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수리실험 및 수치모의를 이용한 제방붕괴 흐름해석

Levee Breach Flow by Experiment and Numerical Simulation

김 주 영* / 이 정 규** / 이 진 우*** / 조 용 식****
Kim, Jooyoung / Lee, Jong-Kyu / Lee, Jin Woo / Cho, Yong-Sik

Abstract

Abrupt and gradual levee breach analyses on the flat domain were implemented by laboratory experiments and numerical simulations. To avoid the reflective wave from the side wall the experiment was performed in a large domain surrounded by waterway. A numerical model was developed for solving the two-dimensional gradual levee breach flow. The results of the numerical simulation developed in this study showed good agreement with those of the experimental data. However, even if the numerical schemes effectively replicated the trends of the observed water depth for the first shock, there were little differences for the second shock. In addition, even though the model considered the Smagorinsky horizontal eddy viscosity, the location and height of the hydraulic jump in the numerical simulation were not fairly well agree with experimental measurements. This shows the shallow water equation solver has a limitation which does not exactly reproduce the energy dissipation from the hydraulic jump. Further study might be required, considering the energy dissipation due to the hydraulic jump or transition flow from reflective wave.

Keywords : levee breach, reflective wave, smagorinsky eddy viscosity, shallow water equation

1. INTRODUCTION

Recently, all over the world, the climate change became a big social issue. Increasing of the local heavy rain or probability precipitation threatens the people and existing disaster prevention measures, especially existing dams or levees designed by previous climate characteristics are being exposed to new danger. To prevent or decrease flood damage, both structural and non-structural countermeasures should be considered. While typical structural countermeasures are levees or dams, including basic and important flood protection functions, nonstructural countermeasures include flood forecasting and Emergency Action Plans (EAPs). In particular, as many levees or dam, one type of the flood defense structure for river areas, are constructed using

* 한국전력기술주식회사 환경기술그룹 책임기술원 (전 한양대학교 건설환경공학과 박사과정) (e-mail: orion@hanyang.ac.kr)
Senior Engineer, Environmental Technology Dept., KEPCO Engineering & Construction Company, Inc., Yongin 446-713, Korea
** 한양대학교 공과대학 건설환경공학과 명예교수
Professor Emeritus, Dept. of Civil and Env. Eng., Hanyang Univ., Seoul 133-791, Korea
*** 한양대학교 공과대학 건설환경공학과 박사과정
Graduate student, Dept. of Civil and Env. Eng., Hanyang Univ., Seoul 133-791, Korea
**** 교신저자, 한양대학교 공과대학 건설환경공학과 교수
Corresponding Author, Professor, Dept. of Civil and Env. Eng., Hanyang Univ., Seoul 133-791, Korea

the soil material, they can be broken easily when the function of flood defense fails. As a result, many levee- or dam-break studies have been conducted in the numerical and experimental approaches during the past few decades.

In the numerical approach, the main interesting issue with regard to levee- and dam-breach is how to solve the discontinuous or shock problem. To solve the discontinuous flows (shock flows) the governing equations must be in conservation form and shock-capturing approaches are required (Toro, 2001). Glaister (1988) solved the one-dimensional dam-break problem on a flat bottom based on the flux difference splitting. Jha et al. (1995) tested one-dimensional dam-break problems using the first- and second-order Flux Difference Splitting schemes. Fraccarollo and Toro (1995) suggested Godunov-type second-order accurate numerical schemes for two-dimensional problems on wet and dry beds. Vincent and Caltagirone (2001) presented a one-dimensional MacCormack total variation diminishing (TVD) scheme and applied it to a dam-break problem on wet and dry beds as well as to a simulation of wave run-up on a sloping beach. Bradford and Sanders (2002) proposed a finite volume method coupled with MUSCL data reconstruction and a Riemann solver. Liang et al. (2004) developed a two-dimensional shallow water solver along with the Harten, Lax and van Leer(HLL) approximate Riemann solver for dam- and dyke-break simulations using adaptive quadtree grids. They found a reasonable agreement with experimental measurements from a CADAM dam-break test and data from a laboratory dyke break undertaken at the Delft University of Technology. In this study, the two-dimensional numerical solver to calculate the gradual and abrupt dam or levee-breach problems was developed. The governing equations of the model are two- dimensional nonlinear shallow water equations and discretized explicitly by using a finite volume method (FVM). An intercell flux is reconstructed with a HLLC approximate Riemann solver which is a modification of the basic HLL scheme to account for the influence of intermediate waves. Also, a TVD version of the Weighted Averaged Flux (WAF) scheme with SUPERBEE type limiter is applied to control the numerical oscillations in the vicinity of large gradients with second-order numerical accuracy. The wet and dry fronts problems are also considered. Especially, in order to consider turbulent eddy viscosity the Smagorinsky (1963) model was adopted for the horizontal eddy viscosity, and the moving boundary condition for simulating gradual levee breach was adopted.

In the experimental approach, some small scale laboratory studies for the levee- or dam-breach problems have been performed for verifying the accuracy of the numerical model by some pre-studies. In 1997, the experiment of the CADAM (Concerted Action Dam-break Modeling) group is representative. They carried out laboratory dam-break experiments using a rectangular upstream reservoir connected to a 90° and 45° bend open channel. Frazão and Zech (2002) performed an experimental study of a dam-break flow, in an initially dry channel with a 90° bend, with refined measurements of water level and velocity field. Frazão and Zech (2008) studied a sudden flow of the dam-break wave type in an idealized city. They focused on the severe transient phase of such flows, with the aim of making available a complete data set with accurate flow depth and velocity measurements, allowing for the validation of numerical models. Yoon (2008) developed a sliding gate system that can open gradually to simulate a gradual levee-break on the inundation area. Especially, on the condition that the flow of the channel exists, he measured the flow rate, wave front propagation and inundation depth of the levee-break flow in the horizontal inundation area. Finally, he found a various characteristics of the levee-break flow. This study was performed in a large scale model to obtain the accuracy of the experiment, and tested on the flat inundation area large enough to avoid the reflective wave and friction of the wall by Kim (2010). In order to effectively measure the flood depth, advanced ultrasonic distance sensors (UDS) were used instead of the resistive level gauges which are normally used in the hydraulic laboratory.

Finally, this study found the flood inundation depth according to the levee-breach into the horizontal inundation area., and the measured water depth was compared with the results of the two-dimensional numerical solver that was developed in this study.

2. LEVEE-BREACH EXPERIMENTS

2.1 Experimental set-up

Experiments of abrupt and gradual levee-breach were performed mainly to find the differences of the levee failure types. Each failure type has three cases in accordance with no block, block and its rotation angle. The test area is 30 m by 30 m with a horizontal concrete bed, and the experimental domain is divided into the reservoir and inundation area by the vertical concrete wall as shown in Fig. 1. To reproduce the levee-breach of the concrete wall, two types of gates were set up on the center of the wall. One is a sliding gate designed to simulate a gradual levee-breach and the other one is a gate that can be fallen down instantaneously in order to simulate an abrupt levee-breach. The speed of the sliding and the width of the gate were fixed at a rate of 0.07 m/s and at 1.0 m, respectively. This gate was designed to simulate a vertical levee-breach into left and right direction at constant speed. Real breaking characteristic of levee or dam is very complex and various, so it is very difficult to simulate the real phenomena. Yoon (2008) performed the levee-breach experiments for various breach types in which vertical and trapezoidal shapes. But, in this study to simplify the experimental and numerical modeling, two levee-breach cases, such as the vertical abrupt and vertical gradual levee-breach types, were selected. Also, the initial velocity of the channel was assumed to be zero and the flow direction in the reservoir after levee-breach is symmetric against the flow direction at the center of the levee-breach. The width and length of the reservoir are 5.0 m and 30.0 m, respectively. The height of the wall is 0.6 m, and the wall is impervious. The initial water depth of the reservoir was 0.4 m; the inundation area was wetted with a thin layer of about 1 cm water. It was difficult to keep the inundation area into the dry condition because of some leakage of water from the sliding gate and remaining water after every experiment.

Two types of experiments were performed. First one is a levee-breach test on the inundation area without the obstruction. The other one is a test on the same area with the idealized buildings. Frazão and Zech (2008) designed an idealized building group that is composed of the 25 rectangular blocks. The arrangement of the blocks is shown in the Figs. 2 and 3: in Figure a square block arrangement was 5 by 5 aligned with the flow direction, and in Fig. 3 the same arrangement but aligned with the direction of approach flow. Size of each rectangular block is 30 × 30 cm, while the streets are 10 cm wide. A group of blocks is rotated by 22.5° relative to the approach flow direction. Frazão and Zech (2008) studied a flow of the dam-break in an idealized city as like this study, but the flow was influenced by the reflective waves from the left and right side walls. Finally, in their experiments,

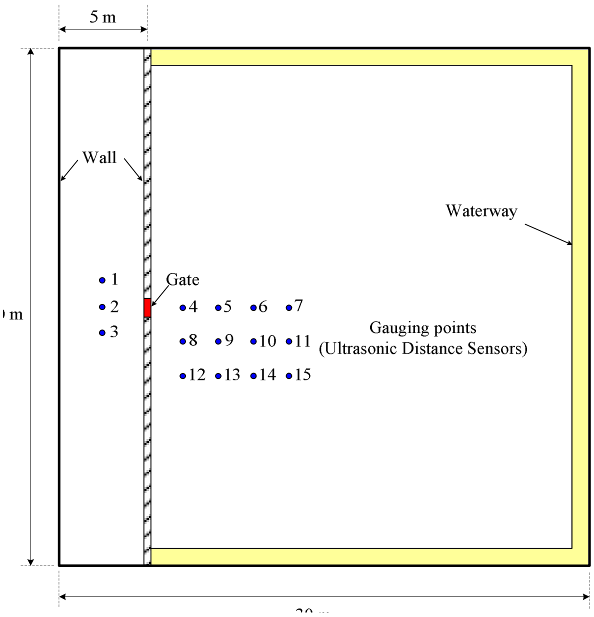


Fig. 1. Sketch of the Experimental Set-up (case 1)

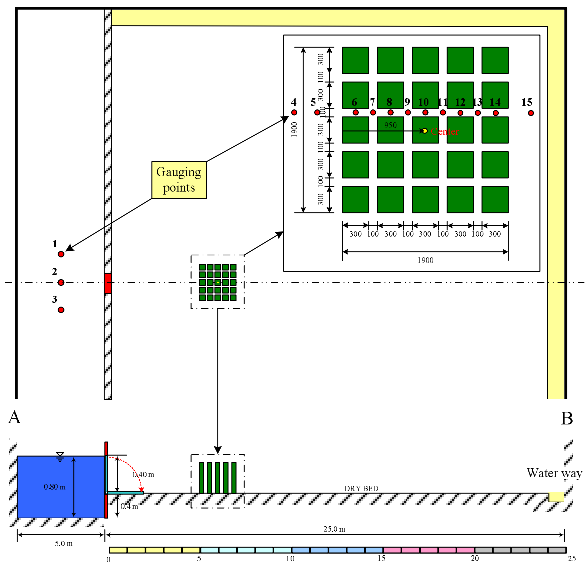


Fig. 2. Sketch of the Experimental Set-up (case 2)

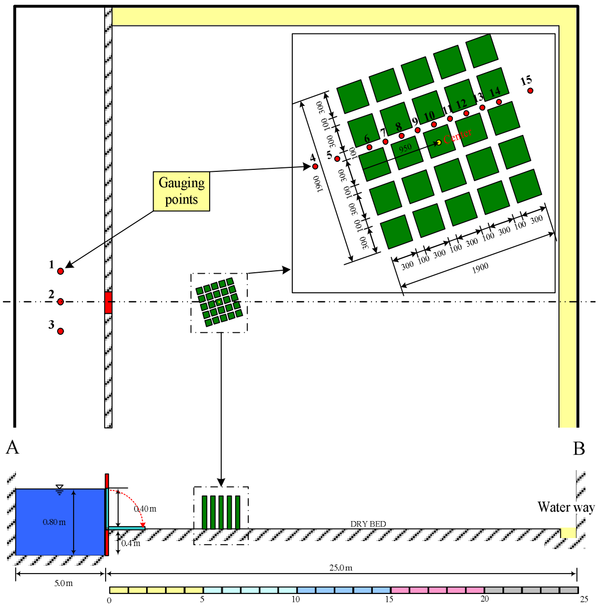


Fig. 3. Sketch of the Experimental Set-up (case 3)

it was impossible to find the flooding depths that is influenced from the dam-break itself. But, this study can focus on the levee-breach flow itself, because an inundation area is wide enough and the water way was installed on the outside boundaries as shown in Fig. 1.

Figs. 1, 2 and 3 show the locations of the gauging points within the laboratory area. Each gauge measured the water depth every 0.2 second (5 Hz) for 300 seconds. In the no block test (case 1), the UDSs were set up on the left side in front of the gate, because the inundation area shows symmetrical on each side from the center of the gate (Fig. 1). In the block test (case 2 and 3), the UDSs were set up on the second row street between the second and the third blocks (Figs. 2 and 3).

2.2 Result (Case1: no blocks)

The levee-breach flow was initially supercritical flow but it changed to subcritical flow as the shock front speed decreased due to bottom friction. As a result, the discontinuous area was occurred at an outer ring where the supercritical changes to the subcritical flow. As the flooding discharge decreases, the velocity decreases, and it was found that the location of the discontinuous area moved toward the gate. Fig. 4 shows the water depth profiles of the abrupt and gradual levee-breach tests for gauges 6 and 7. The overall water depth profile patterns of the abrupt and gradual levee-breach test are very similar, but the differences between results are shown in just after the levee-breach and the starting time of the hydraulic jump.

3. NUMERICAL SIMULATIONS

3.1 Governing equations

Two-dimensional shallow water equations can be derived by integrating the Reynolds equation on the water depth direction. Based on hyperbolic conservation form, the shallow water equations with the hydrostatic pressure assumption omitting the wind and Coriolis effects can be written as Eqs. (1), (2) and (3) (Kuipers and Vreugdenhil, 1973).

$$\frac{\partial h}{\partial t}+\frac{\partial (hu)}{\partial x}+\frac{\partial (hv)}{\partial y}=0 \tag{1}$$

$$\frac{\partial (hu)}{\partial t}+\frac{\partial (hu^2)}{\partial x}+\frac{\partial (huv)}{\partial y}+gh\frac{\partial (h+z_b)}{\partial x} = -\frac{1}{\rho}\tau_{xb}+\nu_t^{h'}\left\{\frac{\partial (hu)}{\partial x}+\frac{\partial (hu)}{\partial y}\right\} \tag{2}$$

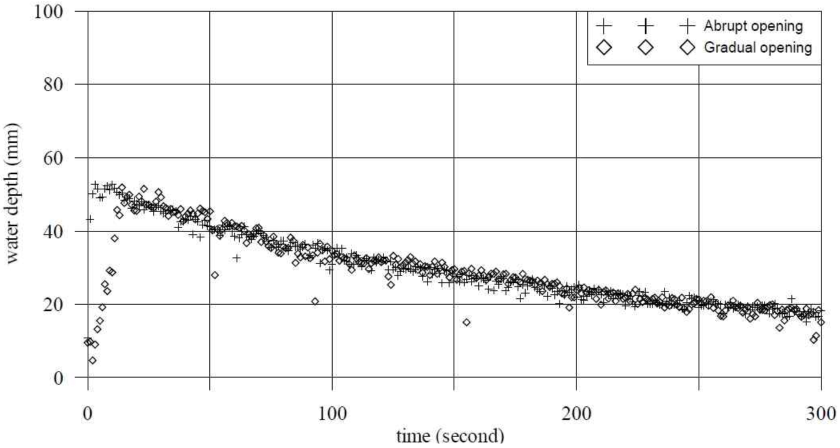
$$\frac{\partial (hv)}{\partial t}+\frac{\partial (huv)}{\partial x}+\frac{\partial (hv^2)}{\partial y}+gh\frac{\partial (h+z_b)}{\partial y} = -\frac{1}{\rho}\tau_{yb}+\nu_t^{h'}\left\{\frac{\partial (hv)}{\partial x}+\frac{\partial (hv)}{\partial y}\right\} \tag{3}$$

where x and y are the independent variables of the flow-direction; h is the water depth; and u and v are the depth-averaged velocity components in the x- and y-directions, respectively; g is the acceleration due to the gravity; z_b is the bottom elevation; ρ is the water density; $\nu_t^{h'}$ is the eddy viscosity; τ_{xb} and τ_{yb} are the bed friction stresses, which can be written as Eqs. (4a) and (4b) using the Chezy coefficient C.

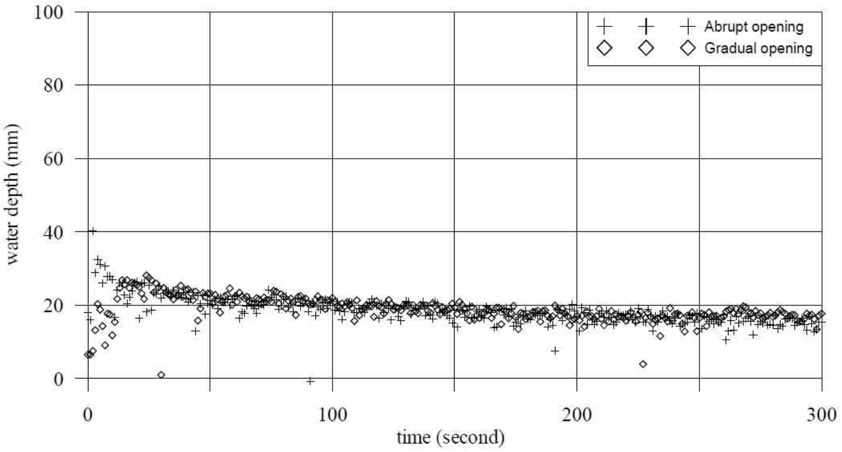
$$\tau_{xb}=\frac{\rho g}{C^2}u\sqrt{u^2+v^2} \tag{4a}$$

$$\tau_{yb}=\frac{\rho g}{C^2}v\sqrt{u^2+v^2} \tag{4b}$$

Turbulence may be defined generally as the effect of temporal variations in velocity, and the momentum exchange associated with their spatial gradient. In order to consider turbulence the Smagorinsky (1963) coefficient was adopted for the horizontal eddy viscosity. $\nu_t^{h'}$



(a) Gauge 6



(b) Gauge 7

Fig. 4. Experimental Measurements (abrupt and gradual levee-breach)

is expressed with Eq. (5).

$$\nu_t^{h'} = C_s^2 \Delta^2 \sqrt{2S'_{ij} 2S'_{ij}} \tag{5}$$

where C_s is a Smagorinsky coefficient adjusted to get the best results for each flow. S'_{ij} is a resolved strain rate tensor and Δ is the grid size.

3.2 HLLC approximate Riemann solver

Two-dimensional numerical model based on a Finite Volume Method was adopted. An intercell flux was reconstructed with the Harten, Lax and van Leer (HLLC) approximate Riemann solver which is modification on the basic HLL scheme. The solution of the Riemann problem consists of three waves with speed separating four constant states as Fig. 5. This scheme was applied to the two-dimensional shallow water equations by Fraccarollo and Toro (1995). In case of the two-dimensional shallow water equations, three eigenvalues can be solved from the Jacobian matrix of the inviscid flux. Eqs. (6a), (6b) and (6c) are the eigenvalues for x- direction.

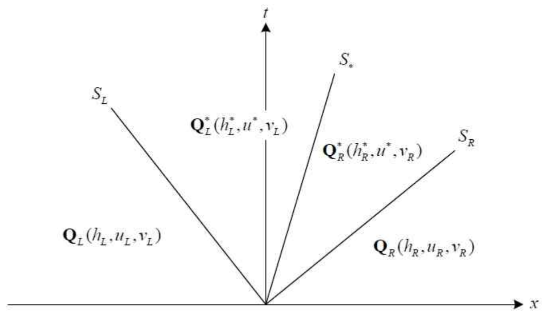


Fig. 5. HLLC Riemann Solver for-Split 2D Shallow Water Equation

$$\lambda_1 = u - \sqrt{gh} \tag{6a}$$

$$\lambda_2 = u \tag{6b}$$

$$\lambda_3 = u + \sqrt{gh} \tag{6c}$$

Assuming all wave speed estimates are available, HLLC numerical flux can be derived as follows for $x-$direction.

$$E_{i+1/2,j}^{HLLC}=\begin{cases}E_L & \text{, for } 0 \le S_L \\ E_L=E_L+S_L(Q_L^*-Q_L) & \text{, for } S_L \le 0 \le S^* \\ E_R^*=E_R-S_R(Q_R-Q_R^*) & \text{, for } S^* \le 0 \le S_R \\ E_R & \text{, for } 0 \ge S_R\end{cases} \qquad (7)$$

where Q_L^* and Q_R^* are given by

$$Q_K^*=h_K^*\begin{pmatrix}1\\ S_*\\ v_K\end{pmatrix}=h_K\left(\frac{S_K-u_K}{S_K-S_*}\right)\begin{pmatrix}1\\ S_*\\ v_K\end{pmatrix} \qquad (11)$$

3.3 TVD version of the WAF scheme

A TVD version of the WAF scheme with SUPERBEE type limiter is applied to control the numerical oscillations in the vicinity of large gradients with second-order numerical accuracy. The WAF flux using the TVD scheme is as follow

$$E_{i+1/2,j}^{TVD-WAF}=\frac{1}{2}(E_{i,j}+E_{i+1/2,j})-\frac{1}{2}\sum_{k=1}^{3}\mathrm{sign}(c_k)\,\phi_{i+1/2,j}^{(k)}\,\Delta E_{i+1/2,j}^{(k)} \qquad (8)$$

in which c_k is the Courant number for a wave speed S_K, $\Delta E_{i+1/2,j}^{(k)}=E_{i+1/2,j}^{(k+1)}-E_{i+1/2,j}^{(k)}$ and $\phi_{i+1/2,j}^{(k)}$ is a WAF limiter function.

3.4 Verification of numerical model

3.4.1 Two-dimensional periodic oscillation tank

Lynch and Gray (1978) derived an exact solution for flow in a square water tank with flat and frictionless bottom. To verify the accuracy of the two-dimensional numerical model periodic oscillation in a square water tank was analyzed. Eqs. (9), (10) and (11) are analytic solutions for water depth and $x-$ and $y-$ directional velocities for periodic wave motion.

$$h(t)=-\frac{\zeta_0}{2}cos\frac{\pi x}{L}cos\frac{\pi y}{L}cos\frac{2\pi t}{T}+h_0 \qquad (9)$$

$$U(t)=-\frac{\zeta_0 gT}{4L}sin\frac{\pi x}{L}cos\frac{\pi y}{L}sin\frac{2\pi t}{T} \qquad (10)$$

$$V(t)=-\frac{\zeta_0 gT}{4L}cos\frac{\pi x}{L}sin\frac{\pi y}{L}sin\frac{2\pi t}{T} \qquad (11)$$

where, ζ_0 is the height of a periodic wave, L is a side length of a square tank, T is the period of flow oscillation which can be calculated by $\sqrt{2}L/\sqrt{gh_0}$. The size of the square tank is 200 × 200 m (L= 200) and the number of $x-$grid and $y-$grid are 201 × 201 (Δx=Δy=1 m), Δt= 0.025 second, ζ_0 = 0.5 m, $T \approx 40.3855$ seconds and h_0= 5.0 m. To find the accuracy of this numerical model, an observation point located at $(x,y)=(L/4,L/4)$ was selected. Figs. 6 and 7 show the compare exact solution, second-order solution and first-order solution of water depth and velocity time histories at the observation point. The results of the second- order numerical model are in excellent agreement with exact solution, but the results of the first-order solution show are dispersion effect with time.

3.4.2 Two-dimensional partial dam-break

Fennema and Chaudhry (1990) used the two-dimensional dam-break problem as shown in Fig. 8. At $t=0$ second, a fictitious thin dam breaks abruptly. All boundaries are treated as a frictionless solid and reflec-

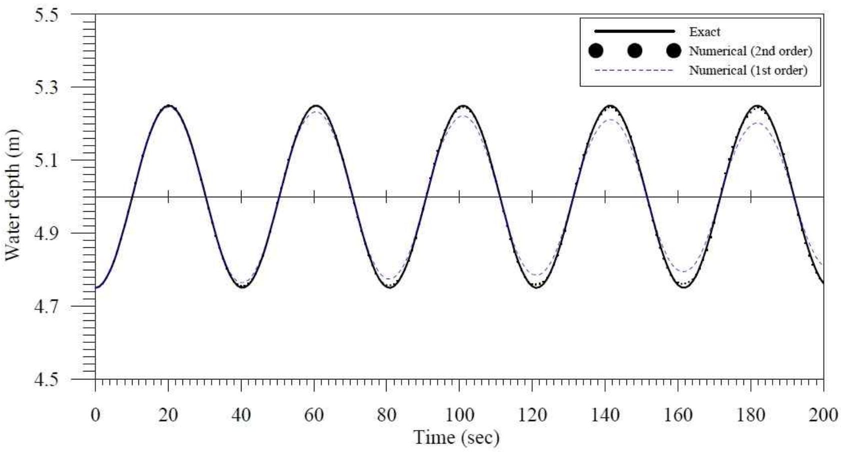


Fig. 6. Comparison of Water Depth Time Histories

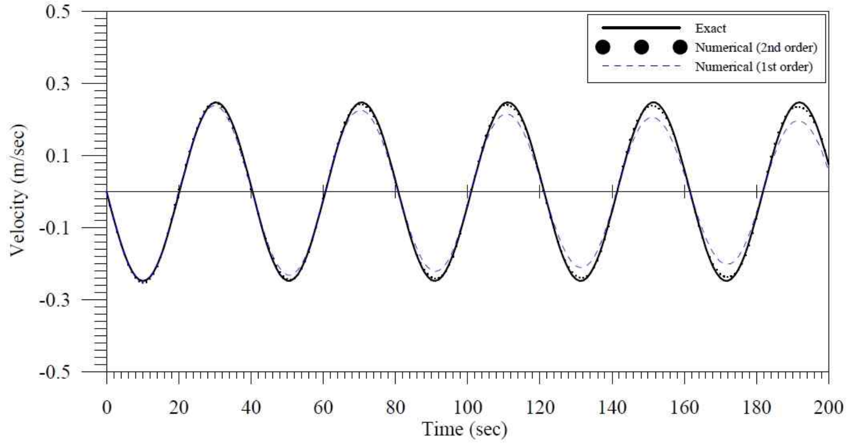


Fig. 7. Comparison of Velocity Time Histories

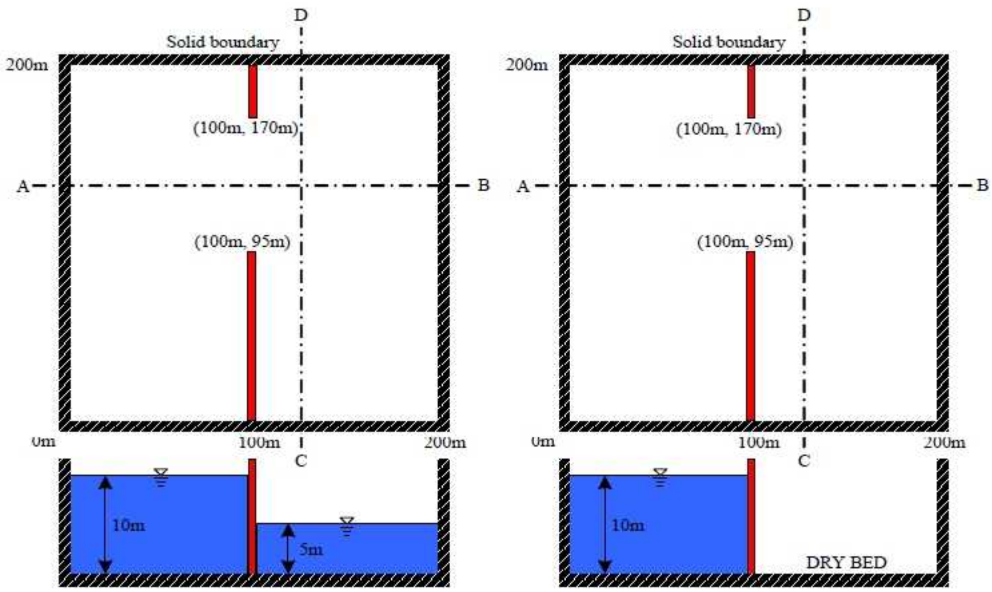


Fig. 8. Two-dimensional Dam-break Problems (Fennema and Chaudhry)

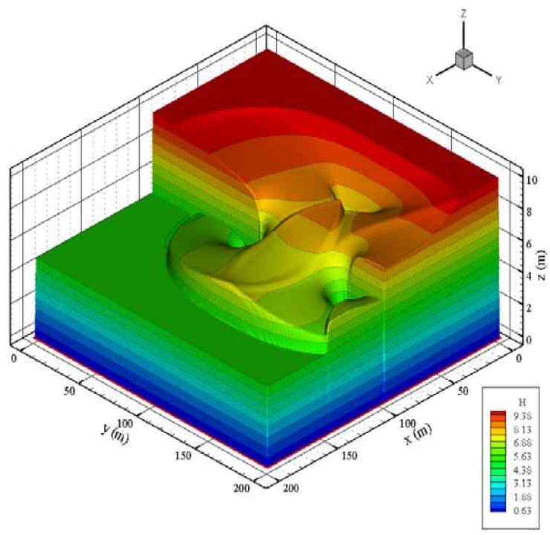


Fig. 9. Wet Bed Problem, t=7.2 sec

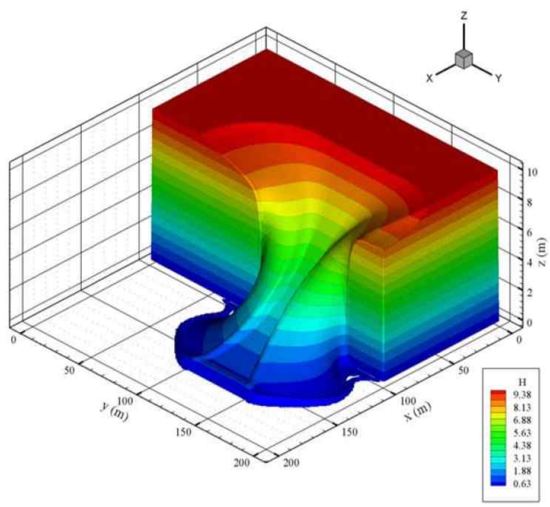


Fig. 10. Dry Bed Problem, t=5 sec

tive wall. The x- and y-grid size is 201×201, Δt is 0.01 second and Manning's roughness coefficient n is zero. Figs. 9 and 10 show the simulation results using the second-order HLLC scheme with TVD scheme after 7.2 seconds and 5.0 seconds, respectively. As shown in Figures the wave speed in dry bed condition was faster than that of the wet bed condition.

4. RESULTS AND DISCUSSION

4.1 Levee-breach flow into horizontal concrete bed (Case 1)

Roughness coefficient is very important factor to analyze a flow of river or inundation area. In this study, the time histories of the computed water depth from the first-order scheme for each gauge were compared with measured water depths for the some roughness coefficient and Smagorinsky coefficients. The optimum roughness coefficient was determined with 0.010. Fig. 11 shows the time histories of the water depth at gauge 4. Comparison was made between measured records and numerical results in the abrupt and gradual tests, respectively. As shown in Fig. 11, the computed results of the first shocks for the abrupt and gradual levee-breach tests show very reasonable agreement with the experimental results. Especially, the arrival time and

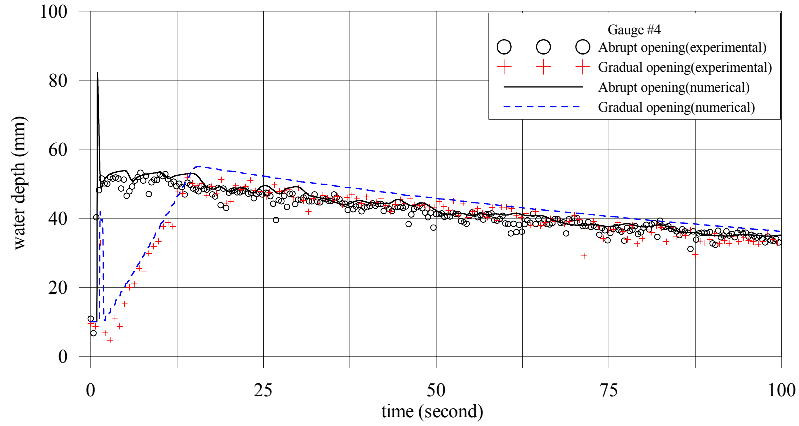


Fig. 11. Abrupt and Gradual Water Depth Profiles (Experimental and numerical)

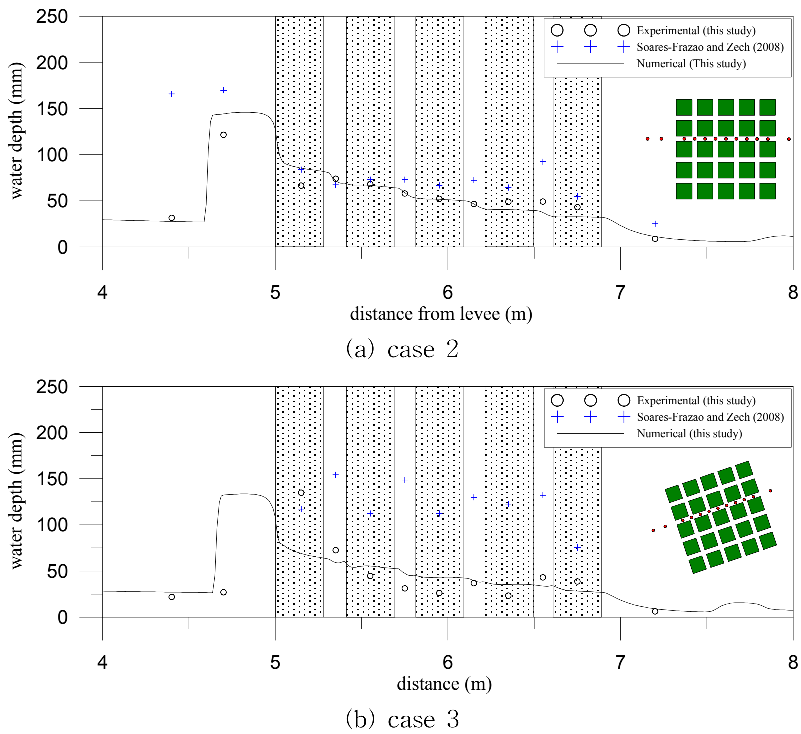


Fig. 12. Abrupt Water Depth Profiles (time = 5 sec)

depth of the first shock were very similar to the numerical results.

4.2 Levee-breach flow into idealized city (Case 2 and 3)

The next test cases involve the idealized city which is aligned (case 2) and rotated by an angle of 22.5° (case 3) with the approach flow direction on the flat area. The dam-break flow through an idealized city was first designed by Frazão and Zech (2008), and their tests were performed in 3.6 m wide laboratory channel. Therefore, their tests were impossible to observe the effects from only dam-break itself due to the reflective wave from the both side walls. Fig. 12 show the water depth profiles in a street for the case 2 and 3. Numerical computations are very well agreed with experimental results, and those of Frazão and Zech (2008) are significantly different from this study due to the reflective wave of the both side wall. In other words, this result shows that the reflective wave influence to the flow into the downstream from the levee. The numerical results effectively replicated the trends of the observed water depth as shown in Fig. 12, but there were little differences for the hydraulic jump. This is a reasonable results because the shallow water equation solver does not have a energy dissipation term for z-direction.

5 CONCLUSION

In this study, abrupt and gradual levee-breach problems were analyzed by laboratory experiments and numerical simulations. In the experimental approaches, the characteristics of the abrupt and gradual levee-breach flow for some experimental conditions were studied. In the numerical approaches, two-dimensional shallow water solver including the effects of horizontal turbulence was developed. Finally, the developed numerical model was applied to the experiments of this study, and its accuracy was verified by comparing with experimental measurements. The numerical simulations developed in this study showed good agreement with the experimental data. However, there were a little difference for the location and height of the hydraulic jump, even if the model considers the Smagorinsky horizontal eddy viscosity. This shows the shallow water equation solver has a limitation to reproduce the energy dissipation due to the hydraulic jump. Further study will be necessary to consider the energy dissipation of the hydraulic jump. Also, it is expected that the levee-breach experiments of this study will give very important data for verifying accuracy of the numerical model.

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REFERENCES

Bradford, S.F., and Sanders, B.F. (2002). Finite-volume model for shallow-water flooding of arbitrary topography. *Journal of Hydraulic Engineering*, Vol. 128, No. 3, pp. 289-298.

Fennema, R.J., and Chaudhry, M.H. (1990). Explicit methods for 2-D transient free-surface flows. *Journal of Hydraulic Engineering*, Vol. 116, No. 8, pp. 1013-1034.

Fraccarollo, L., and Toro, E.F. (1995). Experimental and numerical assessment of the shallow water model for two-dimensional dam break type problems. *Journal of Hydraulic Research,* Vol. 33, pp. 843-864.

Frazão, S., and Zech, Y. (2002). Dam Break in Channels with 90° Bend. *Journal of Hydraulic Engineering*, Vol. 128, pp. 956-968.

Frazão, S., and Zech, Y. (2008). Dam-break flow through an idealized city. *Journal of Hydraulic Research,* Vol. 46, pp. 648-658.

Glaister, P. (1988). Approximate Riemann solutions of the shallow water equation. *Journal of Hydraulic Research*, Vol. 26, pp. 293-300.

Jha, A.K., Akiyama, J., and Ura, M. (1995). First and second order flux difference splitting schemes. *Journal of Hydraulic Engineering*, Vol. 121, pp. 877-884.

Kim, J.Y. (2010). Analysis of Dam Break Flows. Ph.D.

Thesis. Seoul, Korea: Hanyang University, Department of Civil Engineering.

Kuipers, J., and Vreugdenhil, C.B. (1973). Calculation of two-dimensional horizontal flow. *Delft Hydraulics Laboratory Technology*, pp. 1-44.

Liang, Q., Borthwick, A.G.L., and Stelling, G. (2004). Simulation of dam- and dyke-break hydrodynamics on dynamically adaptive quadtree grids. *International Journal for Numerical Methods in Fluids,* Vol. 46, pp. 127-162.

Lynch, D.R., and Gray, W.G. (1978). Analytic solutions for computer flow model testing. *Journal of the Hydraulics Division*, Vol. 104, pp. 1409-1428.

Smagorinsky, J. (1963). General circulation experiments with primitive equations. *Monthly Weather Rieview,* Vol. 91, pp. 99-164.

Toro, E.F. (2001). Shock-capturing methods for free-surface shallow flows. John Wiley, New York.

Vincent, S., and Caltagirone, J.P. (2001). Numerical modeling of bore propagation and run-up on sloping beaches using a MacCormack TVD scheme. *Journal of Hydraulic Research*, Vol. 39, pp. 41-49.

Yoon, K.S. (2008). Analysis of hydraulic characteristics of flood wave on inundation area. Ph.D. Thesis. Seoul, Korea: Hanyang University, Department of Civil Engineering, (in Korean).

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