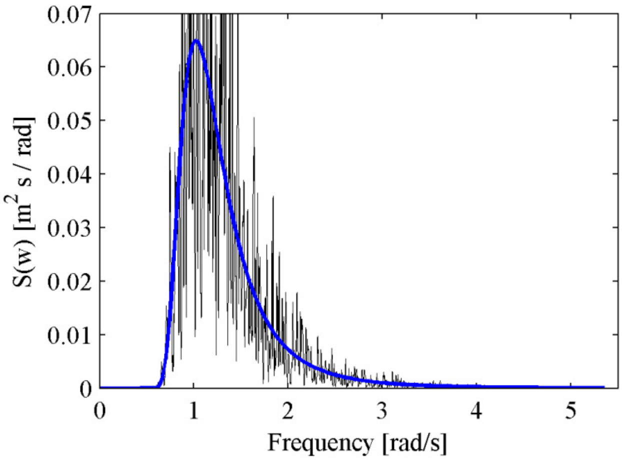

Fig. 5. The wave spectrum of the input random waves and the TMA spectrum (H_s=0.86, T_p=6.16). The continuous bold line is the TMA spectrum the light line is the random wave spectrum without smoothing.

Fig. 6 shows the result of the numerical model for a train of random wave, obtained by assuming a TMA spectrum and H_s=0.86 and T_p=6.16. The shoreline horizontal position values for this simulation were varying between -4.64 m<ξ<11.30 m around a mean of 1.94 (Fig. 6a), while the horizontal shoreline velocity fluctuated from u_s = -3.79m/s to 3.23 m/s with the average equal to 0.002m/s(Fig. 6b). The run-up varied from R=-0.32 and R=0.79 with an average of 0.14 m (Fig. 6c).

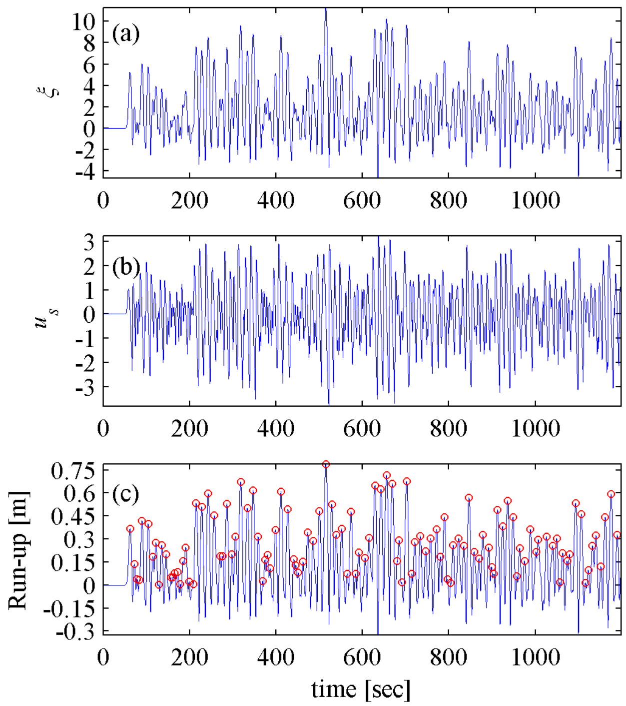

Fig. 6. The Boussinesq model results. In the first panel the horizontal shoreline movement, in the second panel the shoreline velocity, in the third panel the wave run-up. The circle markers represent maximum run-up.

Fig. 7 shows the run-up versus the probability and the empirical frequencies which are calculated using the Weibull formula $f_i = i/(1+N)$. In the figure are also showed the main run up parameters and the *Rs*.

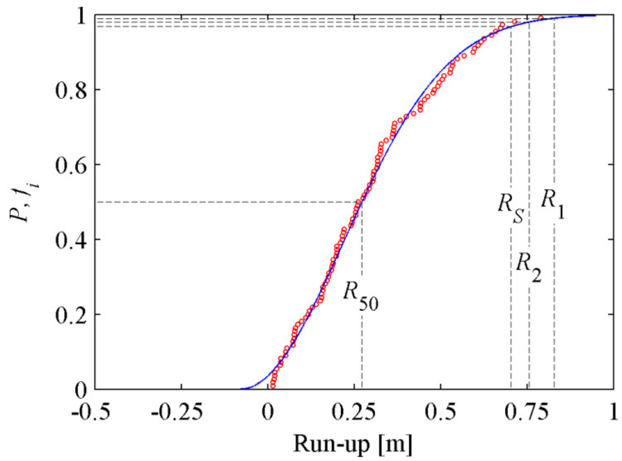

Fig.7. The Rayleigh CDF and the empirical frequencies of numerical wave run-up (*Hs*= 0.86, *Tp*=6.16). The continous line is the Rayleigh CDF the circle markers are the empirical frequencies.

Table 1 shows the comparison between the measured field data and the numerical model results in terms of the 1%, 2%, and significant wave run-up. The matching between the numerical model and the field measurements appears to be rather good. In particular run-up comparisons showed that the model slightly underestimates wave run-up (relative errors are in the range of 0%÷24%).

Table 1. Run-up comparison between field measurements and the numerical model of Lo Re et al. (2012).

	Wave parameters		Field measurements [m]			Numerical model [m]			Percentage error		
Time	H_s [m]	T_p [s]	$R_{1\%}$	$R_{2\%}$	R_s	$R_{1\%}$	$R_{2\%}$	R_s	$R_{1\%}$	$R_{2\%}$	R_s
11:30-12:00	0.86	6.16	0.94	0.89	0.73	0.83	0.76	0.70	-11.7%	-14.6%	-4.1%
12:00-12:30	0.90	7.51	0.99	0.93	0.76	0.93	0.86	0.80	-6.1%	-7.5%	5.3%
12:30-13:00	0.89	6.10	0.90	0.86	0.72	0.75	0.69	0.65	-16.7%	-19.8%	-9.7%
13:00-13:30	0.86	7.30	0.88	0.84	0.71	0.87	0.80	0.74	-1.1%	-4.8%	4.2%
13:30-14:00	0.87	7.47	0.93	0.89	0.75	0.91	0.84	0.78	-2.2%	-5.6%	4.0%
14:00-14:30	0.94	7.50	0.93	0.89	0.75	0.93	0.86	0.80	0.0%	-3.4%	6.7%
14:30-15:00	0.82	7.03	0.95	0.91	0.76	0.75	0.69	0.64	-21.1%	-24.2%	-15.8%
15:00-15:30	0.89	6.49	0.95	0.91	0.77	0.81	0.74	0.69	-14.7%	-18.7%	-10.4%

6. Discussions and concluding remarks

The numerical prediction of run-up in the presence of sandy beaches was considered, using as case study a beach on the south west coast of Sicily (Italy), where a field measurement campaign was performed. On the basis of wave parameter records in a near offshore buoy, the nearshore wave climate was simulated by using the SWAN model. The obtained wave characteristics were used in order to simulate the run-up by means of a Boussinesq type model.

The comparison between the simulated wave run-up values and the measured data showed that the method provides reasonable results, although the numerical model tends to generally underestimate the run-up. It should be noticed that although a relative error of about 20% may appear large, in absolute terms it amounts only to a difference between the simulated and measured values of about 15 cm. Moreover, it is interesting from the engineering point of view that the relative errors are smaller when the highest value of wave run-up are considered (e.g $R_{1\%}$).

A possible cause of the recovered mismatch could be the way input data are transferred to the Boussinesq model, i.e. by means of the TMA spectrum, rather than the actual SWAN simulated spectrum. Moreover, another source of error could be related to the fact that, due to the lack of onshore wave data, it was not possible to perform a calibration of the model parameters, e.g of the friction coefficient.

Future developments of the research will be focused on the run-up assessment obtained by considering as input wave signals measured in situ at intermediate water depths, just near the beach. This should allow to eliminate the inevitable approximations introduced by the geographic transposition of offshore significant wave data and by the assumption of a specific wave spectrum (e.g. TMA).

Acknowledgements

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