Coastal Engineering 99 (2015) 46–63

Contents lists available at ScienceDirect

Coastal Engineering

journal homepage: www.elsevier.com/locate/coastaleng

Large-scale experiments on tsunami-induced pressure on a vertical tide wall

Naoto Kihara [a,*], Yasuo Niida [b], Daisuke Takabatake [a], Hideki Kaida [a], Atsushi Shibayama [a], Yoshinori Miyagawa [a]

[a] Civil Engineering Research Laboratory, Central Research Institute of Electric Power Industry, 1646 Abiko, Abiko-shi, Chiba 270–1194, Japan
[b] Environmental Science Research Laboratory, Central Research Institute of Electric Power Industry, 1646 Abiko, Abiko-shi, Chiba 270–1194, Japan

ARTICLE INFO

Article history:
Received 26 October 2014
Received in revised form 10 February 2015
Accepted 19 February 2015
Available online 16 March 2015

Keywords:
Tsunami
Large-scale experiment
Hydrodynamic pressure
Impact pressure

ABSTRACT

Large-scale experiments on the pressure exerted on a tide wall due to the impact, reflection, and overflow of tsunami inundation flow were carried out to investigate the characteristics of the pressure and the flow in front of the tide wall in the bore impact, initial reflection, and quasi-steady-state phases. The Central Research Institute of Electric Power Industry (CRIEPI)'s Large-scale Tsunami Physical Simulator, in which large and long tsunami inundation flows can be reproduced, was used in the experiments. In the impact phase, two types of short-duration pressures with durations in the ranges of 10^{-3} to 10^{-2} s and 10^{-1} to 1 s were observed. The latter pressure can be partially predicted by Cumberbatch's theory (1960). For cases of tsunami bore with high impact speeds, the theory obviously overestimates the impact force. For cases of tsunami bore with low impact speeds, the forces predicted by the theory agreed reasonably well only with the forces measured just after the bore impact. By contrast, the forces predicted by the modified solution of the theory agreed well with the measured forces for these cases. In the initial reflection phase, pressure greater than the hydrostatic pressure at the depth of the incident bore was exerted at heights above the depth of the bore. The most likely cause of this is the collapse of the water column. In the quasi-steady-state phase, the pressures were almost equal to the hydrostatic pressure upstream of the wall. The pressure and the flow reached the quasi-steady state within a few seconds after the bore impacted the wall.

1. Introduction

The 2011 Tohoku earthquake tsunami struck a wide area of the northeastern coast of Japan. The maximum inundation depths near shoreline were reported to be more than 15 m, and the measured run-up heights were more than 30 m (Mori et al., 2012). The earthquake and tsunami caused more than 15,000 deaths, left 2000 people missing, and caused extensive infrastructure damage (Nandasena et al., 2012). For the purposes of disaster prevention and tsunami mitigation in coastal areas in Japan, new tsunami tide walls are being constructed and the heights of existing tide walls are being increased.

To assess the tsunami fragility of tide walls, it is necessary to predict the tsunami loads that act on the walls. The hydrodynamic loads, buoyancy, debris impact loads, and scouring effects should be considered in the fragility assessment. Of these forces, the hydrodynamic loads, i.e., the tsunami wave pressures on the seasides of tide walls, are most likely to cause damage to the walls.

When a tsunami approaches a shoreline, it breaks where the water depth is approximately equal to the incident wave height, and becomes a turbulent bore. Near the shoreline, the propagation speed of the bore decelerates by compressing its wave form as it approaches the shoreline. At the shoreline, the propagation speed suddenly accelerates. Yeh et al. (1989) showed the acceleration mechanism at the shoreline. When the bore approaches the shoreline, the bore pushes a small initially quiescent mass of water in front of it. Yeh et al. (1989) called this acceleration mechanism 'momentum exchange'. Due to the momentum exchange, a single bore generates two successive run-up motions; one is a turbulent run-up motion, which has its origin in the water mass pushed up by the momentum exchange, followed by the original bore motion.

The run-up motion of the tsunami bore on a dry bed is in a manner similar to the motion of a dam-break flow (Chanson, 2006, 2009), and thus the structure of the tsunami bore can be explained by using knowledge on that of the dam-break flow. In order to explain the structure of the dam-break flow, we refer the theoretical model developed by Chanson (2006). Fig. 1 shows the schematic view of the bore. In the main body of the flow, the flow resistance is negligible, and the flow

* Corresponding author. Tel.: +81 70 6576 8935; fax: +81 4 7184 7142.
E-mail addresses: kihara@criepi.denken.or.jp (N. Kihara), niida@criepi.denken.or.jp (Y. Niida), tdaisuke@criepi.denken.or.jp (D. Takabatake), h-kaida@criepi.denken.or.jp (H. Kaida), atushi@criepi.denken.or.jp (A. Shibayama), miyagawa@criepi.denken.or.jp (Y. Miyagawa).

http://dx.doi.org/10.1016/j.coastaleng.2015.02.009
0378-3839/

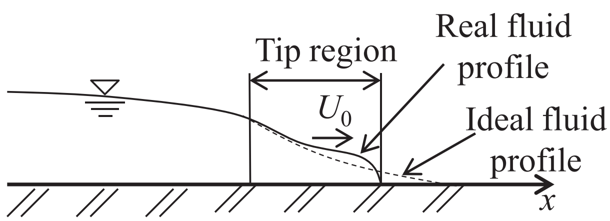

Fig. 1. The schematic view of the bore of the dam-break flow.

profile can be described by an analytical solution to the shallow water model, or the ideal flow model, of Ritter (1892). On the other hand, in the tip region, the role of the flow resistance is important (Chanson, 2006, 2009; Hogg and Pritchard, 2004; Whitham, 1955). In the forward tip region, the flow velocity is approximately equal to the traveling speed of the bore tip (e.g., Dressler, 1952), and the flow resistance term is dominant rather than the acceleration and inertial terms in the momentum equation. Next to the leading edge of the bore, the slope of the free-surface counterbalances the flow resistance.

The characteristics of tsunami wave pressures or wave forces on structures in tsunami inundation areas (e.g., tide walls, buildings, and houses) and the methods for predicting them have been investigated theoretically, experimentally, and numerically (e.g., Asakura et al., 2002; Fukui et al., 1963; Lukkunaprasit et al., 2009; Yeh, 2006). Fukui et al. (1963) measured pressures exerted by bores on a levee with a sloped face on the seaside, using a two-dimensional flume. The bores were generated by rapidly opening a hinged gate. The researchers reported observing two characteristic pressure patterns. The first of these pressure patterns was an impulsive/dynamic pressure observed just after the bore impacted the levee. The second was a continuous pressure observed after the reflection of bores. The vertical distribution of this second pressure pattern was approximately hydrostatic. Arikawa et al. (2005, 2006) measured pressures on a rectangular block due to long sinusoidal waves with periods in the range of 14–60 s in a large wave flume with a length of 184 m, a width of 3.5 m, and a depth of 12 m. The rectangular block was set at the shoreline. The sinusoidal waves were generated by a moving panel. Arikawa et al. (2005, 2006) reported observing the same two characteristic pressure patterns as those observed by Fukui et al. (1963).

Nouri et al. (2010) and Palermo et al. (2013) measured the pressure and horizontal forces exerted by bores on two structures with square and circular cross sections on a dry bed. The bores were generated by rapidly opening a hinged gate. The researchers showed that there are three phases in the pattern of the horizontal forces: impulsive, run-up, and quasi-steady hydrodynamic forces. The quasi-steady hydrodynamic force exhibits the same pattern as the continuous pressure/force observed by Fukui et al. (1963) and Arikawa et al. (2005, 2006). The run-up force is observed during the transition between the impulsive and quasi-steady hydrodynamic forces and occurs due to the effects of the flipped water mass resulting from the bore impact. In a series of numerical studies (Kihara et al., 2012; Takabatake and Kihara, 2014; Takabatake et al., 2013), we have also observed the three-phase pressure pattern exerted on structures. A local maximum pressure was observed during the transition between the impulsive and quasi-steady phases, and we presumed that this local maximum pressure is caused by convergence of the upward flow accelerated by the continuous burst of the incident flow in low areas on the wall and the downward flow accelerated by gravity in high areas. However, the mechanism that produces this local maximum pressure remains unclear.

The equation used in structural design to estimate the hydrodynamic force on a structure produced by a tsunami bore is often expressed as follows (FEMA, 2000):

$$F = \frac{1}{2}\rho_w C_D V^2 hb \tag{1}$$

where ρ_w is the water density, C_D is the drag coefficient, h is the inundation depth, V is the velocity of tsunami inundation flow, and b is the width of the structure. The predicted force is the difference between the forces exerted on the upstream and downstream faces of the structure. Arnason et al. (2009) measured the forces exerted on cylindrical and square structures in a flume with a 20-mm water depth due to dam-break bores. They showed that values of $C_D = 2$ for the square structure and $C_D = 1$ to 2 for the cylindrical structure are reasonable for the quasi-steady state. Yeh (2006) suggested a value of $C_D = 3$ for the maximum surge force for bores, which is consistent with values obtained by Ramsden (1993) and Arnason (2005) in separate, independent experiments.

The equation used to estimate the pressure exerted on structures such as tide walls that stem tsunami flow and whose backside state is dry is often expressed as follows (e.g., Asakura et al., 2000, 2002):

$$p = \alpha\rho_w g(h_{max} - z) \tag{2}$$

where p is the pressure exerted on the upstream face of the structure, z is the height from the bottom, h_{max} is the maximum inundation depth, g is the acceleration due to gravity, and α is a coefficient. This expression means that the pressure profile is expressed as the hydrostatic pressure with depth (αh_{max}). Asakura et al. (2000) measured pressures and forces exerted on the upstream faces of rectangular structures on a dry bed due to tsunami-like flows generated by controlling the flow volume of a pump at the edge of a test flume with a 0.92-m water depth. The tsunami-like flows traveled over the sloped bottom of the flume, overflowed a dike, and impacted the structures. The researchers carried out experiments for 84 combinations of the slope gradient, the flow type, and the location of the structures. The maximum pressure at each measured height for each flow type was less than the pressure calculated using a value of $\alpha = 3$, except for the flow types with soliton fission. In contrast, the pressures measured at low heights for the flow types with soliton fission were sometimes larger than the pressure calculated using a value of $\alpha = 3$. The waves generated by soliton fission were relatively short and tended to break in shallow areas. These breaking waves became bores and impacted the structures, resulting in impulsive pressure at low heights in the experiments. Asakura et al. (2002) suggested that the pressure could be predicted more accurately by using a value of α that depended on the Froude number. However, these prediction methods do not distinguish among the three characteristic pressure patterns mentioned previously.

Cumberbatch (1960) developed a theory for describing the impact of a water wedge on a vertical wall. The theory is based on assumptions of incompressible and inviscid fluid and irrotational flow without the effect of gravity. The pressure profile exerted on the vertical wall is predicted from the constant wedge angle θ ($\leq 45°$) and impact speed U_0, using the following equation:

$$\frac{p}{\frac{1}{2}\rho_w U_0{}^2} = C_p(\theta) f_\theta(\lambda), \tag{3}$$

where $\lambda = z / U_0 t$, C_p is a parameter that depends on θ, and f_θ ($f_\theta(0) = 1$ and $f_\theta(\infty) = 0$) is the distribution function in the λ-direction, which decreases as λ increases. Thus, the maximum pressure is $1/2\rho_w U_0^2 C_p(\theta)$ and is invariant with time, and the pressure profile extends upward at a constant speed because $\lambda = z / U_0 t$. From Eq. (3), we can derive the following expression for the horizontal force F per unit width exerted on a vertical wall:

$$F = \frac{1}{2}\rho_w U_0{}^3 C_p(\theta) t \int_0^\infty f_\theta(\lambda) d\lambda. \tag{4}$$

Eq. (4) shows that F increases linearly with time t.

Cross (1967) investigated bore tip profiles traveling over smooth and roughened bottoms and measured the forces exerted on a vertical