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Large-amplitude long wave interaction with a vertical wall

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Abstract

The rigorous solution of the nonlinear shallow-water equations for the wave field near a vertical wall is derived; it is used to calculate the height of the water oscillations near the wall versus the height of the incident wave. This solution is used to find the exceedance probability of characteristics of the wave motion near the vertical wall. It is shown that the wave height and trough distributions of water oscillations near vertical structures are less than the Rayleigh distribution. Beside of this, the distribution of the crest heights exceeds the Rayleigh distribution (using normalization of the amplification factor for such geometry within the framework of the linear theory), and this explains the high frequency of freak wave events on breakwaters and coasts.

Keywords: Water waves; Nonlinear shallow-water equations; Wave statistics

1. Introduction

The freak wave phenomenon is usually discussed for waves in seas and oceans far from the shores, and a large collection of ship accidents has been accumulated up to now [1,2]. For the last 10 years a lot of instrumental records and also satellite images of unusual waves have been collected; see references above. These data confirm the existence of the rogue wave phenomenon when large-amplitude "monsters" appear on the sea surface for short time. According to the observations, the shapes of the rogue wave can be very different: from single crest to group of large-amplitude waves ("three sisters"); they may have very steep front accompanied by long troughs. The physical mechanisms of the freak wave phenomenon in the open sea discussed in literature are: 1) the spatial-temporal focusing due to wave generation in the wind field varied in space and time; 2) the wave-current interaction in the vicinity of large oceanic streams; 3) the nonlinear self-modulation of the wave field (Benjamin–Feir instability); see for instance, review papers and books [1–6].

Such unusual waves are observed also in the coastal zone. Perhaps, the paper by Sand et al. [7] was the first, where freak wave records obtained in the North Sea on depth 20 m are given. Paper by Chien et al. [8] reported 140 freak wave events (about 500 dead persons) in the coastal zone of Taiwan for 50 years (1949–1999). Most of these events occurred near the shore. Excellent photos of freak waves on rock coasts are given by Palmer [9], when a freak wave

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reached height of 25 m approximately 4 sec after it was visible near the coast (Vancouver Island, Canada). A freak wave attacked the breakwater in Kalk Bay (South Africa) on April 21, 1996 and August 26, 2005 [10]. In both events the freak wave washed off the breakwater people, some of them were injured. The freak waves induced panic at Maracas Beach (Trinidad Island, Lesser Antilles) on October 16, 2005, when a series of towering waves, many more than 25 feet high (maximal height of 8 m), flooded the beach, carried sea-bathers, venders and lifeguards, running for their lives. Photos and description of this event can be found in [11]. Some other descriptions of freak waves in 2005 can be found in [12]. About 30 coastal freak wave events have been recorded in 2006 [13].

The physical mechanisms of the freak wave formation in deep water remain to work in coastal waters also, but some of them are modified. In particular, the Benjamin–Feir instability appears in shallow water for almost transversal perturbations of the wave field meanwhile for deep water it appears for longitudinal perturbations. The wave field contains strong coherent components and can be presented as the nonlinear superposition of solitary (solitons), cnoidal and breaking (plunging and spilling breakers) waves. Their interaction may generate narrow "spots" of large-amplitude freak waves. The bottom relief plays a significant role in spatial (geometric) interference of the waves, resulting in the formation of random focus and caustic points, where the wave field is amplified. Particularly, these mechanisms are discussed in review papers; see for instance [5].

The wave amplification of water waves in the coastal zone is well-known. It means that probability of large-amplitude waves is increasing in the coastal zone. The main goal of our study is to investigate the probability of freak waves (tails of the distribution function) on the background high-amplitude coastal waves. The present paper deals with one of the nonlinear effect in the coastal zone when the wave comes near vertical walls (rocks, breakwaters, vertical structures) and is reflected from it. The wave dynamics near a vertical wall like also the violent wave overtopping can be described in the framework of the shallow water equations [14]. The rigorous solution of the nonlinear shallow water equations is derived in Section 2. It is used to obtain the algebraic relations between the amplitude of the water oscillations at the wall and the amplitude of the incident wave. The conditions of applicability of the analytical solution are discussed. The transformation of the distribution function of the wave amplitude near a vertical wall is discussed in Section 3 based on the rigorous solution. It is demonstrated that the distribution functions of the crest amplitudes and trough amplitudes are transforming on different ways; the probability of highest crests exceeds the Rayleigh prediction while the probability of the deepest troughs is less. The present results are summarized in the conclusion.

2. Nonlinear theory of wave interaction with the vertical wall

Let us consider the simplified geometry of the coastal zone as it is shown in Fig. 1. The wave comes near the vertical wall located at $x = 0$ from the left. For simplicity here, the incident wave is presented as a single long crest, but then we will consider the incident wave as a continuous function representing random crests and troughs.

The basic equations for water waves in shallow water are

$$\frac{\partial \eta}{\partial t} + \frac{\partial}{\partial x}\big[(h+\eta)u\big] = 0,$$
$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial \eta}{\partial x} = 0, \tag{2.1}$$

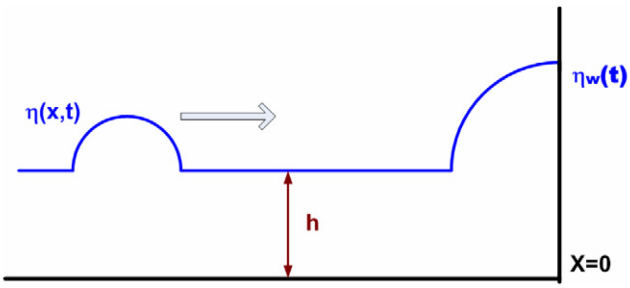

Fig. 1. Definition sketch of the considered geometry.

where $\eta(x,t)$ is the vertical displacement of the sea level, $u(x,t)$ is the depth averaged velocity of the water flow, g is the gravity acceleration and h is the unperturbed water depth assumed to be constant. The boundary condition on the vertical wall corresponds to the total reflection of wave energy and no penetration of fluid through the wall

$$u(x=0,t)=0. \tag{2.2}$$

Another condition which concerns the approach of the incident wave to the wall from the left will be described later.

To solve Eqs. (2.1), it is convenient to introduce the Riemann invariants

$$I_{\pm}=u\pm 2\left[\sqrt{g(h+\eta)}-\sqrt{gh}\right], \tag{2.3}$$

and re-write system (2.1) in the following form

$$\frac{\partial I_{\pm}}{\partial t}+c_{\pm}\frac{\partial I_{\pm}}{\partial x}=0, \tag{2.4}$$

where the characteristic speeds are

$$c_{\pm}=\pm\sqrt{gh}+\frac{3}{4}I_{\pm}+\frac{1}{4}I_{\mp}. \tag{2.5}$$

According to Eqs. (2.4) each invariant remains constant along the characteristics' curves

$$\frac{\mathrm{d}I_{\pm}}{\mathrm{d}t}=0 \quad \text{along} \quad \frac{\mathrm{d}x}{\mathrm{d}t}=c_{\pm}, \tag{2.6}$$

and corresponds to the progressive wave. Notice that the slopes of the characteristics depend from both invariants; they are shown in Fig. 2. In the linear theory the characteristics are straight lines everywhere (they are given by dashed lines). The nonlinearity bends the characteristics in the vicinity of the wall area where the incident and reflect waves interact (shaded area). Taking into account the conservation of the Riemann invariants, the effect of the wave interaction yields to phase corrections in the travel times of various parts of the wave profile. As a result, the water displacement at the vertical wall $\eta_w(t)=\eta(x=0,t)$ depends from the incident wave in a very complicated manner and cannot be presented in an explicit form. Nevertheless the relation between values of the wave in the incident wave field and in the near-wall water oscillations can be derived explicitly. Outside of the interaction near-wall zone the incident and reflected waves propagate independently. The incident wave (propagated on the right) is characterized by $I_-=0$ and from (2.3) follows

$$u=2\left[\sqrt{g(h+\eta)}-\sqrt{gh}\right], \qquad I_+=4\left[\sqrt{g(h+\eta)}-\sqrt{gh}\right]. \tag{2.7}$$

Due to the boundary condition (2.2) the incident invariant at the wall is

$$I_+=2\left[\sqrt{g(h+\eta_w)}-\sqrt{gh}\right]. \tag{2.8}$$

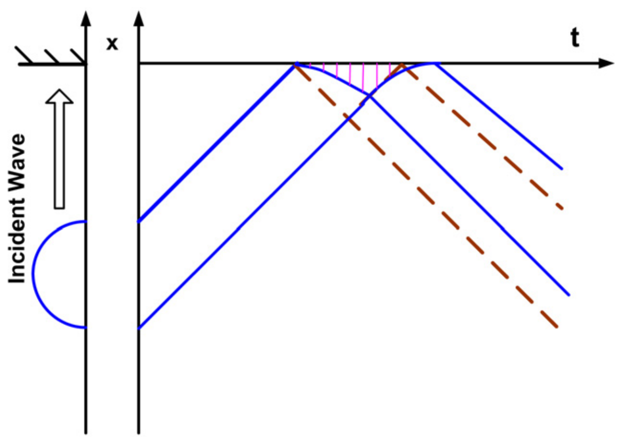

Fig. 2. Characteristics patterns at the reflection from the wall.

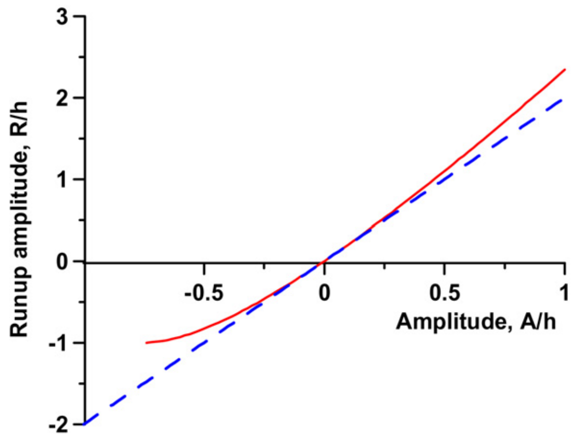

Fig. 3. Amplitude of water oscillations at the wall versus the incident wave amplitude.

As it follows from the conservation of I_+ along the characteristics the expressions (2.7) and (2.8) should be equal, hence

$$\frac{\eta_w(t)}{h} = 4\left[1 + \frac{\eta(t-\tau)}{h} - \sqrt{1 + \frac{\eta(t-\tau)}{h}}\right], \tag{2.9}$$

where τ is the phase shift between the incident and reflected waves. Formula (2.9) is obtained using the quadratic operation with roots in the Riemann invariants, thus, the "one-digit" solution exists if

$$\frac{\eta}{h} > -\frac{3}{4}. \tag{2.10}$$

So, the water level on the wall can be expressed through the water displacement of the incident wave. Unfortunately, this method cannot predict the time shift τ between the incident and reflected waves, which is generally the unknown functional from the wave field in the interaction zone.[1] It is why it is not easy to use expression (2.9) for the calculation of water level oscillations near the vertical wall even if all the characteristics of the incident wave are given. Meanwhile, a practical formula can be derived from (2.9) – it is the relation between the extreme values of the wave field

$$\frac{R}{h} = 4\left[1 + \frac{A}{h} - \sqrt{1 + \frac{A}{h}}\right], \tag{2.11}$$

where A is a positive or negative amplitude (crest or trough height) of the incident wave (it should also exceed $-3h/4$), and R is an amplitude of the water level oscillations. This relation is displayed in Fig. 3 by red solid line.[2] For comparison, note that the linear theory gives the following relation

$$R = 2A \tag{2.12}$$

which corresponds to blue dashed line. As it can be seen, the nonlinearity increases the crest height and decreases the trough height at the wall. In fact, the weak increasing of the wave height, when the positive wave (crest) comes near the wall, has been analyzed earlier by [17,18]. A more interesting case occurs when the negative wave (trough) comes near the wall. The nonlinear effects become significant when the total depth tends to zero. The algebraic solution (2.11) exists only if the trough amplitude is less than $3h/4$, or total depth under the trough greater than $h/4$.

Formally, the process of the wave interaction with the vertical wall is considered above for the wave of solitary shape (pulse) of any polarity, but really this assumption is not necessary. The expression (2.11) can be obtained for arbitrary function $\eta(t)$, finite or continuous, if its shape is smooth enough. As an example, we may calculate the height

[1] The same problem appears at the solitary wave collision [15,16], but here the time shift can be expressed explicitly through the ratio of incident soliton amplitude to water depth.

[2] For interpretation of the references to color, the reader is referred to the web version of this article.

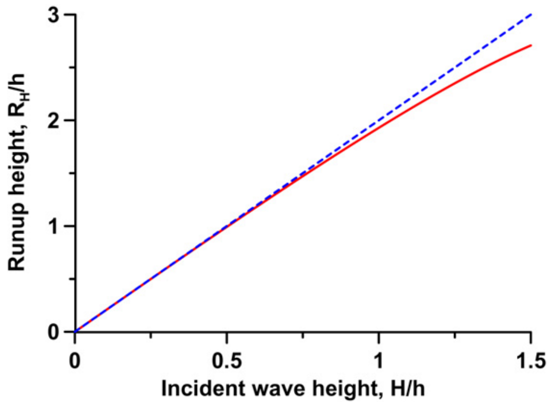

Fig. 4. Height of water oscillations at the wall versus the incident wave height.

of the water oscillations at the wall, if we assume that the incident wave is symmetrical about the horizontal axis like sine wave ($|A|_\pm = H/2$)

$$\frac{R_H}{h} = 4\frac{H}{h} - 4\sqrt{1+\frac{H}{2h}} + 4\sqrt{1-\frac{H}{2h}}. \tag{2.13}$$

This expression is displayed in Fig. 4 by solid red line. The linear relation $R_H = 2H$ is represented also by dashed blue line. As it can be seen, the difference between the linear and nonlinear functions is relatively weak if $H < h$.

The conditions of application of the derived relation between the amplitude of the water oscillations at the wall and the incident wave amplitude should be clarified through the exact solution of the governing equations (2.1). Some of them can be obtained from a physical point of view using particular rigorous solutions. Firstly, the wave breaking can occur when the wave comes near the vertical wall before its interaction with the reflected wave. This process can be investigated from the analytical solution for the incident wave derived from (2.1); see, for instance [19–21]

$$\eta(x,t) = \eta_0\left[t - \frac{x}{c(\eta)}\right], \qquad c = 3\sqrt{g(h+\eta)} - 2\sqrt{gh}, \qquad u = 2\left[\sqrt{g(h+\eta)} - \sqrt{gh}\right], \tag{2.14}$$

where $\eta_0(t)$ is the shape of the incident wave, for instance generated by a paddle in wave tank. The necessary condition of smooth wave shape for the wave propagated along the right direction [21] is

$$\frac{\eta}{h} > -\frac{5}{9} \tag{2.15}$$

(positiveness of the propagation speed, c), and this condition gives the limited amplitude of the wave trough less than (2.10).

The second limitation is the infinite steepness of the initially smoothed wave shape with distance predicted from (2.14). For instance, if the generated wave is a sine wave with frequency, ω, the wave breaking occurs at the distance, which is a function of the wave amplitude through the following parametric equations [21]

$$kX = \frac{2\sqrt{1+\zeta}(3\sqrt{1+\zeta}-2)^2}{3\sqrt{a^2/h^2-\zeta^2}}, \qquad \frac{A}{h} = \sqrt{\frac{\zeta^2(3\sqrt{1+\zeta}+2) - 2\zeta(3\sqrt{1+\zeta}-2)}{9\sqrt{1+\zeta}-2}}, \tag{2.16}$$

where $k = \omega/(gh)^{1/2}$ is the wave number of the sine wave according to the linear approximation and $\zeta = \eta_*/h$, η_* is the water level of the breaking point. Solution of (2.16) is shown in Fig. 5. If the wave amplitude tends to the critical value (2.15), the wave breaks near the paddle and does not propagate. For small and moderate wave amplitudes, the breaking distance becomes large, and the wave propagates to the vertical wall with no breaking. The condition (2.16) should be used for the planning of laboratory experiments or estimation of observation of wave breaking effects in the coastal zone.

The third condition for the incident wave amplitude can be found considering the initial phase of wave reflection from the wall. The reflected wave near the wall propagates in the field of the incident wave, and if the wave has a large

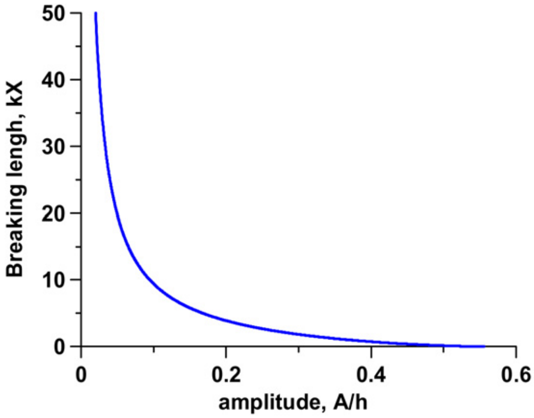

Fig. 5. Breaking length of the initial monochromatic wave versus the wave amplitude.

amplitude, the induced current can block the propagation of the reflected wave. This condition can be obtained from the definition of the characteristic speed of weak reflected wave

$$c_- = -\sqrt{gh} + \frac{I_+}{4} \leqslant 0, \tag{2.17}$$

and using (2.7) it follows

$$A \leqslant 3h. \tag{2.18}$$

So, from these several physical conditions using the rigorous solution, we can conclude that the analytical expression (2.11) is valid at least for smooth incident wave if the crest amplitude is less than $3h$, and trough amplitude less than $5h/9$. These criteria are obtained from the shallow-water theory, where waves can have either amplitude from weak to large, and formally the wave amplitude can exceed water depth if it is smooth enough. With application to the "real" sea waves which are relatively short, only the case $A < h$ is interesting. The shallow water theory does not include wave dispersion which is important on intermediate depth. In the framework of the nonlinear-dispersive theory, the steady-state waves (cnoidal or solitary waves) have limited height $H = 2A < 0.78h$. According to many laboratory data, where the role of dispersion is important, the wave height is limited to $0.55h$; see, for instance [22]. It is that we will use in the next section, the criterion for the significant wave height/depth, $H_s/h <$ 0.5–0.7 closed to criteria of other authors.

3. Distribution functions of the wave amplitudes

Distribution functions of wave characteristics are important for ocean and coastal engineering to design offshore and coastal structures. Simplified model for the wave height distribution uses the Rayleigh distribution based on the Gaussian description of narrow-banded linear ocean waves (see, for instance, [23]). Due to wave nonlinearity (finite wave steepness) and non-narrowness of wave spectrum (broad-banded spectrum), the wave height distribution differs from the Rayleigh distribution, leading to an increase of the probability of crest heights and decrease – trough heights; see for instance, the book by Massel [23] and last papers [24–27]. As we show the nonlinear effects will be essential for the distribution functions of the wave field near a vertical wall.

The approach applied above is valid for any incident wave as regular, as well as irregular due to wave separation along the characteristics. In the latter case, it can be used to analyze distribution functions of the wave field and its spectrum. Unfortunately, it cannot predict the time shift between the incident wave and water oscillations at the wall, and, therefore, the function $\eta_w(\tau)$ is not fully determined within the framework of the nonlinear theory. The process is not Gaussian due to nonlinearity, and all moments cannot be calculated, including the significant height of water oscillations at the wall. Meanwhile, the relation between the random wave amplitudes of the incident wave and water oscillations at the wall (2.11) is explicit and does not include the time shift. Hence, if the distribution function of the wave amplitude of the incident wave field is known, expression (2.11) can be used to obtain the distribution function of the amplitude of the water oscillations at the wall. The non-inertial ("instant") transformation of random processes is described in several books; see, for instance [23]. The probability density function of the amplitudes of the near-wall water oscillations is

$$W_R(R) = W_A(A)\left|\frac{\mathrm{d}A}{\mathrm{d}R}\right|\Bigg|_{A(R)}, \tag{3.1}$$

where $W_A(A)$ is the probability density function of amplitudes of the incident wave, and $A(R)$ is the inverse function to (2.11) which is calculated explicitly

$$\frac{A}{h} = \frac{R}{4h} + \frac{1}{2}\left[\sqrt{1+\frac{R}{h}} - 1\right]. \tag{3.2}$$

Beside the probability density function, the exceedance probability function

$$P_A(A) = \int_A^{+\infty} W_A(b)\,\mathrm{d}b \tag{3.3}$$

is used in practice for ocean engineering purposes. Using (3.1) the exceedance probability function of the amplitude of the water oscillations at the wall can be determined explicitly also

$$P_R(R) = P_A(A)|_{A(R)}. \tag{3.4}$$

For detailed calculations, the exceedance probability function of the incident wave should be specified. Below, the Rayleigh distribution for wave heights is used (indexes of distribution functions will be omitted in following formulas)

$$P(H) = \exp\left(-\frac{H^2}{8\sigma^2}\right) = \exp\left(-\frac{2H^2}{H_s^2}\right), \tag{3.5}$$

where the significant wave height, $H_s = 4\sigma$, and σ^2 is the variance of the initial Gaussian field. In fact, the wave field in shallow water (as well in deep water) is not Gaussian [23,28], but for simplicity we will use the assumption of the narrow-banded Gaussian process resulting in the Rayleigh distribution for the wave heights. For quasi-monochromatic wave $H = 2A$, and (3.5) can be re-written as the amplitude distribution

$$P(A) = \exp\left(-\frac{2A^2}{A_s^2}\right), \tag{3.6}$$

where $A_s = H_s/2$. We would like to emphasize that any empirical wave distribution can be used below, and the Rayleigh distribution (3.6) is used here only for simplicity and demonstration of the role of the nonlinear effects near the vertical wall.

Using the relations (3.2) and (3.4), the exceedance probability functions of the positive (crest) and negative (trough) amplitudes of the water oscillations at the vertical wall can be determined explicitly

$$P(R_+) = \exp\left\{-\frac{2}{A_s^2}\left[\frac{R_+}{4} + \frac{1}{2}\left(\sqrt{h+R_+} - h\right)\right]^2\right\}, \tag{3.7}$$

$$P(R_-) = \exp\left\{-\frac{2}{A_s^2}\left[\frac{R_-}{4} - \frac{1}{2}\left(\sqrt{h-R_-} - h\right)\right]^2\right\}, \tag{3.8}$$

where both amplitudes (heights of the crests and troughs) have positive values. For convenience of the graphic presentation of the distribution function, the amplitudes of the water oscillations at the wall will be normalized by $H_s = 2A_s$, taking into account that the "wall" amplitude in the linear theory is twice the amplitude of the incident wave. In this case all deviation from the Rayleigh distribution will characterize the nonlinear effects. Main definition parameter here is

$$\varepsilon = \frac{H_s}{h}, \tag{3.9}$$

which is a natural nonlinear parameter of the shallow-water theory.

Fig. 6 displays the exceedance probability function of the crest heights of the water oscillations at the wall for different values of the parameter ε from 0 (linear case) to 0.7 (large-amplitude waves). As it is expected, weak and

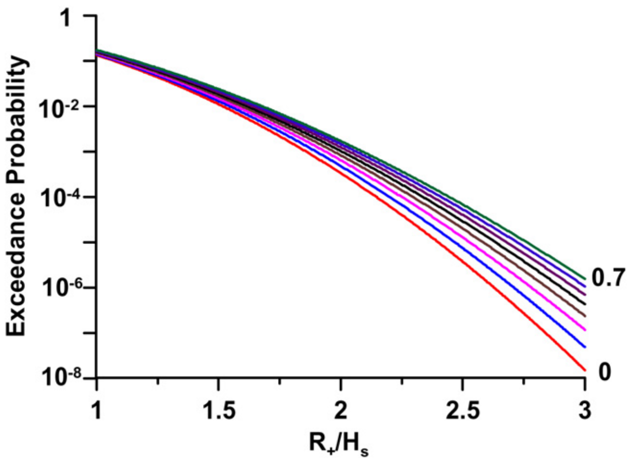

Fig. 6. Exceedance probability function of crest heights of water oscillations at the wall. Numbers on curves – values of $\varepsilon = H_s/h$ with increment $\varepsilon = 0.1$.

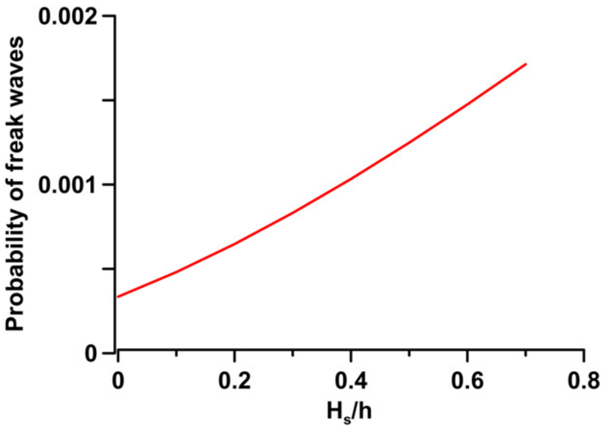

Fig. 7. Probability of freak crest occurrence, $P(R_+ = 2H_s)$ versus H_s/h.

moderate water oscillations have almost the same Rayleigh distribution as the incident wave, but their crest heights are twice the incident wave amplitudes (this factor is included by normalization of the crest heights on incident significant wave height, not on significant wave amplitude). For extreme waves, including freak waves (their amplitude exceeds twice or more the significant wave height) the probability of the large crests is increased with increase of the ratio of significant wave height to water depth. This is illustrated by Fig. 7, where the probability of wave occurrence with heights $R_+/H_s \geqslant 2$ versus H_s/h is presented. As we can see, the probability of freak waves increases with H_s/h almost linearly. From Figs. 6 and 7 it can be concluded that anomalous high crests will occur in the coastal zone more often than in the open sea, and this effect is related with nonlinear mechanism of wave transformation in the coastal zone. Such waves will overflow through breakwaters and flood the coasts, causing the accidents described in Introduction.

Exceedance probability of the trough heights (negative amplitudes) at the vertical wall calculated from (3.9) is shown in Fig. 8 for different values of $\varepsilon = H_s/h$. As it has been mentioned above, the incident wave heights should be not too high, or troughs will reach the seafloor. It is why we could not consider large values of ratio H_s/h. Generally, the probability of occurrence of the deepest troughs near the wall is less than the Rayleigh prediction, and, therefore, freak waves should often have the shape of crests than that of troughs.

Using (2.18) the distribution function of the heights of water oscillations at the vertical wall can be calculated; it is displayed in Fig. 9. The nonlinearity decreases the probability of the highest waves compared with the Rayleigh

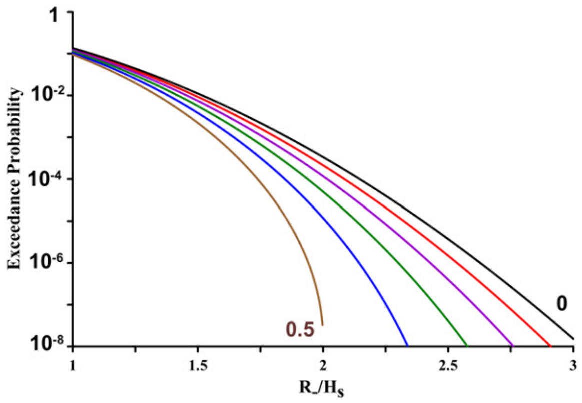

Fig. 8. Exceedance probability function of trough heights of water oscillations at the wall. Numbers on curves – values of $\varepsilon = H_s/h$ with increment 0.1.

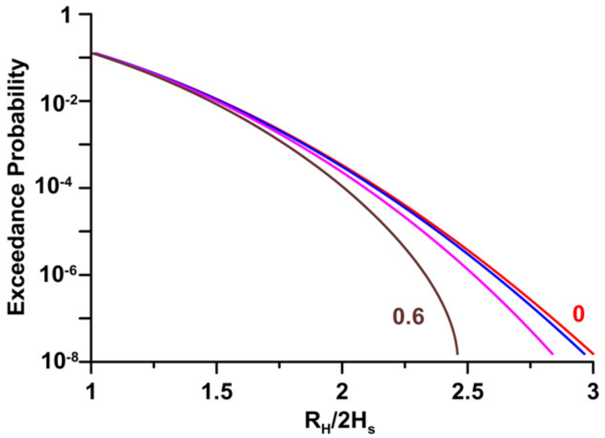

Fig. 9. Exceedance probability function of heights of water oscillations at the wall for several values of $\varepsilon = H_s/h$ with increment 0.2.

distribution. It means that the probability to meet unusual high waves for ships and boats near rocks and breakwaters is less than in the open sea, but the shallow water waves may be significantly steeper.

So, we have shown that nonlinearity changes the exceedance probability of the wave characteristics near vertical walls in shallow waters. Qualitatively, the variations of the distribution functions are the same as for deep water (increase of the crest height and decrease of the trough height probabilities), but herein the effect is related to the interaction with the vertical wall.

The process of the violent wave overtopping was studied experimentally in the Edinburgh wave flume, and also modeled in the framework of the nonlinear shallow-water equations [14]. In these experiments, a near-vertical sloping structure is used, and, therefore, it is difficult to compare with our model. The latter authors compared the numerical results based on the shallow-water theory and laboratory data and claimed that the numerical model over-predicts the wave heights due to the lack of dispersion within shallow-water theory. Both, laboratory and numerical models showed a good agreement with the expected Rayleigh distribution in the range of probability from 1 to 10^{-2}. For highest waves the probability of the wave heights is decreased; see, Fig. 7 from this paper. It is in qualitative agreement with our prediction for the wave heights; see Fig. 9 above.

4. Conclusion

The nonlinear wave transformation in shallow water is studied for a simplified geometry of the coastal zone bounded by a vertical wall. The rigorous solution of the nonlinear shallow water equations is derived and the analytical expression for the height of the water oscillation near the wall is found. It is valid for regular waves and irregular waves as well. This solution is used to compute the distribution functions of the characteristics of the water oscillations at the vertical wall. It is shown that the exceedance probability of the heights of the large-amplitude waves is less than that predicted by the Rayleigh distribution. This is in qualitative agreement with the laboratory experiments conducted in the Edinburgh wave flume for near-vertical slopping structures, and also with the results of direct numerical simulations in the framework of the nonlinear shallow-water theory [14]. Thus, the probability for ships and boats to meet unusual waves with huge heights near rocks and breakwaters is less than in the open sea. Meanwhile the exceedance probability of crest heights is higher than that given by the Rayleigh distribution. Therefore, freak wave events corresponding to unusual and short-lived overtopping of breakwaters or flooding on the coasts, have a large probability. Perhaps, this explains why the accidents on breakwaters and coasts described in Introduction occur very often.

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