International Conference on Estuaries and Coasts
November 9-11, 2003, Hangzhou, China

WAVE CHARACTERISTICS IN FRONT OF VERTICAL SEA-WALLS

Hanbin GU[1,3], Pengzhi LIN[2], Yanbao LI[1], Taiwen HSU[4] & Jianlue HSU [4]
[1] School of Civil Engineering , Tianjin University, Tianjin 300072 China;
[2] Dept. of Civil Engineering, National University of Singapore , Singapore 117576
[3] Tianjin research Institute of Water Transport Engineering , Tianjin 300456 China
[4] Cheng Kung University, 1st Da-shue Rd. 701 Tainan, China)
Tel. 25795508-271 e-mail. Hanbin.gu@163.com

Abstract: Wave characteristics in front of an emerged or submerged sea wall are studied by the vertical 2D numerical model NEWFLUME. Firstly the wave processes and the reflection coefficients in front of the sea wall are compared with experimental data, then the vertical distribution of maximal horizontal velocity u and vertical velocity v are compared with standing wave theory. It is found that the results have good agreement. Lastly the velocity field near the sea wall, the vertical distribution of horizontal and vertical velocity and wave pressure in front of the sea wall, and the vertical distribution of horizontal and vertical velocity at the wave node are studied and analyzed.

Key words: Vertical sea wall, Breakwater, Characteristics of wave, Wave pressure, Velocity field

1. INTRODUCTION

Vertical sea-walls or breakwaters are vertical structures built on rubble mound foundations or sea bed directly. These types of structure are traditional and popular in the protection of waves or used as wharfs. A characteristic of vertical sea walls is that the kinetic energy of the wave is stopped suddenly at the wall face. The energy is then reflected or transformed to vertical motion of the water along the wall face. This upward flow motion can double the deep water height for non-breaking case. The downward component causes very high velocities at the base of the wall and the location 1/2 wavelength away, thus causing erosion and scour.

Many analytical and laboratory studies and field observations have been undertaken to understand the wave pressure, particle velocities, reflection etc. Most of the formulas are based on monochromatic regular wave of constant height and period. For example, Sainflou's formula (1928) based on non-breaking standing waves, is derived from trochoidal wave theory. Hiroi's Formula based on field observations assumes uniform pressure distribution which was used in Japan up to 1979 when Goda's (1974) formula was adopted. Goda's formula based on model tests is still the current Japanese standard and it unifies breaking and non-breaking waves. Minikin's (1963) formula is used in U.S. In China, Qiu's (1996) formula is used for standing waves, while Li and Yu's (1975) formula for breaking waves.

The mentioned formulas only describe wave pressures, while those describing wave particle velocities and reflections are rare. However, water particle velocities at the base of sea wall are important in studying erosion and scour. In this case, model tests (Li et al. 2002a) and numerical simulations (Li et al. 2002b) need to be carried out.

In the recent years, due to the advance of computer power, the numerical wave tank (NWT) has been widely applied in the field of coastal engineering. Furthermore, it removes the scale effect in the experiment and it is comparably less expensive and time consuming. In this study, The NWT developed by Lin (2002) is employed to study wave action on vertical sea walls which are surface piercing or submerged. The wave surface processes recorded by numerical wave gauge are compared with experimental data, and velocity fields and wave pressure are analyzed. Wave reflections in both experimental tests and numerical simulation are calculated by 3-Probe method (Mansard and Funke 1980), and compared.

2. NWT'S BACKGRAND

In this section the governing equations and boundary conditions for a turbulent flow with a free surface are summarized, which are used in the NWT. The details of the derivation of these equations can be found in the literature (Lin & Liu 1998)

2.1 GOVERNING EQUATIONS

If the fluid is assumed to be incompressible, the mean flow field is governed by the Reynolds equations:

$$\frac{\partial u_i}{\partial x_i} = 0$$

$$\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = -\frac{1}{\rho}\frac{\partial p}{\partial x_i} + g_i + \frac{\partial \tau_{ij}}{\partial x_j}$$

The density change equation that can be used to track the free surface is written as:

$$\frac{\partial \rho}{\partial t} + u_i \frac{\partial \rho}{\partial x_i} = 0$$

The k-ε equations is used to close the Reynolds equations, and they read:

$$\frac{\partial k}{\partial t} + u_j \frac{\partial k}{\partial x_j} = \frac{\partial}{\partial x_j}[(\frac{\nu_t}{\sigma_k} + \nu)\frac{\partial k}{\partial x_j}] + 2\nu_t \sigma_{ij}\frac{\partial u_i}{\partial x_j} - \varepsilon$$

$$\frac{\partial \varepsilon}{\partial t} + u_j \frac{\partial \varepsilon}{\partial x_j} = \frac{\partial}{\partial x_j}[(\frac{\nu_t}{\sigma_\varepsilon} + \nu)\frac{\partial \varepsilon}{\partial x_j}] + C_{1\varepsilon}\frac{\varepsilon}{k}2\nu_t \sigma_{ij}\frac{\partial u_i}{\partial x_j} - C_{2\varepsilon}\frac{\varepsilon^2}{k}$$

and the total stresses τ_{ij} in the Reynolds equations is related to k, ε and σ_{ij} in the following way,

$$\tau_{ij} = 2\left(\nu + C_d \frac{k^2}{\varepsilon}\right)\sigma_{ij} - \frac{2}{3}k\delta_{ij}$$

in which g_i is the i-th component of the gravitational acceleration, and the rate of strain tensor of the mean flow $\sigma_{ij}=(\partial U_i/\partial x_j+\partial U_j/\partial x_i)/2$. ν is the molecular kinematic viscosity. The eddy viscosity is expressed as are empirical coefficients $\nu_t=C_d k^2/\varepsilon$. The coefficients have been determined by performing many simple experiments; the recommended values for these coefficients are:

$$C_{1\varepsilon} = 1.44, C_{2\varepsilon} = 1.92, C_d = 0.09, \sigma_k = 1.0, \sigma_\varepsilon = 1.3$$

2.2 NUMERICAL IMPLEMENTATION

In the numerical model, the Reynolds equations are solved by the finite difference two-step projection method. The first step is used to obtain the tentative velocity without considering the pressure effect. The second step is to update the velocity information using the new pressure information, which is obtained by solving the Poisson pressure equation. In this method the central difference method is used to discretize all the pressure and stress gradients. The combination of central difference and upwind scheme is used to discretize the advection terms. The same discretization method is used for solving the k-ε transport equations. Acceptor donor method is implemented to discretize the VOF equation.

3. PROBLEM SETUP

In this study, we will attempt to verify the reliability of the NWT named NEWFLUME which is improved recently, and study the wave characteristics in front of vertical sea walls. In the mean time, a laboratory experiments were conducted in Cheng Kung University .

The NWT used in this study has the length of 10m and height of 0.4m. All numerical tests run in a constant water depth h=0.2m. The vertical sea walls that are located at 7.78m from the left side of the NWT have four kinds of height which are 0.1m, 0.15m, 0.24m, and 0.3m respectively. Four wave gauges are deployed at 4.43m, 5.03m, 5.8m, 6.94m respectively from the left side of the NWT, and the front three are used to measure the wave reflection coefficients, the latter one located at 6.94m is use to measure the waves process which can be compared to the experimental data. A typical example of problem is shown in Fig.1. The internal wave maker (Lin & Liu 1999) is set at 1.0m to the left side. In this study only regular waves are generated whose height is 0.06m and period 1.5s. The cnoidal wave theory is used in internal wave maker.

In the numerical computations, a non-uniform grid system with is used in which the minimum Δx is 0.02m, and the minimum Δy 0.01m. A mesh refinement test has been conducted to ensure that the currently used mesh is fine enough to provide convergent numerical results.

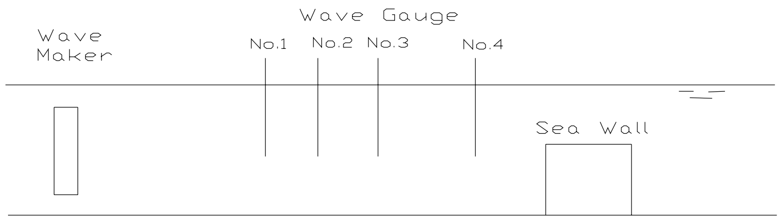


Fig. 1 Schematic Plot of Problem Setup

4. RESULTS AND DISCUSSION

4.1 COMPARISON OF WAVE SURFACE PROFILES AND VELOCITY

In order to verify the reliability of the model, the time histories of the free surface displacements at four locations are compared with laboratory measurements (Fig. 2).The overall agreement is reasonably good. The gauge data are used to calculate wave reflection coefficients, which are important measures of the effectiveness of sea wall protection. Table 1 shows the comparisons of the reflection coefficients calculated by numerical model and laboratory measurement. It is noted that the reflection coefficients calculated by numerical model are slightly smaller than measurement. Fig. 3 shows that the vertical distribution of maximal horizontal velocity u and vertical velocity v which are calculated by the numerical model compared with the results of standing wave theory and it is found that the results have good agreement. So we think the NWT is accurate and it can be used to study the wave actions on structures.

Table 1 Comparisons of the Reflection Coefficients

Water Depth h (m)	Wave period T(sec)	Wave Length L(m)	Wave Height H(m)	Sea Wall Height D(m)	Reflection Coefficients	
					Laboratory Measurement	Numerical Result
0.20	1.5	1.976	0.06	0.10	0.156	0.123
				0.15	0.367	0.338
				0.24	0.813	0.756
				0.30	0.893	0.891

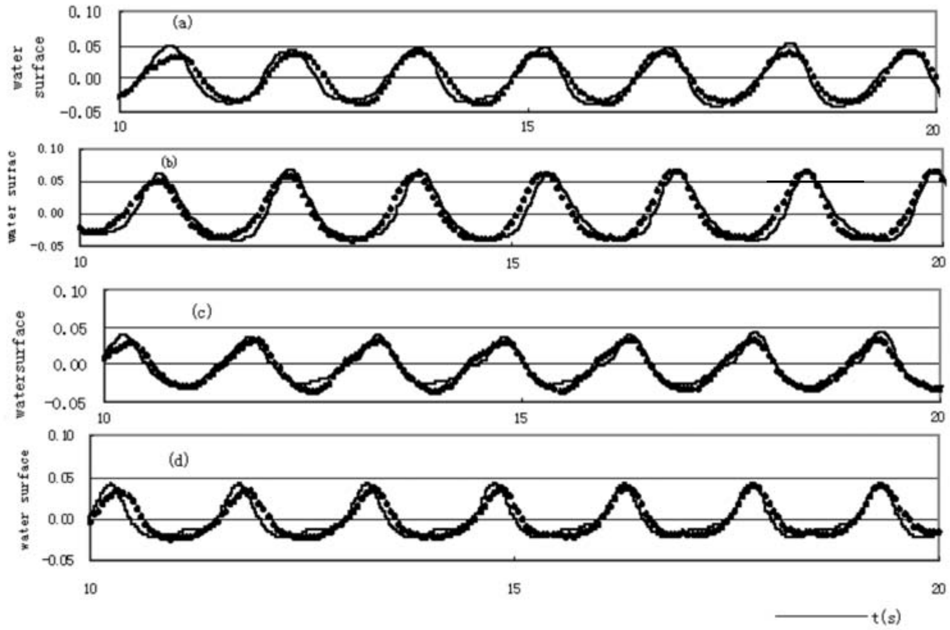


Fig. 2 Comparisons of Free Surface Profiles between Numerical Results (——) and Laboratory Measurements (……) at Gauge No.4 and Sea Wall Height is (a)0.3m, (b) 0.24m, (c)0.15m, (d) 0.10m.

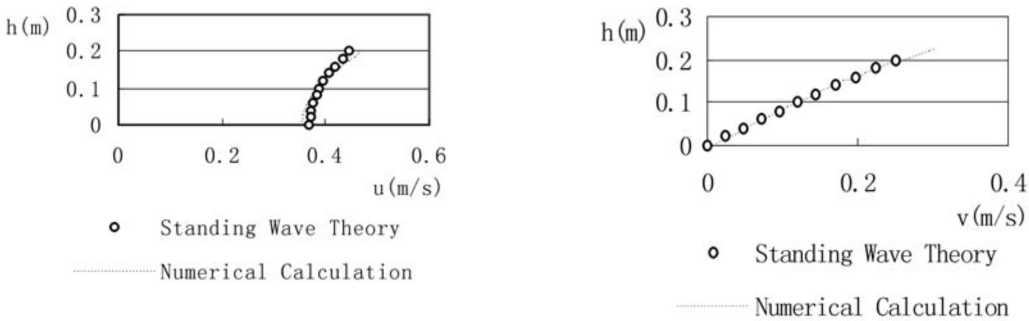


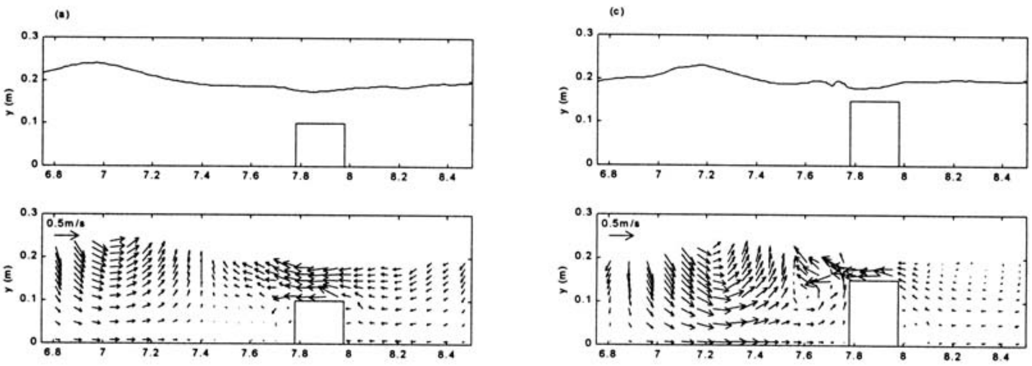
Fig. 3 Vertical distribution of horizontal velocity *u* and vertical velocity *v* compared with standing wave theory

4.2 ANALYSIS OF VELOCITY FIELD

The variation of velocity field in a wave period for different sea wall height is calculated. When the sea wall is submerged, as wave crest propagates over the sea wall, there is a vortices in front of the sea wall, and with the sea wall height increasing the vortices becomes large [Fig. 4 (a) and (b)]. The phenomena may be caused by two reasons. The one is that when the sea wall is submerged, the section of water above the sea wall is reduced and the fluid velocity there increases， especially true under the wave trough, where, the direction of velocity is reversal to wave direction. Another is that larger wave energy is reflected from a higher sea wall that may induce stronger vertical motion. When the sea wall is surface-piercing, the phenomena is different from the above mentioned. In the case of the sea wall height being not high enough and a little water running over the top of the sea wall, the partial standing wave is formed, the oscillation before the sea wall is nearly symmetry, and there is small vortices behind the sea wall caused by the overtop water [Fig. 5 (a)]. In the case of the sea wall height being enough high and no water going over the sea wall, the oscillation before the sea wall is symmetry [Fig. 5 (b)], and the water behind the sea wall is not disturbed. The interesting phenomena are important when we analyze the scour and sediment before the sea wall.

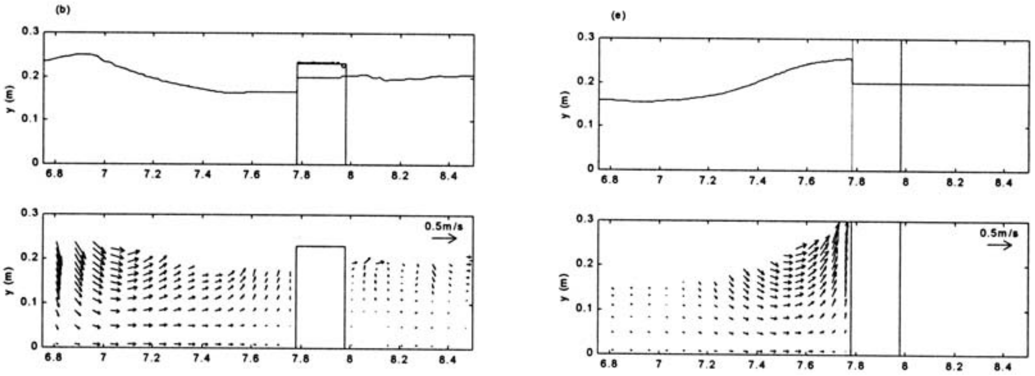
4.2.1Vertical Distribution of Velocity

The vertical distribution of horizontal velocity and vertical velocity is analyzed at the wave node in front of the sea wall and the front face of the sea wall when the sea wall is surface piercing or submerged. It is shown that the forward and backward maximal horizontal and the upward and downward vertical velocity are nearly symmetry in a wave period at the wave node in front of the sea wall and the front face of the sea wall when the sea wall is surface piercing. While when the sea wall is submerged, the phenomena are different. Fig. 6 shows the case of the sea wall height being 0.15m, and Fig. 7 the case of the sea wall height being 0.10m. In these figs, the vertical coordinate is distance from the bottom, and horizontal coordinate is the maximal forward and backward horizontal component or the maximal upward and downward vertical component of velocity. The forward and upward is assigned to be positive. Because of that when the sea wall is submerged the water can go through the top of sea wall and the section is smaller than that in front of the wall, the velocity field is disturbed. The closer the top of wall to the still water level, the more disturbing the current field is.



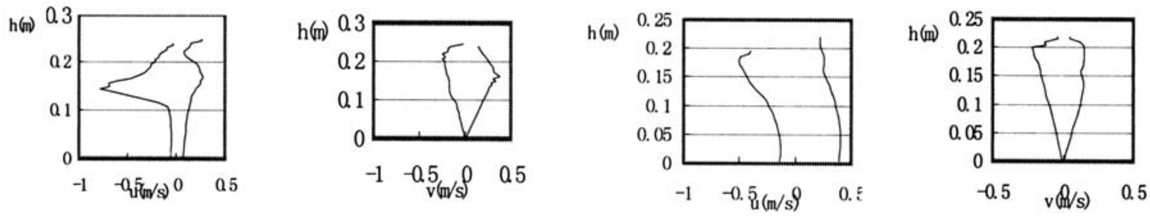
(a) (b)

Fig. 4 Velocity field at (a) the sea wall height 0.1m (b) the sea wall height 0.15m



(a) (b)

Fig. 5 Velocity field at (a) the sea wall height 0.24m, (b) the sea wall height 0.30m

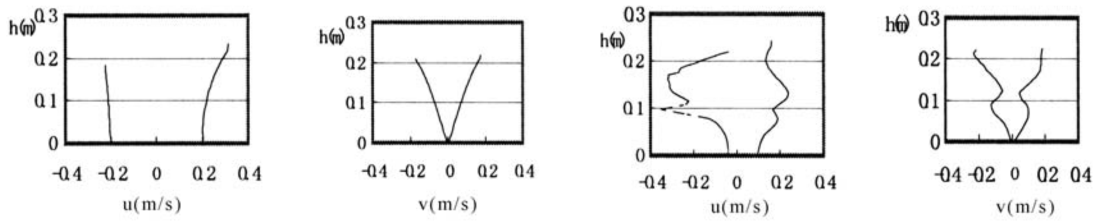


(a) (b) (c) (d)

Fig. 6 Vertical Distribution of Velocity when the height of wall is 0.15m: (a) horizontal velocity and (b) vertical velocity at 0.05m in front of the Wall and (c) horizontal velocity and (d) vertical velocity at 0.50m in front of the Wall

4.2.2 Vertical Distribution of pressure

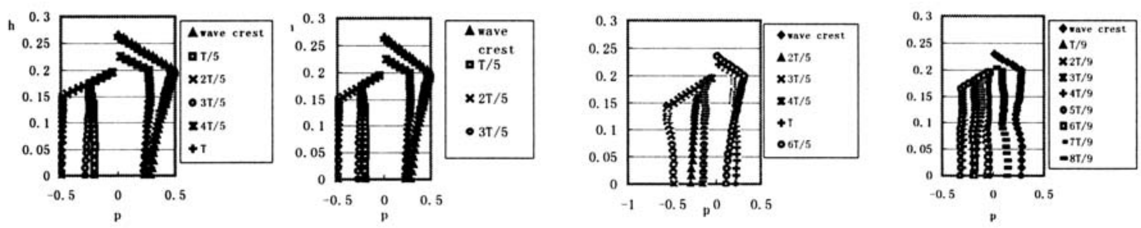
Fig. 8 shows the vertical distributions of pressure which are at the front of the sea wall and at different times in a wave period when the height of the sea wall is 0.3m, 0.24m, 0.15m and 0.10m. In these Figs, the vertical distributions of pressure are plotted at the time interval T/5 or T/9(T is the wave period) from a time at wave crest. The standing wave is formed in Fig. 8 (a), the positive wave force arrives maximum at wave crest and the negative wave force arrives minimum at wave trough, the wave pressure from the bottom to surface of the water arrives maximum or minimum at the same time. The incomplete standing wave is formed in Fig. 8 (b), in which there is a little overtop at wave crest, and the wave pressure phenomena is the same as Fig. 8 (a). In Fig. 8 (c), the top of the sea wall is closely and below the still water level, the water section up the top of the sea wall is very small when the wave trough is up the top of the sea wall. In this case the positive wave pressure from bottom to the top of the sea wall does not arrive at the maximum; the negative wave pressure is not similar to Fig. 8 (a) and (b), which is not gradually increasing from the bottom to the surface of water. The Fig. 8 (d) is more interesting in which the positive wave pressure apparently decrease at the top of the sea wall but the negative increase at this point.



(a) (b) (c) (d)

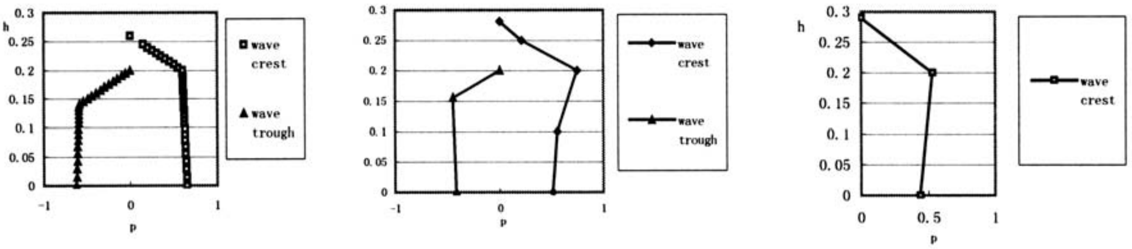
Fig. 7 Vertical Distribution of Velocity when the height of wall is 0.10m: (a) horizontal velocity and (b) vertical velocity at 0.05m in front of the Wall and (c) horizontal velocity and (d) vertical velocity at 0.50 in front of the Wall

Usually we use empirical formula to calculate the wave force acting on the sea wall. Fig. 9 shows the corresponding results by three typical empirical formulas. It shows that the empirical formula often overestimates the wave pressure and the vertical distributions are not as the same as the fact. For example, the formula of Code of Hydrology for Sea Harbour of China and Goda's formula can describe the shape of the positive vertical distribution of wave pressure when the sea wall is surface-piercing, the Sainflou's formula can describe the shape of the negative vertical distribution of wave pressure when the sea wall is surface-piercing in these cases. But none can describe the shape of the vertical distribution of wave pressure when the sea wall is submerged. The numerical simulation can give the details in these cases.



(a) (b) (c) (d)

Fig. 8 Vertical Distribution of Pressure at the front of the sea wall at different time in a wave period : (a) the height of the sea wall 0.3m (b) the height of the sea wall 0.24m (c) the height of the sea wall 0.15m (d) the height of the sea wall 0.10m



(a) (b) (c)

Fig. 9 Vertical Distribution of Pressure at the front of the sea wall: (a) Sainflou's Formula (1928), (b) Code of Hydrology for Sea Harbour of China, (c) Goda's Formula

5. CONCLUSIONS

In this paper, the NWT is used to study the characteristics of waves in front of the vertical sea walls. It is found that wave surface profiles in front of the sea walls surface-piercing or submerged are well consistent with the experimental data and the wave reflection coefficients of numerical test have good agreement with that of physical experiment. Moreover it is noted that the vertical distributions of horizontal and vertical velocity which are calculated by the NWT in the condition of standing wave agrees excellently to that of standing wave theory. Then the velocity fields near the sea wall and wave pressure at the front of the sea wall are studied. The interesting results are gained which are that there are vertices at the front of the sea wall when its top is close but below the still water level. The vertical distributions of horizontal and vertical velocity at the front of the sea wall and at the wave node are described for different heights of the sea wall. Lastly wave pressure at the front of the sea wall is discussed. The findings are that the vertical distribution of wave pressure varies with the heights of the sea wall, and the wave pressure from the bottom to the water surface does not reach the maximum at the same time when its top is close to the still water level.

REFERENCES

Code of Hydrology for Sea Harbour, JTJ 213-98 .The PRC Communication Ministration.

Goda Y. 1974. New wave pressure formulae for composite breakers. Proc. 14th Int. Conf. Coastal Eng., Copenhagen, 1702-1720.

Herbich, J. B. 1991. Coastal Engineering Handbook.

Li Y. B., Huang L. Y. and Gu H. B. 2002a. Simulation of Partial Standing Wave by Two-dimensional Numerical Wave Flume, *ACTA Oceanologica Sinica*, Vol.24,No.4, 105-110.

Li Y. B., Gu H. B. and Zhang S. S 2002b. Wave Reflection Caused by Wave Overtopping and Sloping Top of Structure, *China Ocean Engineering*, Vol.16, No.2, 211-218.

Li Y. C. et al. 1975. Breaking wave forces on vertical breakwater, Transaction of Dalian university of Technology 14（3）：81-104. (in Chinese)

Lin, P. 2002 Transmission and Reflection of Solitary Waves over Structures: A Numerical Wave Tank (NWT) Approach, 25th Proc. Int. conf. Coastal Eng. 1757-1769.

Lin, P. and Liu, P. L.-F. 1998. A Numerical Study of Breaking Waves in the Surf Zone. *J. Fluid Mech*. Vol.359, 239-264

Lin, P. and Liu, P. L-.F. 1999. Internal Wave-Maker For Navier-Stokes Equations Models. *Journal of Waterway, Port, Coastal, and Ocean Engineering*. Vol.125,No.4, July/August, 207-215

Liu, P. L.-F. and Lin, P. A Numerical Model for Breaking Waves: The Volume of Fluid Method. School of Civil and Environmental Engineering, Cornell University, Ithaca, NY 14853.

Mansard, E. P. D. and Funke, E. R., 1980. The Measurement of Incident and Reflected Spectra Using a Least Squares Method, Proc. 17th ICCE, Sydney, Australia, 154-172.

Minikin, R. R. 1963. Winds, waves, and maritime structures: studies in harbor making and in the protection of coasts. 2nd Edition, Griffin, London, England.

Qiu D. H. et al. 1996. Interation between Cnoidal wave and vertical wall. Journal of Hydraulic Engineering（9）11-21. (in Chinese)

Sainflou, M. 1928. Essai sur les Digues Maritimes Verticales. *Annales des Ponts et Chaussees*. Vol. 98, Pt. 1, Tome 11.
Yu Y. X. et al. 1975. Broken wave forces on vertical breakwater,. Transaction of Dalian university of Technology 14（2）：1-14. (in Chinese)
Yu Y. X. 2000. Random Wave and Its Applications for Engineering. Dalian University of Technology Press.