



*Citation for published version:*

Gao, J, Ma, X, Chen, H, Zang, J & Dong, G 2021, 'On hydrodynamic characteristics of transient harbor resonance excited by double solitary waves', *Ocean Engineering*, vol. 219, 108345.  
<https://doi.org/10.1016/j.oceaneng.2020.108345>

*DOI:*

[10.1016/j.oceaneng.2020.108345](https://doi.org/10.1016/j.oceaneng.2020.108345)

*Publication date:*

2021

*Document Version*

Peer reviewed version

[Link to publication](#)

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1 On hydrodynamic characteristics of transient harbor resonance excited by double  
2 solitary waves

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12  
13 **Abstract:**

14 The harbor resonance triggered by double solitary waves (DSWs) with different wave  
15 parameters (including various wave heights and relative separation distances) is simulated based on  
16 the fully nonlinear Boussinesq model, FUNWAVE-TVD. A long and narrow harbor with different  
17 topographies is adopted. In the current study, effects of incident wave height, relative separation  
18 distance and bottom profile on hydrodynamic characteristics related to the transient oscillations are  
19 mainly investigated. The hydrodynamic characteristics considered include the evolution of the  
20 maximum free-surface elevation, the maximum runup, the wave energy distribution and the total  
21 wave energy inside the harbor. Results show that Green's law can accurately estimate the evolution  
22 of the maximum free-surface elevation in most part of the harbor area. The impacts of the  
23 topography on the maximum runup exhibit a strong dependence on the incident wave height. The  
24 smaller mean water depth inside the harbor, the larger relative separation distance, and the higher  
25 incident wave height tend to result in greater uniformity of the wave energy distribution. The  
26 normalized total wave energy is always shown to decrease gradually with the incident wave height,  
27 and to increase remarkably at first and then decrease slightly with the increase of the mean water  
28 depth.

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1 **Key Words:**

2 Harbor resonance; Oscillations; Double solitary waves; Numerical simulation; Topographic effects;  
3 FUNWAVE-TVD

4

5 **1. Introduction**

6 Harbor resonance is resulted from the gathering and magnification of incident wave energy at  
7 bays or harbors when the incident wave frequencies approach or equate to their eigenfrequencies  
8 (Gao et al., 2016a, 2020). Common triggering factors for the phenomenon include low-frequency  
9 waves, short wave groups, tsunamis, barometric variation, vessel-induced bores, local water-surface  
10 disturbance, and shear flow (Dong et al., 2013, 2020b; De Jong et al., 2003; Ma et al., 2020; Rupali  
11 et al., 2020; Fabrikant, 1995; Gao et al., 2017c, 2018b, 2018c, 2019c; Shao et al, 2020; Wang et al.,  
12 2020). Harbor oscillations has direct negative impacts on ship movement and port management. It  
13 can excite excessive movements of ships and unacceptable forces on mooring lines, and cause port  
14 operations poor efficient (Kumar et al., 2016).

15 Among the above-mentioned triggering factors, transient harbor resonance events excited by  
16 tsunamis have been often observed and many of them were devastating. As tsunamis travel into the  
17 coast, their wave heights usually rise dramatically owing to the persistent decline of the water depth.  
18 Take the Indian Ocean tsunami happening on December 26, 2004, for example. It was generated by  
19 the Sumatra earthquake and travelled for ~2 hours to Colombo Harbor in Sri Lanka, inducing  
20 remarkable resonance with the eigenperiod of ~1.25 hours and the maximum oscillation of ~3.9 m.  
21 It then travelled for ~14 hours to Bunbury Harbor in Australia, producing the maximum oscillation  
22 of ~1.8 m (Pattiaratchi and Wijeratne, 2009). Hence, to minimize both the interference to port  
23 operations and potential devastating impacts, further studies on hydrodynamic characteristics  
24 associated with harbor resonance are of vital importance to enhance related knowledge and thus  
25 improve our predictive ability.

26 Solitary waves have been widely adopted to represent real tsunamis in lots of studies due to  
27 the fact that some characteristics in real tsunami events, such as the persistent hump-like waveforms,  
28 can be imitated by solitary waves very well (e.g., Craig et al. (2006); Dong et al. (2020a);  
29 Hayatdavoodi et al. (2014); Liu et al. (1995); Seiffert et al. (2014); Synolakis (1987); Wang and Liu  
30 (2020)). However, based on some in-situ surveys which proved that the leading tsunamis was often

1 preceded by a depression, Tadepalli and Synolakis (1994) proposed the so called “N-waves” to  
2 represent tsunamis for the first time, and this type of tsunami waveform was further developed by  
3 Madsen and Schäffer (2010). Undular bores have also been frequently seen in the coast as tsunamis  
4 approach the shoreline (Madsen et al., 2008), and their evolutions in the coast can generate a train  
5 of multi-crested waves that are regarded as a train of solitary waves with various wave parameters  
6 (El et al., 2012; Grilli et al., 2012). The concept of double solitary waves (DSWs) was also proposed  
7 by a few scholars in the last few years (Dong et al., 2014; Lo et al., 2013).

8 The majority of the harbor-oscillation investigations triggered by tsunamis were performed via  
9 utilizing solitary waves or N-waves (e.g., Dong et al. (2010; 2020a); Gao et al. (2019a; 2019b)). On  
10 the other hand, the wave effects in the coast related to DSWs are basically confined to predicting  
11 their runup/backwash on beaches (e.g., Dong et al. (2014); Lo et al. (2013)). To the best of our  
12 knowledge, up to now, the studies of the transient harbor oscillations induced by DSWs are rare.  
13 Gao et al. (2018a) introduced DSWs for the first time to carry out the research on transient resonance  
14 inside an elongated harbor with a flat bottom, and the influences of various waveform parameters  
15 (including the incident wave height, the number of component solitary waves, and the relative  
16 separation distance between adjacent crests) on the relative wave energy distribution inside the  
17 harbor were investigated. Except Gao et al. (2018a), no more literature on the harbor resonance  
18 triggered by DSWs has been seen by the authors.

19 The present work also utilizes DSWs to further investigate the hydrodynamic characteristics  
20 associated with the transient oscillations. This article focuses on the impacts of different incident  
21 wave parameters and the bottom profile inside the harbor on various hydrodynamic characteristics  
22 during oscillations. The resonant hydrodynamics concerned include the evolution of the maximum  
23 free-surface elevation, the maximum runup, the wave energy distribution, and the total wave energy.  
24 It needs to be emphasized that it is of crucial importance to accurately estimate and comprehensively  
25 study these hydrodynamic characteristics. Both the evolution of the maximum free-surface elevation  
26 and the maximum runup are tightly correlated with the tsunami-triggered flood around the harbor  
27 area (e.g., Kumar and Gulshan (2017, 2018); Gao et al. (2016b; 2019b)); both the wave energy  
28 distribution and the total wave energy offer vital information for accurately predicting the movement  
29 of ships in harbors (Gulshan et al., 2020).

30 Compared with Gao et al. (2018a) where the transient harbor resonance triggered by DSWs

1 has been studied, the research progresses of this article are embodied in three respects. Firstly, Gao  
2 et al. (2018a) assumed the constant water depth inside the harbor. Nevertheless, for real harbors,  
3 their topographies are generally uneven (Gulshan et al., 2020; Kumar and Rupali, 2018). Therefore,  
4 in this article, more complicated bottom profiles are taken into consideration. Secondly, the wave  
5 heights of the DSWs adopted in Gao et al. (2018a) were relatively small; the wave condition was  
6 confined to the weak nonlinearity. On the contrary, the current study expands the wave fields in the  
7 harbor to the strong nonlinearity. Thirdly, the investigations on both the evolution of the total wave  
8 energy and the maximum free-surface elevation inside the harbor triggered by the DSWs are  
9 implemented in this article for the first time. In current research, all numerical cases are performed  
10 utilizing a fully nonlinear Boussinesq model. The harbor is assumed to be long and narrow; the  
11 motion of the free surface inside the harbor then substantially becomes one-dimensional (Gao et al.,  
12 2016c; Losada et al., 2008).

13 The remaining parts of this article is as follows. the numerical model and the data analysis  
14 methodology are first introduced in Section 2. Then, the setups associated with the numerical  
15 experiments are described in Section 3. Simulation results are analyzed and explained in Section 4.  
16 Main conclusions are summarized in Section 5.

17

## 18 **2. Numerical model and data analysis methodology**

### 19 2.1. Numerical model

20 In the current study, all numerical experiments are performed utilizing the proverbial numerical  
21 model, FUNWAVE-TVD (Shi et al., 2012). The set of fully nonlinear Boussinesq equations  
22 proposed by Chen (2006) combined with a moving reference level are utilized as the control  
23 equations that are numerically discretized by using a hybrid finite-difference/finite-volume scheme.  
24 To carry out a high-order Total Variation Diminishing (TVD) scheme easily, the control equations  
25 are reorganized as in Ma et al. (2012) and Liu et al. (2020). To parallelize the numerical model, the  
26 Message Passing Interface (MPI) technique with non-blocking communication is introduced into  
27 the model as well. In addition, an adaptive time-stepping algorithm is adopted, which follows the  
28 Courant–Friedrichs–Lewy (CFL) criterion.

29 With these enhancements mentioned above, the numerical model becomes more robust and  
30 more powerful in predicting various hydrodynamic processes associated with water waves in the

1 offshore and coastal areas. These processes include wave shoaling, diffraction, refraction, breaking,  
2 and wave runup/backwash on beaches (Liu et al., 2019; Ning et al., 2019b). To validate the  
3 capability of the numerical model in simulating the harbor resonance excited by transient long waves  
4 (e.g., tsunamis), the physical experiments of Dong et al. (2010) were fairly well reproduced in Gao  
5 et al. (2017a) by using FUNWAVE-TVD. It should be stressed in particular that the capacity of  
6 FUNWAVE-TVD to predict the wave profile evolution of tsunamis (e.g., solitary waves) over  
7 uneven topographies has been fully validated by Ning et al. (2019a) via comparing with  
8 experimental data, which ensures the correctness of the research results about the topographic  
9 influences on the harbor oscillations triggered by DSWs in this article.

## 11 2.2. Data analysis methodology

12 In order to acquire the total wave energy and the wave energy distribution for the harbor  
13 resonance excited by DSWs accurately, a data analysis methodology, namely, the normal mode  
14 decomposition (NMD) method, needs to be adopted.

15 The NMD method was developed by Sobey (2006) to estimate eigenmodes, eigenfrequencies,  
16 and response amplitudes of various modes for harbors suffered from tsunamis. The technique  
17 includes two calculation steps. In the 1<sup>st</sup> step, a series of eigenmodes and corresponding  
18 eigenfrequencies of the harbor are estimated. In the 2<sup>nd</sup> step, the prediction of the response  
19 amplitudes at various resonant modes are performed, which is expressed as a multi-dimensional  
20 optimization issue. Both the eigenmodes and the eigenfrequencies estimated in the former step are  
21 treated as known parameters in this process. Then, this technique was further ameliorated in Gao et  
22 al. (2015) to estimate eigenmodes and eigenfrequencies more precisely.

23 Based on the method, Gao et al. (2019a; 2019b) have comprehensively investigated the wave  
24 energy distribution aiming at the harbor resonance triggered by N-waves and solitary waves. For its  
25 detailed theory, the interested reader is referred to Gao et al. (2015) and Sobey (2006).

## 27 3. Numerical setups

### 28 3.1. Incident wave parameters

29 If a solitary wave propagates along the  $x$ -axis over a flat bottom, its leading-order wave surface  
30 is expressed as follows (Mei, 1983):

1 
$$\eta(x, t) = A_0 \operatorname{sech}^2 [k(x - ct)], \quad (1)$$

2 where

3 
$$k = \sqrt{\frac{3A_0}{4h^3}}, \text{ and } c = \sqrt{g(A_0 + h)}. \quad (2)$$

4  $A_0$  and  $c$  are the wave height and the traveling speed of the solitary wave, respectively, and  $h$  is the  
5 water depth.

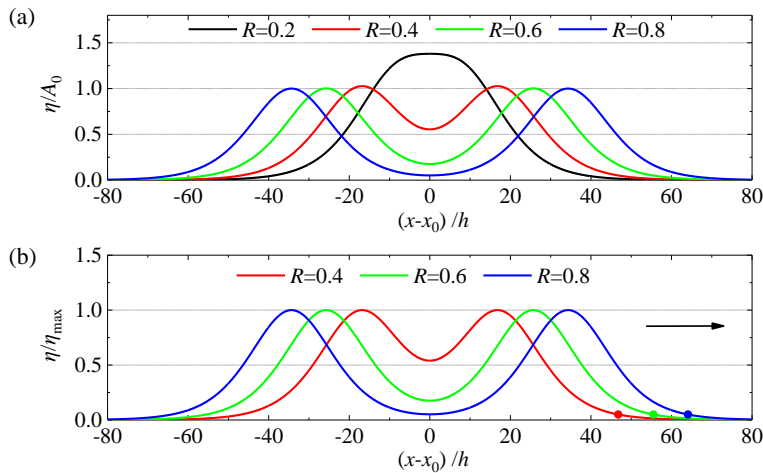
6 Similar to Dong et al. (2014), in this article, the incident DSWs are created by the linear  
7 superposition of two component solitary waves with identical wave heights and different relative  
8 separation distances. Therefore, the incident DSWs adopted in all the numerical experiments can be  
9 formulated as

10 
$$\eta(x, t) = A_0 \left[ \operatorname{sech}^2 (k(x - x_0 - ct - R \cdot L / 2)) + \operatorname{sech}^2 (k(x - x_0 - ct + R \cdot L / 2)) \right], \quad (3)$$

11 where  $x_0$  is the central location of the initial waveform, and  $L = 2\pi/k$  can be viewed as the effective  
12 wavelength of the component solitary wave.  $R$ , defined as the ratio of the distance between adjacent  
13 crests to  $L$ , is the so-called “*relative separation distance*”. In FUNWAVE-TVD, the initial velocity  
14 of the incident DSWs takes a linear expression as following:

15 
$$u(x, t = 0) = \sqrt{\frac{g}{h}} \eta(x, t = 0). \quad (4)$$

16



17

18 **Fig. 1.** Waveforms of the DSWs under conditions of  $A_0=0.100$  m and  $h=14.0$  m: (a) waveforms with  
19  $R$  varying from 0.2 to 0.8 with an interval of 0.2; (b) the definition of the wavefront. The circle and  
20 the arrow in figure (b) indicate the wavefront and the direction of wave propagation, respectively.

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For all the simulations considered in the present research, a constant water depth of 14.0 m is arranged outside the harbor. Fig. 1 presents the comparisons of the waveforms for the DSWs with  $A_0=0.100$  m and various relative separation distances under the condition of  $h=14.0$  m. It is seen that when  $R=0.2$ , the two component single waves fuse into one wave. As  $R$  rises to 0.4, there appear two distinctly-separated crests in the waveform, and their height,  $\eta_{\max}$ , has already become extremely close to the wave height of the component solitary wave,  $A_0$ , that is,  $\eta_{\max}=1.026A_0$ . Therefore, only the DSWs with  $R \geq 0.4$  are considered in this article.

**Table 1.** Incident wave parameters for DSWs and topographic parameters for the harbor

Group	Initial waveform	$A_0$ (m)	Type of topography	topographic parameters
A		0.025,	arc-tangent-type	$h_1=14$ m, $h_0=4$ m, $\alpha=6.8$ m, $\bar{h}=12.43$ m
B		0.050,		$h_1=14$ m, $h_0=4$ m, $\alpha=8.3$ m, $\bar{h}=10.73$ m
C	DSWs with $R=0.4$	0.075, 0.1,	constant slope	$h_1=14$ m, $h_0=4$ m, $\bar{h}=9.00$ m
D		0.15, 0.2,	hyperbolic-cosine-type	$h_1=14$ m, $h_0=4$ m, $\kappa=0.15$ , $\bar{h}=7.69$ m
E		0.25, 0.3		$h_1=14$ m, $h_0=4$ m, $\kappa=200$ , $\bar{h}=6.54$ m
F		0.025,	arc-tangent-type	$h_1=14$ m, $h_0=4$ m, $\alpha=6.8$ m, $\bar{h}=12.43$ m
G		0.050,		$h_1=14$ m, $h_0=4$ m, $\alpha=8.3$ m, $\bar{h}=10.73$ m
H	DSWs with $R=0.6$	0.075, 0.1,	constant slope	$h_1=14$ m, $h_0=4$ m, $\bar{h}=9.00$ m
I		0.15, 0.2,	hyperbolic-cosine-type	$h_1=14$ m, $h_0=4$ m, $\kappa=0.15$ , $\bar{h}=7.69$ m
J		0.25, 0.3		$h_1=14$ m, $h_0=4$ m, $\kappa=200$ , $\bar{h}=6.54$ m
K		0.025,	arc-tangent-type	$h_1=14$ m, $h_0=4$ m, $\alpha=6.8$ m, $\bar{h}=12.43$ m
L		0.050,		$h_1=14$ m, $h_0=4$ m, $\alpha=8.3$ m, $\bar{h}=10.73$ m
M	DSWs with $R=0.8$	0.075, 0.1,	constant slope	$h_1=14$ m, $h_0=4$ m, $\bar{h}=9.00$ m
N		0.15, 0.2,	hyperbolic-cosine-type	$h_1=14$ m, $h_0=4$ m, $\kappa=0.15$ , $\bar{h}=7.69$ m
O		0.25, 0.3		$h_1=14$ m, $h_0=4$ m, $\kappa=200$ , $\bar{h}=6.54$ m

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The incident wave parameters for the DSWs utilized in all numerical experiments are listed in Table 1. Fifteen groups of numerical experiments (namely, Groups A-O) are taken into consideration, and each consists of eight cases with  $A_0$  varying from 0.025 m to 0.300 m to systematically investigate the effects of the wave height on the resonant hydrodynamic characteristics. The DSWs with  $R=0.4$ , 0.6, and 0.8 are used as the incident waves in Groups A-E, F-J, and K-O, respectively. The main purpose of designing Groups F-O lies in finding out how the relative separation distance affects the transient harbor oscillations triggered by DSWs.



1 In order to facilitate describing the setups of the NWT that will be shown in Subsection 3.2 and  
2 interpreting some research results that will be shown in Section 4, the concepts of “*wavefront*” and  
3 “*wavelength*” for the DSWs are defined here. The wavefront refers to the location at which the free-  
4 surface elevation equates to 0.05 times  $\eta_{\max}$  (refer to Fig. 1b). In this paper, the wavelength of the  
5 DSWs,  $L_0$ , is defined as double the length between the wavefront and  $x_0$ . For the same  $A_0$ , the  
6 wavelength of the DSWs increases gradually with  $R$ . Table 2 further lists all the wavelengths of the  
7 incident DSWs with various wave heights and relative separation distances adopted in the present  
8 study. For the three waveforms of DSWs shown in Fig. 1b, their wavelengths gradually increase  
9 from 1310.2 m when  $R=0.4$  to 1794.6 m when  $R=0.8$ . Besides, for the DSWs with the same relative  
10 separation distance, their wavelengths decrease gradually with the increase of  $A_0$ . Take the DSWs  
11 with  $R=0.4$  for example. Their wavelengths decrease from 2620.0 m to 756.0 m as  $A_0$  rises from  
12 0.025 m to 0.300 m.

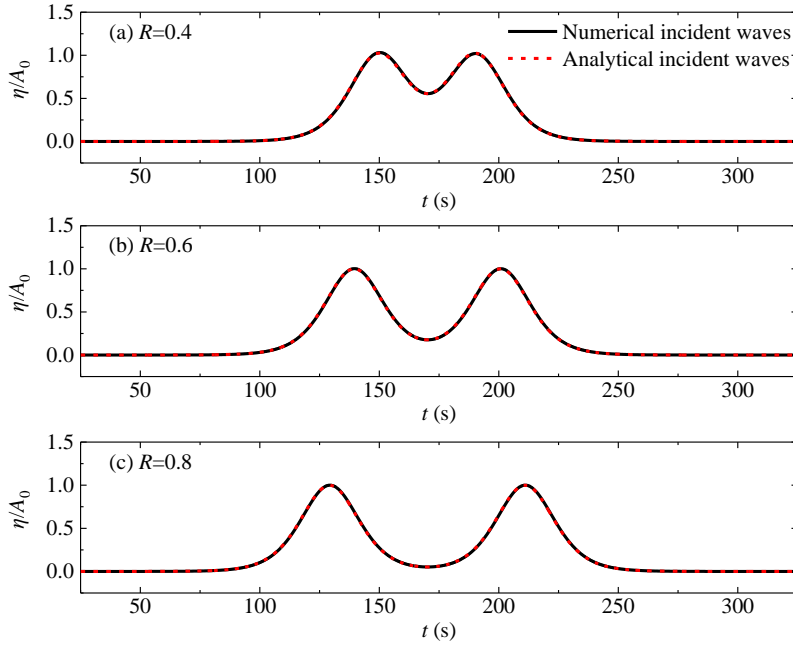
13  
14 **Table 2.** Wavelengths,  $L_0$ , of the DSWs with various relative separation distances and wave heights.

15 The unit of the wavelength is meter.

$A_0$ (m)	0.025	0.050	0.075	0.100	0.150	0.200	0.250	0.300
$L_0$ when $R=0.4$	2620.0	1852.8	1512.6	1310.2	1069.8	926.4	828.6	756.0
$L_0$ when $R=0.6$	3108.0	2198.0	1794.2	1554.0	1268.8	1099.0	982.6	897.2
$L_0$ when $R=0.8$	3590.0	2538.0	2072.0	1794.6	1465.4	1269.0	1135.0	1036.2

16  
17 Prior to performing the numerical experiments of the current study, it is necessary to check out  
18 the ability of the numerical model to generate DSWs. For this purpose, a simple numerical wave  
19 tank (NWT) is established (not shown in the paper). The NWT has a length of 8000 m, and the still  
20 water depth  $h$  for the whole computational domain is set to 14.0 m. The generated waves are set to  
21 propagate from left to right. The coordinate origin (i.e.,  $x=0$ ) is positioned at the left boundary. At  
22 the moment of  $t=0$ , the central location of the waveform is set to  $x_0=2000$  m. To record the incident  
23 wave train, a wave gauge is placed at  $x=4000$  m. The uniform grid size of 1.0 m is adopted. The  
24 comparisons of the simulated and the analytical incident DSWs with  $A_0=0.100$  m and various  
25 relative separation distances are presented in Fig. 2. The incident DSWs generated by the model are  
26 shown to be in good agreement with the analytical ones.

1



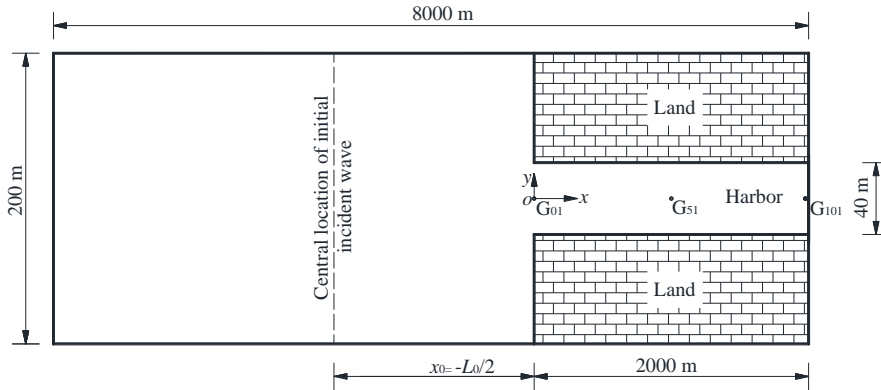
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3 **Fig. 2.** Comparisons of the simulated and the analytical incident DSWs with  $A_0=0.100$  m and various  
 4 relative separation distances.

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6 3.2. Numerical wave tank

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9 **Fig. 3.** Top view of the NWT used in all simulations.

10

11 Fig. 3 presents the NWT used in all simulations. The dimensions of the NWT and the harbor  
 12 are  $8000 \text{ m} \times 200 \text{ m}$  and  $2000 \text{ m} \times 40 \text{ m}$ , respectively. All the boundaries are fully reflective. There  
 13 are 101 wave gauges (i.e.,  $G_{01} - G_{101}$ ) deployed along the central line of the harbor, and the space  
 14 between any two adjacent gauges is 20 m.  $G_{01}$  is arranged at the harbor entrance ( $x=0$ ), and  $G_{101}$  is  
 15 at the backwall ( $x=2000$  m). The water depth outside the harbor is set to a constant of  $h_1=14.0$  m.

1 The grid sizes are uniform in 1 m along both the  $x$ - and the  $y$ -axes. As the initial condition, the  
 2 wavefronts of the incident waves are always set to be located at the entrance (i.e.,  $x_0$  is always set to  
 3  $-L_0/2$ ) at the beginning of the simulation. The total simulation time for all cases is set to 800 s. In  
 4 the adaptive time-stepping algorithm of FUNWAVE-TVD, one parameter  $C$ , termed as *Courant*  
 5 *number*, needs to be presetted, which is set to 0.5 for all cases. The output time interval for the free-  
 6 surface elevations at all gauges is set to 0.04 s.

7 To study the impacts of the topographic variation inside the harbor on the resonant  
 8 hydrodynamic features induced by DSWs, three types of bottom profiles (including a hyperbolic-  
 9 cosine-type bottom, a plane slope bottom, and an arc-tangent-type bottom) are adopted in this paper.

10 The water depth inside the harbor,  $h_d(x)$ , is formulated as (Gao et al., 2017b):

$$11 \quad h_d(x) = \begin{cases} h_0 \cosh^\kappa [\mu(L-x)] & \text{hyperbolic-cosine-type bottom} \\ h_1 - \gamma x & \text{constant-slope bottom} \\ h_0 + \alpha \operatorname{atan}[\beta(L-x)] & \text{arc-tangent-type bottom} \end{cases}, \quad (5)$$

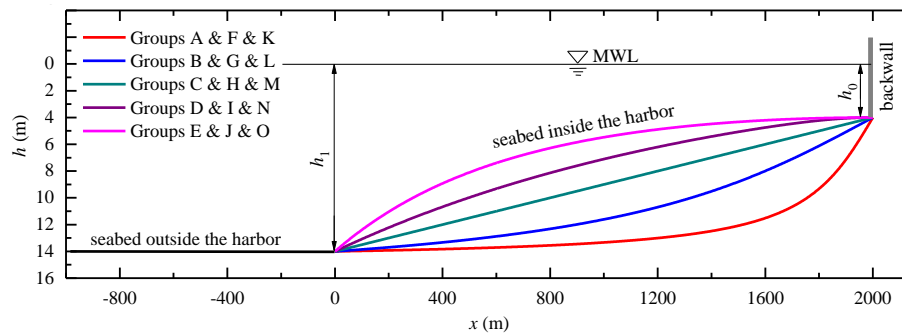
12 where  $h_0$  denotes the water depth at the backwall and is set to 4.0 m.  $\alpha, \beta, \gamma, \kappa$  and  $\mu$  are topographical  
 13 parameters, and comply with the following relationships:

$$14 \quad \beta = \frac{1}{L} \tan\left(\frac{h_1 - h_0}{\alpha}\right), \gamma = \frac{h_1 - h_0}{L}, \text{ and } \mu = \frac{1}{L} \operatorname{acosh}\left[\left(\frac{h_1}{h_0}\right)^{1/\kappa}\right]. \quad (6)$$

15 The types of the bottom profiles and the values of the topographic parameters used in all groups are  
 16 also presented in Table 1. In this table,

$$17 \quad \bar{h} = \frac{1}{b} \int_0^L h^1(x) dx, \quad (7)$$

18 which denotes the mean water depth inside the whole harbor.

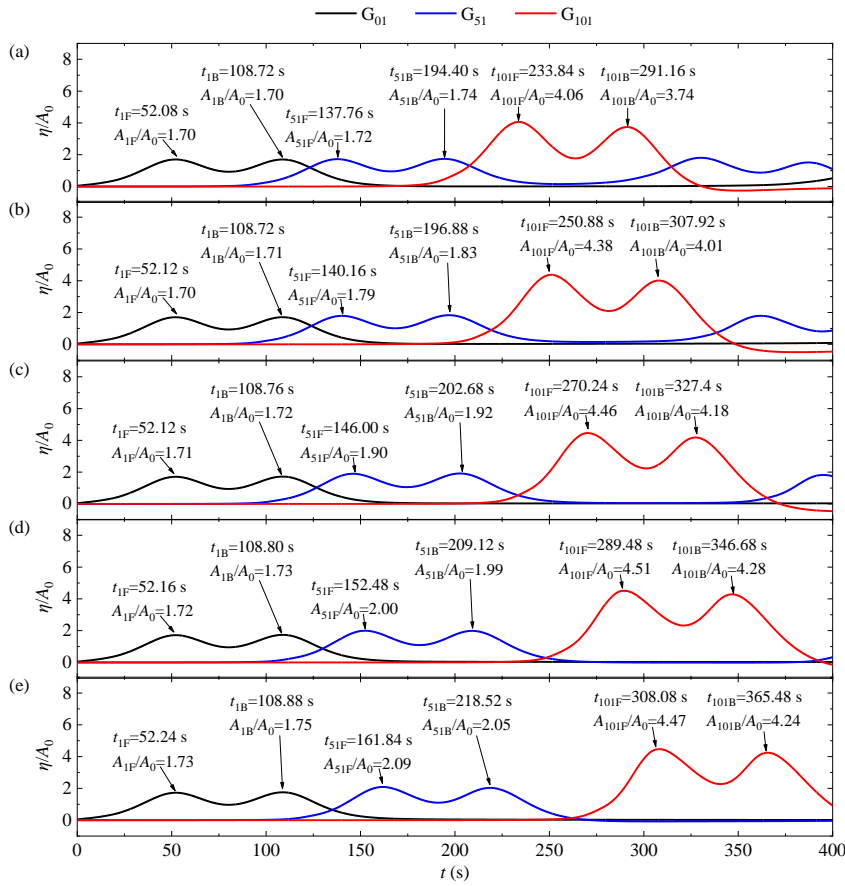


20  
 21 **Fig. 4.** Comparisons of various topographies inside the harbor

22  
 23 Fig. 4 illustrates all the five topographies adopted in the present study. The harbor utilized in

1 Groups A, F, and K has the arc-tangent-type topography, and the mean water depth  $\bar{h}$  is 12.43 m.  
 2 The harbor in Groups B, G and L has the arc-tangent-type bottom profile as well. Nevertheless,  $\bar{h}$   
 3 declines to 10.73 m due to a larger magnitude of  $\alpha$ . The harbor in Groups D, I, and N has the  
 4 hyperbolic-cosine-type topography, and  $\bar{h}$  is 7.69 m. Because of the increased  $\kappa$ ,  $\bar{h}$  in Groups E,  
 5 J, and O decreases to 6.54 m. The harbor in Groups C, H, and M has a constant-slope bottom, and  
 6 has  $\bar{h}=9.00$  m.

7



8

9 **Fig. 5.** Time histories of the free-surface elevations at various wave gauges under conditions of  
 10  $A_0=0.050$  m and  $R=0.4$ . (a)-(e) correspond to Groups A-E, respectively.

11

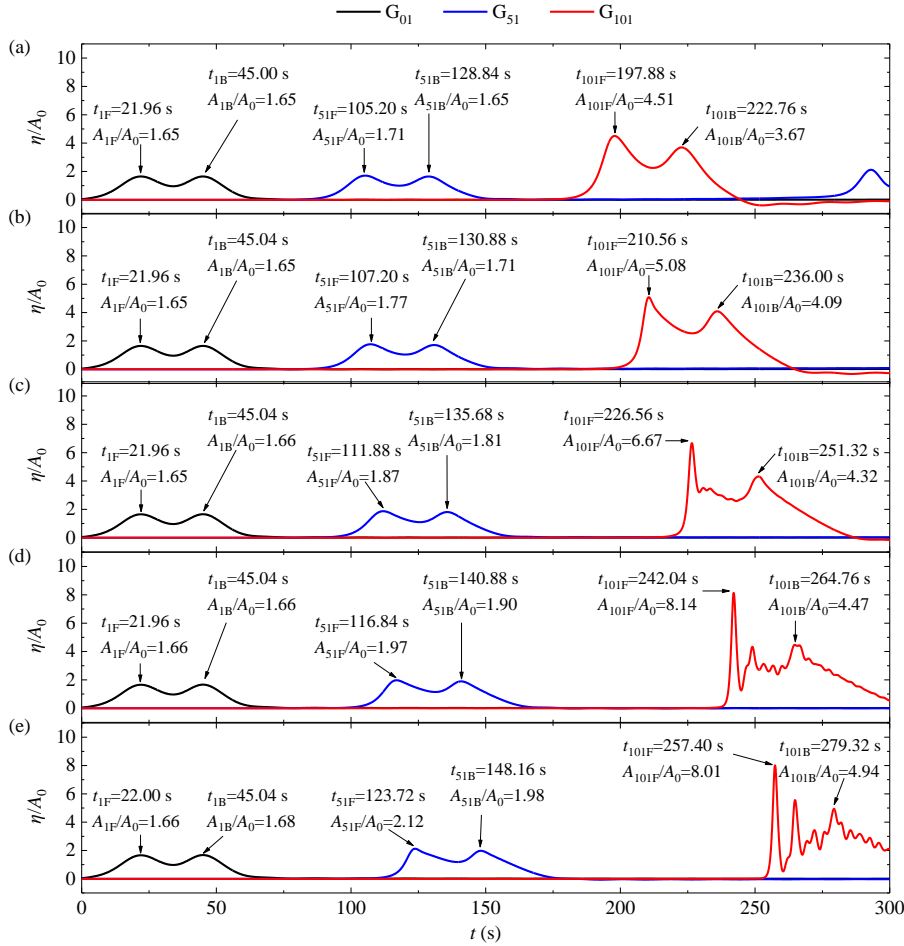
## 12 4. Results and discussion

### 13 4.1. Time histories of free-surface elevations

14 Fig. 5 illustrates the time histories of the free-surface elevations at gauges  $G_{01}$ ,  $G_{51}$ , and  $G_{101}$   
 15 for the five cases with  $A_0=0.050$  m and  $R=0.4$  in Groups A-E.  $A_{1F}$  and  $A_{1B}$  in the figure  
 16 respectively denote the wave-crest elevations of the front and the back component solitary waves at

1 the  $i^{\text{th}}$  gauge. The five cases have the same incident waves but various bottom profiles (refer to Table  
 2 1). Three phenomena can be seen from the figure. Firstly, for all these cases, the wave-crest  
 3 elevations of the front component solitary waves at gauges  $G_{01}$  and  $G_{51}$  are almost equal to the  
 4 corresponding ones of the back component solitary waves. However, for gauge  $G_{101}$ , the value of  
 5  $A_{101F}$  becomes remarkably larger than that of  $A_{101B}$ . Secondly, because the local water depth declines  
 6 continuously as the incident DSWs travel from the entrance to the backwall, the maximum  
 7 elevations at all the three gauges increase gradually. Thirdly, the wave-crest elevations at gauges  
 8  $G_{51}$  and  $G_{101}$  gradually rise with the decline of the mean water depth inside the harbor, overall. It  
 9 can be attributed to the fact that the smaller mean water depth leads to the more obvious shoaling  
 10 effect (i.e., the more remarkable wave height amplification).

11



12

13 **Fig. 6.** Time histories of the free-surface elevations at various wave gauges under conditions of  
 14  $A_0=0.3$  m and  $R=0.4$ . (a)-(e) correspond to Groups A-E, respectively.

15

16 Fig. 6 further demonstrates the time histories of the free-surface elevations at gauges  $G_{01}$ ,  $G_{51}$ ,

1 and  $G_{101}$  for the five cases with  $A_0=0.300$  m and  $R=0.4$  in Groups A-E. Compared to Fig. 5, some  
2 different phenomena appear. Although the wave-crest elevations of the front component solitary  
3 wave still keep almost identical to the ones of the back component solitary wave at gauge  $G_{01}$ , the  
4 former becomes notably larger than the latter at gauge  $G_{51}$ . In addition, because of the larger incident  
5 wave height, the free-surface elevations at gauges  $G_{51}$  and  $G_{101}$  present remarkable wave  
6 nonlinearity, which is specifically embodied in two respects. Firstly, apparent asymmetry can be  
7 seen in the front component solitary wave at gauges  $G_{51}$  and  $G_{101}$ . For the front component solitary  
8 waves at gauge  $G_{101}$  in Fig. 6a and b and that at gauges  $G_{51}$  in Fig. 6e, their free surfaces rise  
9 promptly from zero to the maximum elevations, while their paces of decline are much slower.  
10 Secondly, for gauge  $G_{101}$  shown in Fig. 6c-e, except the two wave crests generated directly by the  
11 incident DSWs, there still exist one or more secondary peaks between the two wave crests. In  
12 general, the smaller mean water depth inside the harbor and the larger incident wave height lead to  
13 the more significant wave nonlinearity.

14

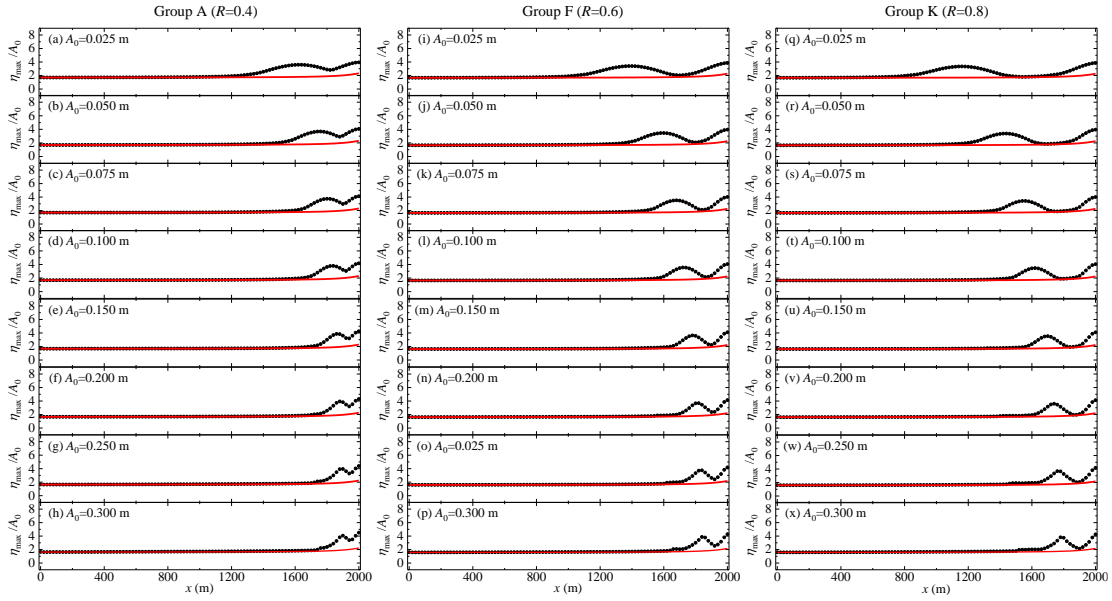
#### 15 4.2. Evolution of the maximum free-surface elevation

16 Considering that the maximum free-surface elevation is tightly related to the tsunami-triggered  
17 flood around the harbor area, its evolution inside the harbor is comprehensively investigated in this  
18 subsection as the incident DSWs propagates from the harbor entrance to the backwall. Via reading  
19 the time histories of the free surfaces at all gauges, the maximum elevation evolution inside the  
20 harbor can be directly acquired. Fig. 7 illustrates the evolution of the maximum free-surface  
21 elevation for Groups A, F and K under different incident wave heights. The three groups have the  
22 maximum mean water depth inside the harbor, 12.43 m. Because the local water depth continuously  
23 decreases as the incident DSWs travel to the backwall, the maximum free-surface elevation  
24 gradually increases as expected, except at the area where the incident and the reflected waves  
25 interact with each other near the backwall. Based on the linear wave theory and the assumption that  
26 the wave energy due to the reflection is neglected, the wave height of the long waves travelling over  
27 mild slopes can be estimated theoretically using Green's law (Mei, 1983):

$$28 \quad \frac{A}{A_\Delta} = \left( \frac{h_\Delta}{h} \right)^{1/4}, \quad (8)$$

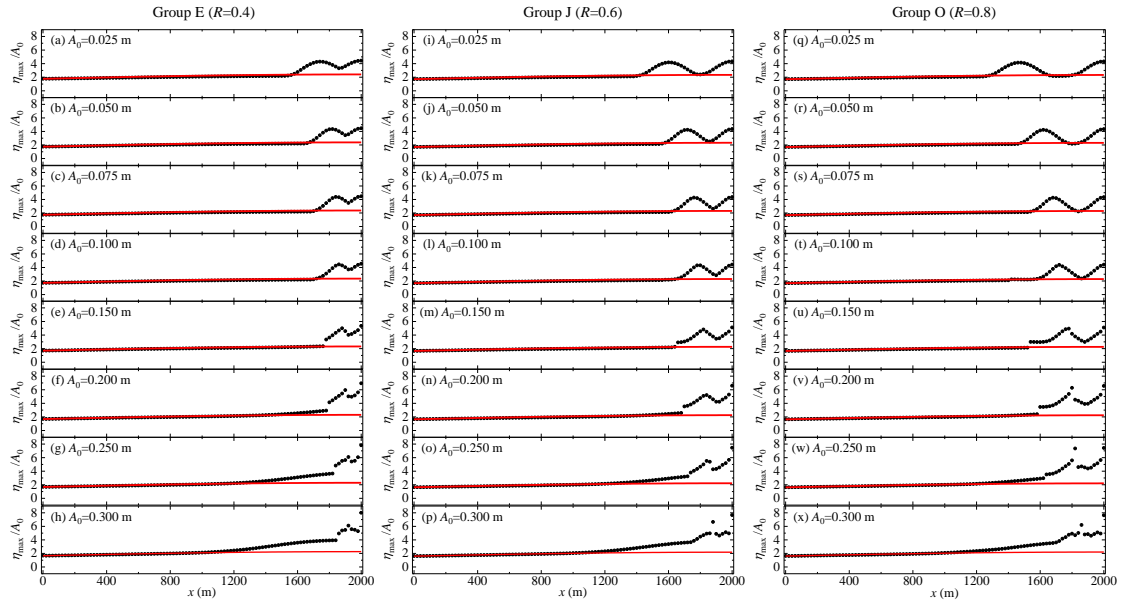
29 where  $A_\Delta$  refers to the wave height at a certain reference position, and  $h_\Delta$  refers to the local water  
30 depth there. The evolution of the maximum free-surface elevation estimated by Green's law is

1 presented in this figure. It is noted here that the reference position for the calculation of Eq. (8) is  
 2 set at gauge  $G_{02}$  rather than at gauge  $G_{01}$  to remove the boundary effect caused by the harbor entrance.



3  
 4 **Fig. 7.** Evolutions of the maximum free-surface elevations for Groups A, F and K. Black dots and  
 5 red curves denotes the maximum free-surface elevations obtained directly by the simulations and  
 6 those predicted by Green's law, respectively.

7  
 8 It is seen from this figure that for all these cases, Green's law predicts the numerical results  
 9 very well inside the whole harbor except at the area where the incident and the reflected waves  
 10 interact with each other near the backwall. In addition, it can be easily observed that for a given  
 11 value of  $R$ , the spatial range where Green's law is valid gradually increases with the increase of the  
 12 incident wave height. Similarly, it can be found that for a given value of  $A_0$ , the valid spatial range  
 13 of Green's law also increases gradually with the decrease of  $R$ , no matter whether  $A_0$  is large or  
 14 small. They are because the interaction area of the incident and the reflected waves contracts  
 15 gradually as  $A_0$  increases and as  $R$  decreases, which coincides with the dependency of the incident  
 16 wavelength on these two parameters (refer to Table 2).



**Fig. 8.** Evolutions of the maximum free-surface elevations for Groups E, J and O. The meanings of black dots and red curves are identical to those in Fig. 7.

Fig. 8 further shows the evolution of the maximum free-surface elevation for Groups E, J and O under various incident wave heights. Different from Fig. 7, all three groups in this figure have the smallest mean water depth, 6.54 m. Similar to Fig. 7, Green's law is also shown to predict the maximum elevation accurately in most part of the harbor except at the small area around the backwall, and its valid spatial range increases gradually with the decline of  $R$  as well.

However, because of the decline of the mean water depth, a different phenomenon can be observed from Fig. 8. As the incident wave height increases, the valid spatial range of Green's law does not monotonically increase any more. Instead, the valid spatial range of Green's law becomes to first increase and then remarkably decrease with the rise of the incident wave height, which can be attributed to the secondary peak phenomenon shown in Fig. 6. As mentioned above, the larger incident wave height and the smaller mean water depth tend to cause the more significant wave nonlinearity (including the occurrence of the secondary peak phenomenon). The secondary peak can result in the narrower spatial span of the front wave crest, and the wave energy there becomes more concentrated. Hence, it results in the remarkable increase of the maximum free-surface elevation there. Through examining the free-surface elevations in these cases (the related results are not shown here), it is found that the secondary peak phenomenon begins to appear when  $A_0=0.150$  m and becomes more and more evident as  $A_0$  further increases. It is accordance with the phenomenon



1 in Fig. 8 that the valid spatial range of Green's law increases notably as  $A_0$  rises from 0.150 m to  
 2 0.300 m.

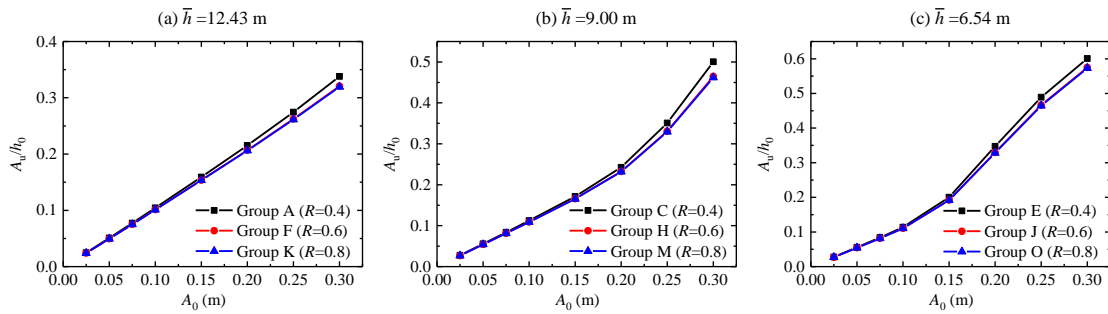
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#### 4 4.3. Maximum runup

5 Observing Figs. 7 and 8 can find that the maximum runup of the incident DSWs always occurs  
 6 at the backwall, no matter whether the mean water depth, the relative separation distance, and the  
 7 incident wave height are large or small. Fig. 9 shows the normalized maximum runups,  $A_u/h_0$ , for  
 8 the three topographies with  $\bar{h}=12.43$  m, 9.00 m, and 6.54 m. Two apparent phenomena can be seen  
 9 from the figure. Firstly, the maximum runups caused by the incident DSWs with the same incident  
 10 wave height and various relative separation distances are almost identical to each other when  $A_0 \leq$   
 11 0.150 m for all these three topographies. However, as  $A_0$  further increases, the maximum runups  
 12 with  $R=0.4$  gradually becomes larger than those with  $R=0.6$  and 0.8. The maximum runups with  
 13  $R=0.6$  are still almost identical to those with  $R=0.8$ , regardless of the bottom profile inside the harbor  
 14 and the incident wave height.

15 Secondly, although the maximum runups increase monotonously with the increase of  $A_0$  for all  
 16 these three topographies at the variation range of  $A_0$  considered, their growth characteristics closely  
 17 depends on the topography. For the topography with  $\bar{h}=12.43$  m (Fig. 9a), the maximum runup  
 18 seems to always increase linearly with  $A_0$ . For the topography with  $\bar{h}=9.00$  m (Fig. 9b), the  
 19 increasing rate of  $A_u$  seems to remain unchanged when  $A_0 \leq 0.150$  m and then rises gradually as  $A_0$   
 20 increases further. While for the topography with  $\bar{h}=6.54$  m (Fig. 9c), the increasing rate of  $A_u$  first  
 21 remains unchanged when  $A_0 \leq 0.100$  m, then increases, and then decreases with the increase of  $A_0$ .

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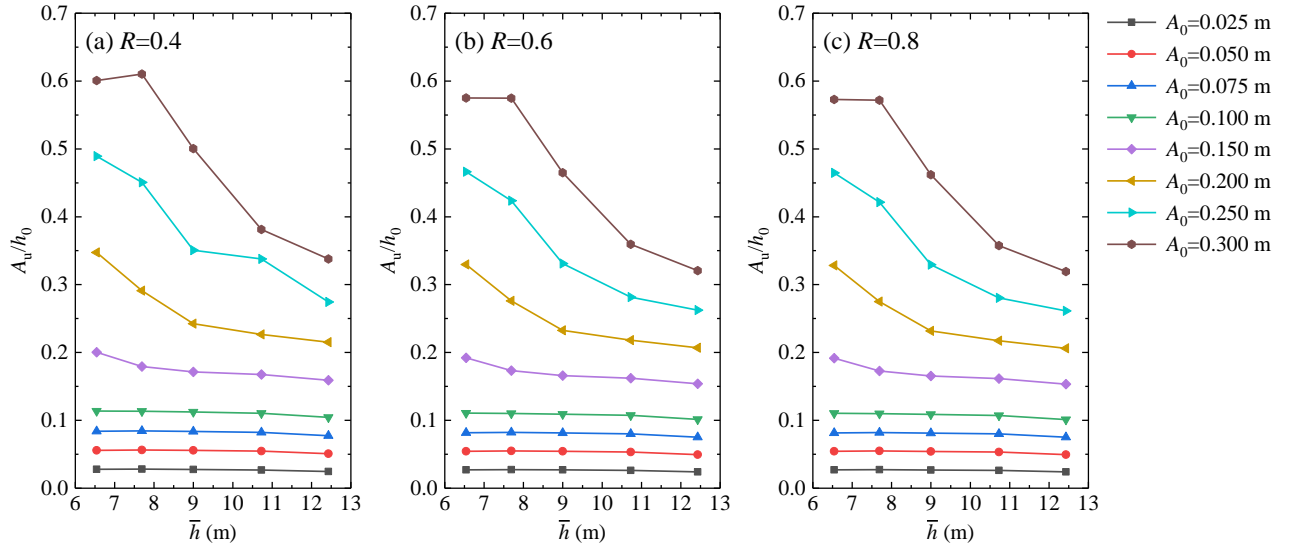


23

24 **Fig. 9.** The normalized maximum runups,  $A_u/h_0$ , for the three topographies with (a)  $\bar{h} = 12.43$  m, (b)

1  $\bar{h} = 9.00$  m , and (c)  $\bar{h} = 6.54$  m .

2



3

4 **Fig. 10.** Variations of the maximum runups with respect to the mean water depth induced by the  
5 DSWs with different values of  $R$ .

6

7 To better explain the impacts of the topography on the maximum runup, Fig. 10 illustrates the  
8 variations of the maximum runups with respect to the mean water depth subjected to the incident  
9 DSWs with different values of  $R$ . It is seen that when  $A_0 \leq 0.100$  m, the topographic variation inside  
10 the harbor has very little effect on the maximum runup. Nevertheless, as  $A_0$  further increases, the  
11 effects of the topographic variation on the maximum runup relies heavily on both the relative  
12 separation distance and the incident wave height. For the incident DSWs with  $R=0.4$  (Fig. 10a),  
13 when  $0.100 \text{ m} \leq A_0 \leq 0.250$  m, the maximum runup decreases monotonously with the increase of  
14  $\bar{h}$ . As  $A_0$  increases to  $0.300$  m, the maximum runup becomes to first increase slightly, and then  
15 decrease sharply with the rise of  $\bar{h}$ . While for the incident DSWs with  $R=0.6$  and  $0.8$  (Fig. 10b and  
16 c), the maximum runup always declines monotonously with the increase of  $\bar{h}$  when  $A_0$  is at the  
17 range of  $0.100 \text{ m} - 0.300$  m.

18

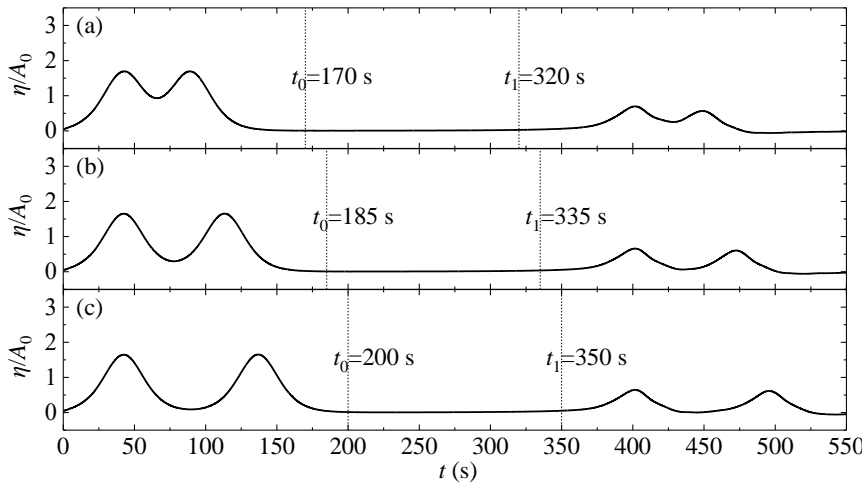
#### 19 4.4. Wave energy distribution and total wave energy

##### 20 4.4.1. Computing process for the response amplitudes of various resonant modes

21 This subsection presents the detailed computing process of the NMD method for extracting the

1 response amplitudes of various modes from the wave fields inside the whole harbor. The time  
 2 histories of the free-surface elevations at gauge G<sub>01</sub> for the three cases with  $A_0=0.075$  m in Groups  
 3 A, F and K are illustrated in Fig. 11. Only the wave fields inside the harbor from  $t_0$  to  $t_1$  are adopted  
 4 to calculate the response amplitudes of various modes.  $t_0$  and  $t_1$  denote the moment that the incident  
 5 DSWs totally travel into the harbor and the moment that the reflected DSWs from the backwall  
 6 begin to travel away from the entrance, respectively. Because the incident DSWs in these three cases  
 7 have different relative separation distances and hence different wavelengths (refer to Table 2), the  
 8 values of both  $t_0$  and  $t_1$  are different from each other. As  $R$  increases from 0.4 to 0.8, the value of  $t_0$   
 9 increases gradually from 170 s to 200 s, and that of  $t_1$  rises gradually from 320 s to 350 s.

10



11

12 **Fig. 11.** Time histories of the free-surface elevations at gauge G<sub>01</sub> for the three cases with  $A_0=0.075$   
 13 m in (a) Group A, (b) Group F, and (c) Group K.

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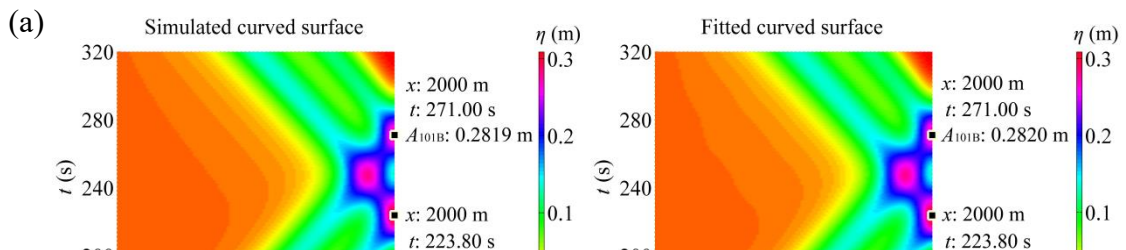
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17 **Fig. 12.** Comparisons of the simulated and the fitted wave fields for the three cases with  $A_0=0.075$   
18 m in (a) Group A, (b) Group F, and (c) Group K.

19

20 Fig. 12 demonstrates the comparisons of the simulated and the fitted wave fields for the three  
21 cases presented in Fig. 11. The simulated wave field is directly acquired via the numerical  
22 simulations, and the fitted one is obtained by the NMD method. Good agreement between them is  
23 observed for all the three cases. Take the case with  $A_0=0.075$  m in Group A for example (Fig. 12a).  
24 The simulated wave field has two maximum runups at  $x=2000$  m. The first one has a value of  
25  $A_{101F}=0.3092$  m at  $t=223.80$  s, and the second one is  $A_{101B}=0.2819$  m at  $t=271.00$  s. At the identical  
26 positions and moments, the fitted wave field has the first and the second maximum runups of 0.3088  
27 m and 0.2820 m, respectively.

28 To assess the fitting accuracy quantitatively, the numerical fitting error (*NFE*) of the NMD  
29 method is defined here (Gao et al., 2018a):

1 
$$NFE = \max \left\{ \frac{|(A_{101F})_{\text{fitted}} - A_{101F}|}{A_{101F}}, \frac{|(A_{101B})_{\text{fitted}} - A_{101B}|}{A_{101B}} \right\} \times 100\% , \quad (9)$$

2 where  $(A_{101F})_{\text{fitted}}$  and  $(A_{101B})_{\text{fitted}}$  respectively denote the fitted first maximum runup and the fitted  
3 second one at the backwall. The *NFEs* for the three cases in Fig. 12a–c are 0.13%, 0.10% and 0.14%,  
4 respectively. Table 3 further presents the *NFEs* for all the cases with  $A_0$  ranging from 0.025 m to  
5 0.100 m. It is seen that the *NFEs* for these cases are all less than 5.00%. It indicates that the data  
6 analysis methodology estimates the response amplitudes of different modes accurately. The  
7 estimated response amplitudes for these cases will be presented in Subsection 4.4.2.

8

9 **Table 3.** Numerical fitting errors (*NFEs*) of the NMD method for all the cases with  $A_0=0.025$  m,  
10 0.050 m, 0.075 m, and 0.100 m. The unit of the *NFE* is %.

$A_0$ (m)	Group														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
0.025	0.14	0.05	0.18	0.17	0.38	0.12	0.11	0.18	0.10	0.28	0.10	0.13	0.38	0.20	0.19
0.050	0.16	0.32	0.72	1.37	2.24	0.31	0.14	0.74	1.32	2.29	0.26	0.20	0.65	1.23	1.79
0.075	0.13	0.85	1.99	2.65	3.24	0.10	0.72	1.78	3.07	3.70	0.14	0.75	1.75	2.83	4.32
0.100	0.53	2.24	2.61	3.50	4.44	0.29	2.10	2.96	4.53	4.70	0.39	1.86	4.00	4.80	4.98

11

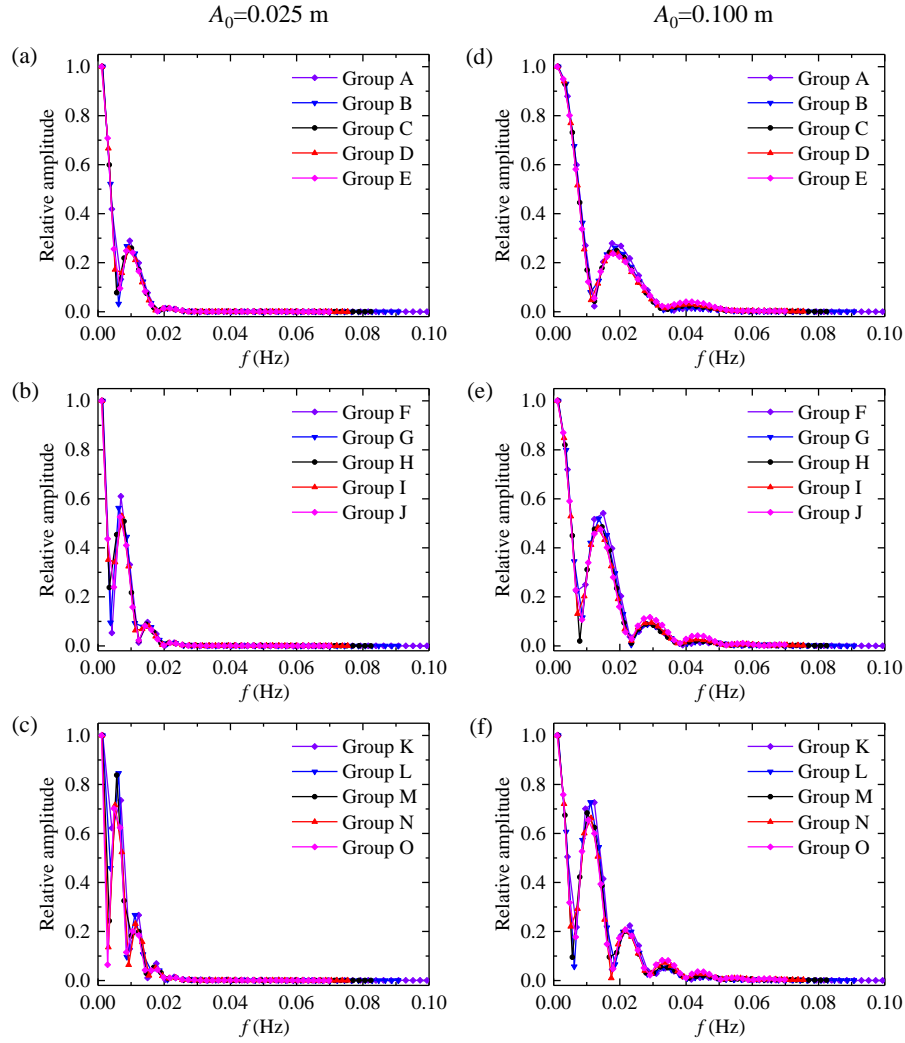
#### 12 4.4.2. Wave energy distribution

13 To better show the wave energy distribution inside the harbor, the response amplitude of each  
14 mode is normalized by that of the corresponding first mode, i.e.,

15 
$$\bar{A}_i = \frac{A_i}{A_1} \quad (i = 1, 2, \dots, 40), \quad (10)$$

16 in which  $A_i$  denotes the response amplitude of the  $i^{\text{th}}$  mode, and the normalized response amplitude,  
17  $\bar{A}_i$ , is referred to as “relative amplitude” hereinafter.

18



**Fig. 13.** Relative amplitudes of the lowest 40 modes inside the harbor with different topographies.

(a)-(c) correspond to the incident DSWs with  $R=0.4, 0.6,$  and  $0.8$  for  $A_0=0.025$  m, respectively; (d)-(f) correspond to the incident DSWs with  $R=0.4, 0.6,$  and  $0.8$  for  $A_0=0.100$  m, respectively.

Fig. 13 presents the relative amplitudes of the lowest 40 modes inside the harbor with different topographies when  $A_0=0.025$  m and  $0.100$  m. Three apparent phenomena can be observed. Firstly, for all these cases, the first resonant mode always has the largest response amplitude. Secondly, the relative wave energy distribution is shown to become more uniform for the larger incident DSWs. Take the cases with  $R=0.4$  for example. When the incident wave height is small (i.e.,  $A_0=0.025$  m) (Fig. 13a), the wave energy is dominated by the lowest few modes. As the incident wave height rises to  $A_0=0.100$  m (Fig. 13d), the proportion of the wave energy occupied by the higher modes is shown to increase notably. This phenomenon can also be observed for the cases with  $R=0.6$  (Fig. 13b and

e) and with  $R=0.8$  (Fig. 13c and f). Thirdly, the larger relative separation distance also tends to result in the greater uniformity of the wave energy distribution, which can be easily seen via comparing the relative-amplitude curves in Fig. 13a–c or those in Fig. 13d–f.

Although the influences of both the relative separation distance and the incident wave height on the wave energy distribution can be observed relatively easily, it seems difficult to directly reveal from Fig. 13 how the variation of the bottom profile influences the wave energy distribution. To measure the uniformity of the energy distribution quantitatively, the coefficient of variance ( $CV$ ) of the response amplitudes of various modes is utilized in the current study.  $CV$  is defined as

$$CV = \frac{\sigma}{\mu}, \quad (11)$$

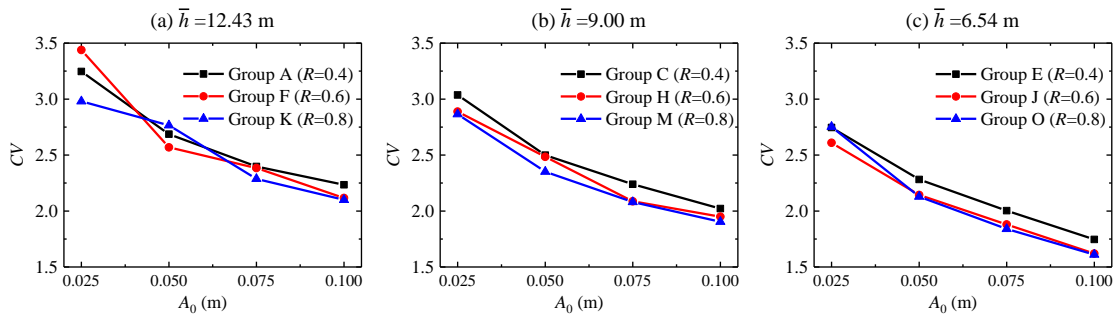
where

$$\sigma = \sqrt{\frac{1}{40} \sum_{i=1}^{40} (A_i - \mu)^2}, \quad (12)$$

and

$$\mu = \frac{1}{40} \sum_{i=1}^{40} A_i. \quad (13)$$

Apparently,  $CV$  directly reflects the deviation degree of these response amplitudes from their average value; the lower  $CV$  indicates the greater uniformity of the energy distribution.

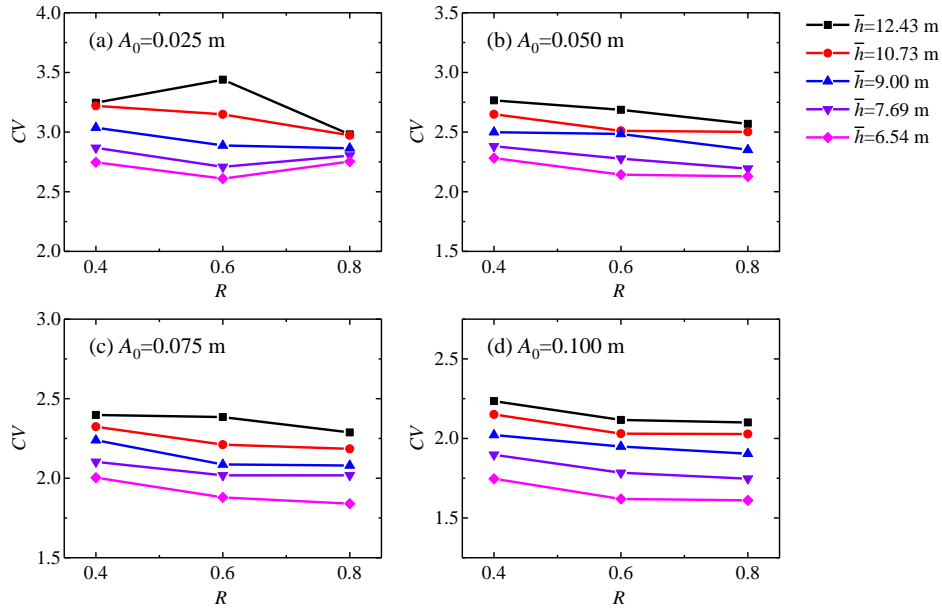


**Fig. 14.** Variations of the  $CV$  value with respect to  $A_0$  for the three topographies with (a)  $\bar{h} = 12.43$  m, (b)  $\bar{h} = 9.00$  m, and (c)  $\bar{h} = 6.54$  m.

Fig. 14 demonstrates the variation of the  $CV$  value with respect to  $A_0$  for the three topographies with  $\bar{h} = 12.43$  m, 9.00 m, and 6.54 m. For all the three topographies and all the relative separation

1 distances,  $CV$  always monotonously decreases with the increase of  $A_0$ , which indicates that the  
 2 energy distribution inside the harbor has the greater uniformity as the incident wave height rises. In  
 3 fact, the similar tendency can also be observed for the other two topographies (i.e.,  $\bar{h}=10.73$  m and  
 4 7.69 m), and their related results are not presented here.

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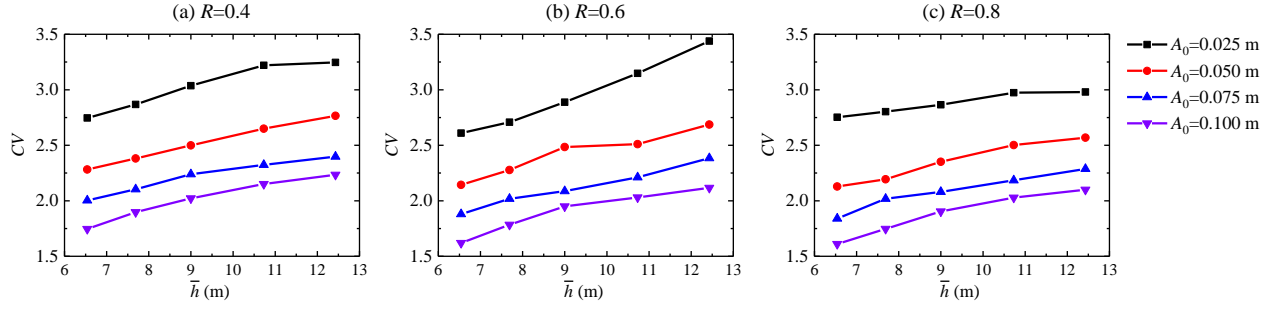
**Fig. 15.** Variations of the  $CV$  value with respect to  $R$  under various incident wave heights.

10 Fig. 15 shows the variations of the  $CV$  value with respect to  $R$  for various incident wave heights.  
 11 The third phenomenon intuitively observed in Fig. 13 is further proved by this figure. For all the  
 12 topographies considered,  $CV$  always gradually decreases with the rise of  $R$  when  $A_0=0.050$  m, 0.075  
 13 m and 0.100 m (Fig. 15b–d). When  $A_0=0.025$  m (Fig. 15a), this tendency is also observed for the  
 14 bottom profiles with  $\bar{h}=10.73$  m and 9.00 m.

15 Fig. 16 further presents the variation of  $CV$  with respect to the mean water depth for the incident  
 16 DSWs with various relative separation distances. For all the relative separation distances considered,  
 17 the  $CV$  values always decrease monotonously with the decrease of the mean water depth, regardless  
 18 of the incident wave height. This indicates that the smaller mean water depth can result in the greater  
 19 uniformity of the wave energy distribution, no matter whether the relative separation distance and  
 20 the incident wave height are large or small.

21





**Fig. 16.** Variations of the CV value with respect to  $\bar{h}$  for the incident DSWs with (a)  $R=0.4$ , (b)  $R=0.6$ , and (c)  $R=0.8$ .

#### 4.4.3. Total wave energy

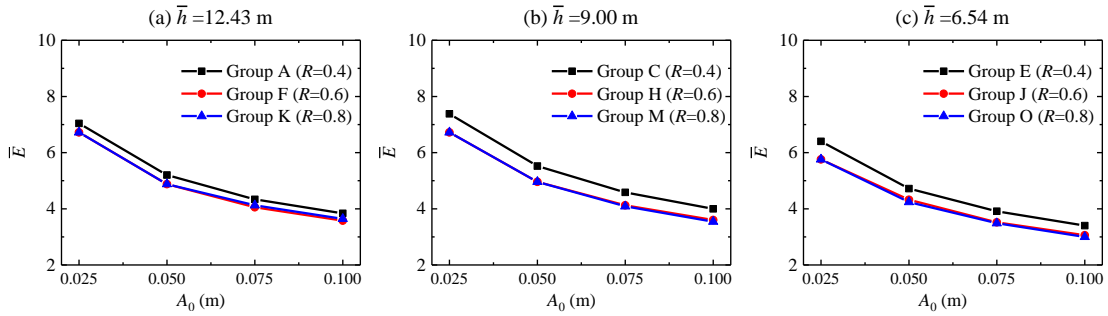
The total wave energy inside the harbor can be accurately calculated as

$$E = \sum_{i=1}^{40} \frac{1}{2} A_i^2, \quad (14)$$

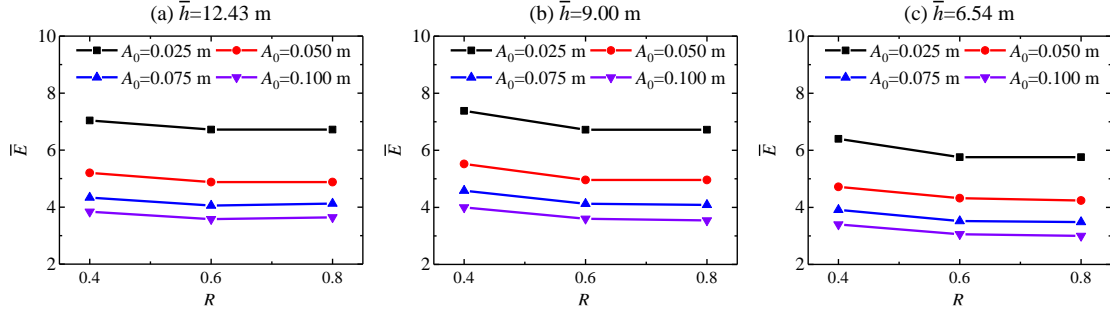
according to the fact that the total wave field inside the harbor is a linear superposition of various modes (Sobey, 2006). In this subsection, the normalized total wave energy defined as

$$\bar{E} = E / (0.5 A_0^2) \quad (15)$$

is investigated.



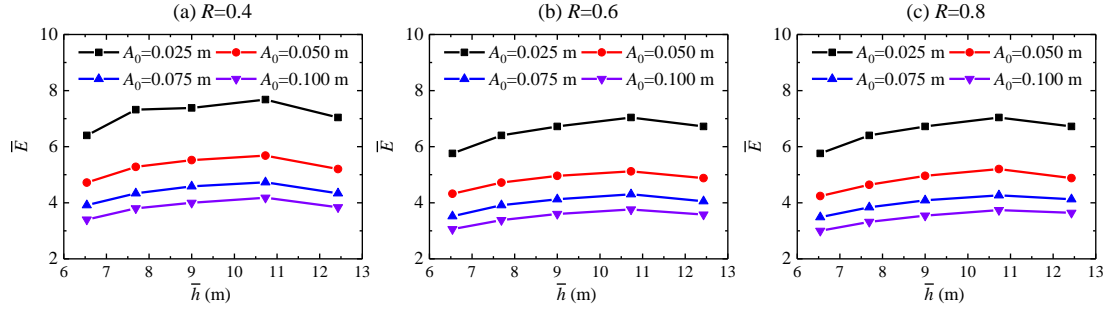
**Fig. 17.** Variation of the normalized total wave energy with respect to  $A_0$  for the three topographies with (a)  $\bar{h} = 12.43$  m, (b)  $\bar{h} = 9.00$  m, and (c)  $\bar{h} = 6.54$  m.



**Fig. 18.** Variation of the normalized total wave energy with respect to  $R$  for the three topographies with (a)  $\bar{h} = 12.43$  m, (b)  $\bar{h} = 9.00$  m, and (c)  $\bar{h} = 6.54$  m.

Fig. 17 shows the variation of the normalized total wave energy with respect to  $A_0$  for the three topographies with  $\bar{h} = 12.43$  m, 9.00 m, and 6.54 m. There are two apparent phenomena that can be easily seen. Firstly, for all these topographies, the normalized total wave energy decreases monotonously with the rise of the incident wave height, no matter whether  $R$  is large or small. Secondly, the normalized total wave energy triggered by the DSWs with  $R=0.4$  is always larger than that excited by the DSWs with  $R=0.6$  and 0.8 at the variation range of  $A_0$  considered. Besides, the normalized total wave energy with  $R=0.6$  is shown to be always extremely close to that with  $R=0.8$ . To present the second phenomenon more intuitively, the variation of the normalized total wave energy with respect to  $R$  for the three topographies are further demonstrated in Fig. 18. It shows that when  $R$  rises from 0.4 to 0.6, the normalized total wave energy presents a certain degree of decline, while as  $R$  grows further from 0.6 to 0.8, the normalized total wave energy becomes insensitive to  $R$ . Similar phenomena to those in Figs. 17 and 18 can also be easily observed for the other two topographies (i.e.,  $\bar{h} = 10.73$  m and 7.69 m).

Fig. 19 further demonstrates the variation of the normalized total wave energy with respect to the mean water depth under conditions of various relative separation distances and incident wave heights. As the mean water depth increases from 6.54 m to 10.73 m, the normalized total wave energy always presents a continuous rise. While as the mean water depth rises further to 12.43 m, the normalized total wave energy shows a slight decrease, no matter whether the relative separation distances and the incident wave heights are large or small. The maximum total wave energy always occurs on the topography with  $\bar{h} = 10.73$  m.



**Fig. 19.** Variation of the normalized total wave energy with respect to  $\bar{h}$  for the incident DSWs with (a)  $R=0.4$ , (b)  $R=0.6$ , and (c)  $R=0.8$ .

## 5. Conclusions

In the present study, the transient harbor resonance inside a long and narrow harbor with various bottom profiles triggered by DSWs with different relative separation distances and wave heights are simulated by adopting the FUNWAVE-TVD model. Influences of the incident wave height, the relative separation distance, and the bottom profile on various hydrodynamic characteristics of the transient oscillation are comprehensively investigated. The hydrodynamic characteristics considered include the evolution of the maximum free-surface elevation, the maximum runup, the wave energy distribution and the total wave energy inside the harbor. The results of the current research have enhanced the understanding on the transient harbor resonance triggered by tsunamis.

Based on the research results of this article, some main conclusions are drawn as follows:

1. The evolution of the maximum free-surface elevation inside the harbor can be estimated well by Green's law, except at the area where the incident and the reflected waves interact with each other near the backwall. The valid spatial range of Green's law is intensively dependent on the topography and the incident wave height.
2. For all the topographies considered, the maximum runups caused by the DSWs with various relative separation distances and the identical wave height are almost identical to each other when the incident wave height is relatively small ( $A_0 \leq 0.150$  m). For the incident DSWs have larger wave heights, the maximum runups with  $R=0.4$  gradually becomes larger than those with  $R=0.6$  and  $0.8$ . The impacts of the topography on the maximum runup rely heavily on the incident wave height.
3. The wave energy distribution inside the harbor has the trend of becoming more uniform as the

1 incident wave height and the relative separation distance increase. In addition, the smaller mean  
2 water depth can also result in the greater uniformity of the wave energy distribution, no matter  
3 whether the incident wave height and the relative separation distance are large or small.

- 4 4. The normalized total wave energy always declines monotonously with the rise of the incident  
5 wave height. As  $R$  rises from 0.4 to 0.6, the normalized total wave energy shows a certain  
6 degree of decline, while as  $R$  grows further from 0.6 to 0.8, the normalized total wave energy  
7 becomes insensitive to  $R$ . The normalized total wave energy is always shown to continuously  
8 increase first and then slightly decrease with the rise of the mean water depth.

9 Finally, we reaffirm here that these conclusions are only valid for the elongated harbor, the  
10 incident DSWs and the variation ranges of the incident wave height, the relative separation distance,  
11 and the mean water depth inside the harbor considered in the current study.

### 12 13 **Acknowledgments**

14 This research is financially supported by the National Key Research and Development Program  
15 (2017YFC1404200), the National Natural Science Foundation of China (Grant Nos. 51911530205,  
16 51809039 and 51609108), and the Royal Society of UK (Grant No. IEC\NSFC\181321).

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