

A New Approach for Disaster Prediction by Sudden Partial Dam-break Outflow (A Case Study on Amagase Dam)

by

Sayed Masihullah AHMADI*¹ and Yoshimichi YAMAMOTO*²

(Received on Apr. 14, 2021, and accepted on June 19, 2021)

Abstract

A two-dimensional (2D) dam-break hydro-morphodynamics simulation model is presented with a newly proposed dam-break outflow rate equation. The results of the proposed new equation are affected by many factors, among which this study evaluated the influence of the dam-break geometry (width and height of a break section) by conducting many dam-breaks experiments in a long, dry-bed, frictionless and rectangular water channel, connected to a finite water reservoir. The hydro-morphodynamics simulation model solves the continuity equation and two-dimensional non-linear shallow water equations for hydrodynamics calculations, and Rebberink's equation and the continuity equation of bed and suspension loads for morphodynamics calculations by using the finite difference method (FDM). Although it is difficult to collect real dam-break data, because both a flood caused by a dam-break and a flood caused by a tsunami are categorized as longwave floods with substantially similar characteristics, to confirm the validity of the hydro-morphodynamics simulation model, the 2011 Great Tohoku Tsunami was simulated on the Sendai-Natori coast of Japan where the results of the simulation came within an acceptable range. Furthermore, to confirm the validity of the proposed dam-break outflow rate equation, Japan's Amagase Dam (an arched concrete dam) was selected for simulating a break; the results of the simulation showed that a flood wave of more than 5 meters in depth could hit the center of Uji City located downstream of Amagase Dam if this dam were to break.

Keywords: Amagase dam, Sudden partial dam-break, hydro-morphodynamics, FDM.

1. Introduction

Dams are constructed to benefit humans by retaining water for different purposes, however, as water is stored behind a dam, enormous potential energy is formed that in the case of a dam-break, devastating catastrophe may follow, especially, when densely populated cities are located downstream of a dam. Dam-break can happen due to many different reasons, varying from natural causes like mega earthquakes, heavy flash floods, etc., to the aging of the dam, miscalculations, or even human attributes like terrorist attacks, wars, and sometimes, major construction projects can increase the potential threats of a dam-break. (You et al., 2012)¹. Therefore, in this paper, the authors present a simplified dam-break outflow rate concept applicable for break simulations of nonerodable dams assuming a sudden-partial break. The proposed concept is then installed on an existing model that is based on the finite-difference method that can be used by the

researchers and consultants of the field. Therefore, it is the main objective of the current publishing. In the literature, dam-break simulation consists of routing the inflow flood through a reservoir, estimating the dam breach characteristics, and downstream flood routing/modeling ("Using HEC-RAS for Dam Break Studies," 2014)² out of which, the two primary tasks in the hydraulic analysis of a dam-break are the prediction of the reservoir outflow hydrograph and the routing of this boundary condition through the tailwater areas (Pilotti et al., 2010)³. The dam-break outflow hydrograph depends on the construction material of a dam and the type of breach, however, generally speaking, a hydrograph is affected by the bathymetry of a dam, valley shape, dam height, and shape and extension of the breach, therefore, one can find many research reports of the type (Pilotti et al., 2010)³, (Saber and Zenz, 2015)⁴, (Basheer et al., 2017)⁵, (Hakimzadeh et al., 2014)⁶. Even though there is a broad amount of research on the topic of dam-break, but because of the complexity of the phenomenon, it is very difficult to anticipate a dam breach formation. However, in this paper, the authors intend to simulate arch-action concrete dams that fail instantaneously

*1 Graduate student, Architecture and Civil Engineering Course,
Graduate School of Science and Technology,

*2 Professor, Department of Civil Engineering

and this type of breach formation simulation is not covered here. Furthermore, the authors have developed a new dam-break outflow-rate equation. The proposed equation has many variables out of which, the influence of the dam-break height and width on the out-flow hydrograph is investigated through many hydraulic experiments, and a suitable coefficient for this attribute is developed, consequently, this outflow equation is integrated into a two-dimensional flood simulation model of Ca et al. (2010)⁷⁾. This model uses the two-dimensional (2D) non-linear shallow water equations and continuity equation for hydrodynamics calculations and the Rebberink's formula (Rebberink et al., 1998)⁸⁾ for modeling sediment transport. Because the data of a real dam-break is very difficult to achieve (He et al., 2020)⁹⁾, the validity of this model for a long wave flood routing was previously investigated by performing tsunami simulations on the Sendai-Natori coast of Japan, where, it experienced the 2011 Great East Japan Tsunami. (Ahmadi et al., 2020)¹⁰⁾. Moreover, as a case study, the dam-break simulation to Amagase Dam located upstream of a population center of Uji City in Japan is executed and the results depicted that a flood wave of more than 5 meters in-depth, with a velocity of about 6 meters per second will possibly hit the downstream and if proper mitigations are not considered, devastating might follow.

2. Calculations

2.1 Existing Numerical Simulation Model

For the prediction of the topographical change by the tsunami, the numerical simulation model of Ca et al. (2010)⁷⁾ is used. The model was developed using the following detailed formulas.

2.1.1 Numerical Model for Fluid Motion

The numerical model used for tsunami simulations in this paper is based on a continuity equation of fluid (Eq. 1) and two-dimensional nonlinear long-wave equations (Eqs. 2 & 3), and these governing equations are solved by finite difference method using the Crank-Nicholson scheme.

$$\frac{\partial f_y q_x}{\partial x} + \frac{\partial f_x q_y}{\partial y} + \frac{\partial S \eta}{\partial t} = 0 \quad (1)$$

$$\begin{aligned} & \frac{\partial q_x}{\partial t} + \frac{1}{S} \frac{\partial}{\partial x} \left(\frac{S q_x^2}{d} \right) + \frac{1}{S} \frac{\partial}{\partial y} \left(\frac{S q_x q_y}{d} \right) + \\ & g d \frac{\partial \eta}{\partial x} - \frac{1}{S} \frac{\partial}{\partial x} \left[d v_t S \frac{\partial (q_x / d)}{\partial x} \right] - \\ & \frac{1}{S} \frac{\partial}{\partial y} \left[d v_t S \frac{\partial (q_x / d)}{\partial y} \right] + \frac{f_c}{d^2} Q q_x = 0 \end{aligned} \quad (2)$$

$$\begin{aligned} & \frac{\partial q_y}{\partial t} + \frac{1}{S} \frac{\partial}{\partial x} \left(\frac{S q_y q_x}{d} \right) + \frac{1}{S} \frac{\partial}{\partial y} \left(\frac{S q_y^2}{d} \right) + \\ & g d \frac{\partial \eta}{\partial y} - \frac{1}{S} \frac{\partial}{\partial x} \left[d v_t S \frac{\partial (q_y / d)}{\partial x} \right] - \\ & \frac{1}{S} \frac{\partial}{\partial y} \left[d v_t S \frac{\partial (q_y / d)}{\partial y} \right] + \frac{f_c}{d^2} Q q_y = 0 \end{aligned} \quad (3)$$

Here, q_x & q_y are the horizontal fluid fluxes in the x & y directions respectively. η is the water surface elevation. f_x & f_y are the x & y direction ratios of the wet portion in a calculation mesh. S is the area ratio of the wet portion in a calculation mesh. d is the water depth (from the static water surface+ η). g is the gravitational acceleration. v_r is the eddy viscosity coefficient. f_c is the ground surface friction coefficient, and $Q = \sqrt{q_x^2 + q_y^2}$ is the compound value of q_x & q_y . To calculate the eddy viscosity coefficient and the ground surface friction coefficient, the following equations are used;

$$v_r = \varepsilon \left[\left(\frac{\partial U}{\partial y} \right)^2 + \left(\frac{\partial V}{\partial x} \right)^2 \right]^{1/2} \cdot d^2 \quad (4)$$

$$\begin{aligned} f_c &= \frac{g n^2}{d^{1/3}} \quad , \quad n^2 = n_0^2 + 0.020 \frac{B_r}{100 - B_r} d^{4/3} \quad , \\ n_0^2 &= \frac{[(n_1)^2 A_1 + n_2^2 A_2 + n_3^2 A_3]}{A_1 + A_2 + A_3} \end{aligned} \quad (5)$$

Here, U and V are the flow velocity in y and x directions. ε equals (0.1). n is Manning's roughness coefficient. B_r is the building ratio (= the ratio of the area of all vertical objects like houses and trees to the mesh area). n_0 is the weighted average roughness coefficient of areas like farms, roads, and waste & wetlands, $A_{1,2,3}$, with relative roughness coefficients of $n_{1,2,3}$.

2.1.2 Numerical Model for Topographical Change

The topographical change based on sediment transport by the flow can be expressed by using the continuity equation, (Eq. 6).

$$\frac{\partial \zeta}{\partial t} = - \frac{1}{1 - \varepsilon_s} \left(\frac{\partial q_{bx}}{\partial x} + \frac{\partial q_{by}}{\partial y} - C_s + C_{ut} \right) \quad (6)$$

Here, ζ is the ground surface elevation, q_{bx} & q_{by} are respectively the bed-load rate per unit width in x & y directions, C_s is the deposition rate of the suspended load, C_{ut} is the entrainment rate of the suspended load from the bed, and ε_s is the porosity of the sediment. For a detailed

illustration of the equations in this section, please refer to Ahmadi et al. (2020)¹⁰.

• Modeling of q_x & q_y on Bed-load Transport

For evaluation of the bed-load rate, Ribberink's formula (1998)⁸ shown in (Eq. 7) is used. Yokoyama et al. (2002)¹¹ performed many calculations of scouring by flow & wave using indoor and outdoor data on sand & gravel with the diameter range of 0.2~10 mm and found that accurate results can be obtained using this formula.

$$q_{bi} = \begin{cases} C_b [|\theta_s(t)| - \theta_{sc}]^{1.65} \frac{\theta_s(t)}{|\theta_s(t)|} \sqrt{\Delta g D_{50}} & (\theta_s(t) \geq \theta_{sc}) \\ 0 & (\theta_s(t) < \theta_{sc}) \end{cases} \quad (7)$$

Here, q_{bi} is the bed-load transport rate per unit width in the i direction, C_b is the bed-load coefficient determined by verification simulations, however, the authors by performing many hydraulic experiments, developed useful diagrams to acquire the value of this coefficient, Figs. (3-5) (Ahmadi et al., 2020)¹⁰, $\theta_s(t)$ is the Shields parameter in the i direction, θ_{sc} is the critical Shields number, Δ is the relative density of the sand, g is the gravitational acceleration, D_{50} is the median diameter of the sediment.

2.2 New Model of Dam-break Out-flow Rate

The cause for a dam-break is varied in each dam-break case where the most common causes can be great earthquakes, overtopping due to heavy flash floods (this failure impetus is expected to increase in the following decades due to climate changes), and the aging of a dam. A more complete list of the most prominent causes of dam-break are listed in the US Army Corps of Engineers Hydrologic Engineering Center, Flood Emergency Plans, Guidelines for Corps Dams as (1) Earthquake; (2) Landslide; (3) Extreme storm; (4) Piping; (5) Equipment malfunction; (6) Structure damage; (7) Foundation failure; (8) Sabotage (Owen et al., 1980)¹². For every dam failure, there might be a different breach formation shape and time that makes a dam-break simulation and accurate calculations very complex. In the literature, many researchers have so far developed models to simulate the dam breach scenarios on different types of dams and as well as different failure cases. "Methodology for Earthen Dam Breach Analysis" (Yue Sheng et al., 2016)¹³, "Hydrodynamic Dam Breach Modelling of Earthfill Saddle Dam" (Sidek, 2011)¹⁴, "Analysis of Hydrograph by Dam Breach Shapes" (ki-bum Park, 2019)¹⁵ are some of the researches which concentrate more on the failure shapes and the discharge rates of different dam type's failure with various break sections, however, in

this paper, the main focus is flood routing and morphodynamics calculations following a dam-break. For this purpose, we have assumed a trapezoidal break section for a dam-break shown in Figs. 1&2 and derived new equations for calculating a run-off rate to this dam-break shape as illustrated in the following:

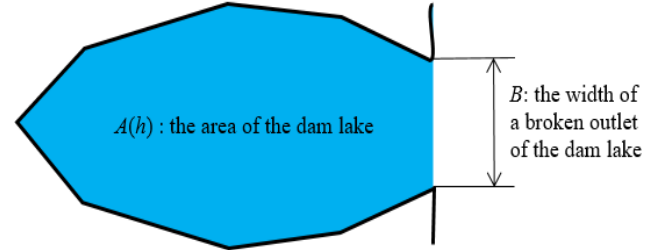


Fig. 1 Plan view of the dam lake.

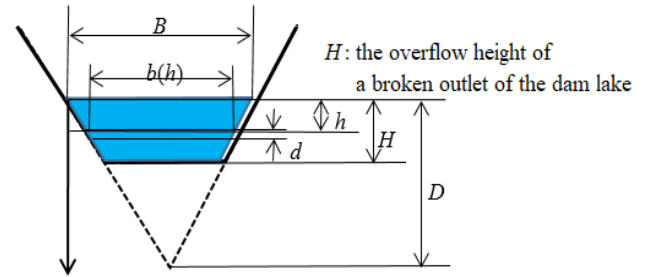


Fig. 2 Dam-break section view.

The run-off rate from the dam lake in the case that the overflow height H is constant (water supply is infinite) can be calculated from bellow equations;

$$Q_r = \int_0^H b(h) \times c \sqrt{2gh} dh \quad (8)$$

$$= \int_0^H \frac{B(D-h)}{D} c \sqrt{2gh} dh$$

$$Q_r = \frac{B}{D} c \sqrt{2gh} \left[D \frac{2}{3} h^{3/2} - \frac{2}{5} h^{5/2} \right]_0^H \quad (9)$$

$$Q_r = \frac{B}{D} c \sqrt{2g} \left(\frac{2}{3} D H^{3/2} - \frac{2}{5} H^{5/2} \right) \quad (10)$$

Here, Q_r is the discharge rate.

Since the case of finite water supply from a dam lake is more realistic in the actual dam-break cases, in this paper, we have used the equation for run-off rate from the dam lake by the following equations;

$$Q_r = A(h) \times \frac{dh}{dt} \quad (11)$$

$$Q_r = \frac{B}{D} c \sqrt{2g} \left(\frac{2}{3} D \left(H - \frac{dh}{dt} T \right)^{3/2} - \frac{2}{5} \left(H - \frac{dh}{dt} T \right)^{5/2} \right) \quad (12)$$

Here, T is the run-off elapse time (sec), V is the total volume of discharge water, and c is the flow rate coefficient.

$$V = \int_0^H A(h) dh = \int_0^{\frac{H}{\frac{dh}{dt}}} Q dT \quad (13)$$

$$V = \int_0^{\frac{H}{\frac{dh}{dt}}} \frac{B}{D} c \sqrt{2g} \left(\frac{2}{3} D \left(H - \frac{dh}{dt} T \right)^{3/2} - \frac{2}{5} \left(H - \frac{dh}{dt} T \right)^{5/2} \right) dT \quad (14)$$

$$= \frac{B}{D} c \sqrt{2g} \frac{-1}{\frac{dh}{dt}} \int_H^0 \left(\frac{2}{3} D X^{3/2} - \frac{2}{5} X^{5/2} \right) dX \quad (15)$$

$$= \frac{B}{D} c \sqrt{2g} \frac{-1}{\frac{dh}{dt}} \left[\frac{4}{15} D X^{5/2} - \frac{4}{35} X^{7/2} \right]_H^0 \quad (16)$$

$$\text{Here, } X = H - \frac{dh}{dt} T, \frac{dX}{dT} = -\frac{dh}{dt} \quad (17)$$

$$T = 0 \rightarrow T = \frac{H}{\frac{dh}{dt}}, X = H \rightarrow X = 0 \quad (18)$$

Thus,

$$\therefore V = \frac{B}{D} c \sqrt{2g} \frac{+1}{\frac{dh}{dt}} \left(\frac{4}{15} D H^{5/2} - \frac{4}{35} H^{7/2} \right) \quad (19)$$

$$\therefore \frac{dh}{dt} = \frac{B c \sqrt{2g}}{D V} \left(\frac{4}{15} D H^{5/2} - \frac{4}{35} H^{7/2} \right) \quad (20)$$

$$\therefore T_{total} = \frac{H}{\frac{dh}{dt}} = \frac{D V}{B c \sqrt{2g}} \frac{1}{\frac{4}{15} D H^{5/2} - \frac{4}{35} H^{7/2}} \quad (21)$$

Here, D is the imaginary depth from the dam crown height and it is assumed many times the depth of a dam in this simulation.

3. Evaluation Methods of Empirical Coefficients

3.1. Rational Evaluation of the Coefficient of Bed-load Transport Rate

To calculate the bed-load transportation, the model of Ca et al. uses Rebberink's formula (Eq. 7). To use this equation, one needs to calculate its c_b value using the graphs provided in Figs. 3-5 as well as (Eq. 22). The presented graphs and illustrations of their production and use are detailed in the research of Ahmadi et al. (2020)¹⁰.

$$C_b = C_{bo} \times C_1 \times C_2 \quad (22)$$

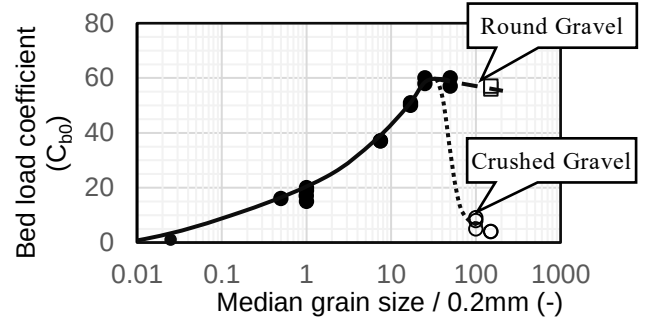


Fig. 3 Influence of the median grain size to the bed-load coefficient.

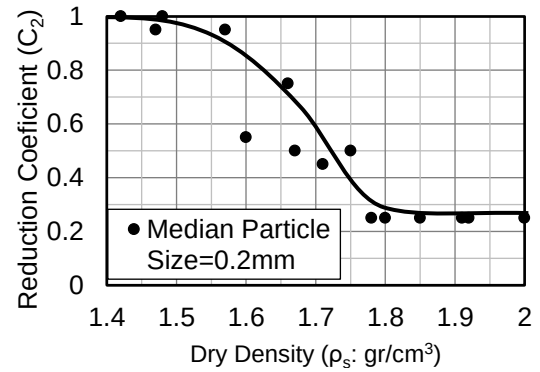


Fig. 4 Influence of dry density to the bed-load reduction coefficient (the median grain size & the uniformity coefficient are 0.2 mm & 20.1).

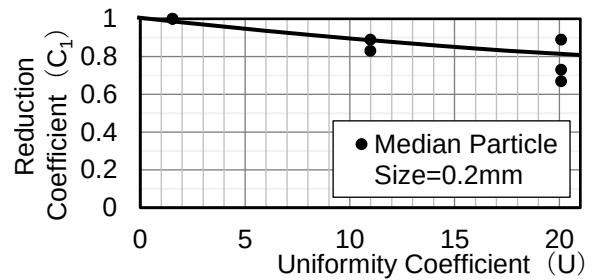


Fig. 5 Influence of the uniformity coefficient to the bed-load reduction coefficient (the median grain size is 0.2 mm, dry densities are around 1.5 g/cm³).

3.2 Evaluation of the coefficient of the outflow rate

For the newly proposed outflow rate equation, (Eq. 12), the coefficient c has a great impact on a calculated hydrograph. This coefficient depends on many variables i.e., dam bathymetry, type of break, break dimensions, and dam width and height. Initially, the dam-break section's impact on this coefficient is evaluated in this paper, by performing many hydraulic experiments. Figs. 6 & 7 shows the apparatus and experimental arrangements respectively, on the left-hand side of the apparatus, there is a water tank reserve as a dam reservoir, at the downstream, there is a dry-bed water channel of 0.5m in width, 0.5m in height, and 9.5 m in length. At the one-meter section from the reservoir, two layers of acryl plates of one centimeter in thickness are placed. The two plates are arranged as one of them is cut to the desired section and the other one acts as a gate. The first acryl plate is glued to the water channel by silicon and the other acryl plate (the gate) is placed before the first plat and there, grease is used as a lubricant in between the plates to let us open the gate abruptly. This complies with our objective of sudden partial dam-break. From these experiments, we intend to develop the hydrographs, therefore, using a digital camera that is focused on the scale bar located about 50 centimeters upstream of the acryl plates, the whole process of the experiments is recorded.

Using this arrangement, one can measure the change in water height behind the dam, and because previously, the bathymetry measurements (reservoir surface area) are taken, the discharge rate is calculated. Fig. 8, shows the calculated and measured hydrographs. For calculated hydrographs, the c value is set as to make the best fit curve in respective to the measured data best-fit curve, as a result, the best value of c is acquired. The experimental data and their results are shown in Table 1.

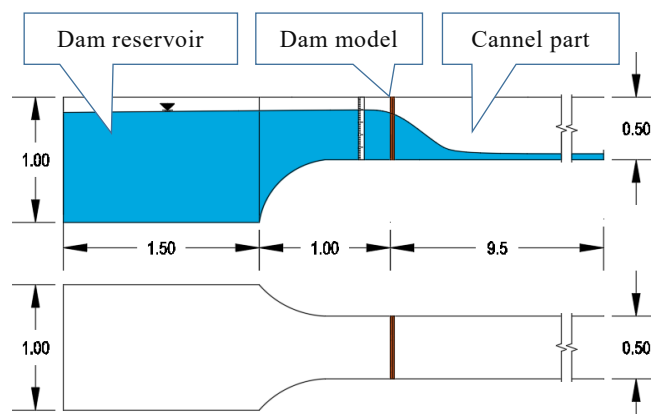


Fig. 6 Dam-break experiment apparatus.



Fig. 7 Dam-break experiment in a rectangular channel.

Because in each of the experimental cases, a different value of c is acquired, an average value of this coefficient is calculated to be 0.28 and It is used for dam-break simulations of the Amagase dam illustrated in the subsequent sections of this paper.

Table 1 Dam-break experiment data and results.

Case	Description	Break dimensions			c
		B (m)	H (m)	Break section thickness (cm)	
1.1	Trapezoidal	0.3	0.25	1	0.30
1.2	Trapezoidal	0.3	0.25	1	0.30
2.1	Trapezoidal	0.3	0.25	1	0.20
2.2	Trapezoidal	0.3	0.25	1	0.20
3.1	Trapezoidal	0.2	0.3	1	0.35
3.2	Trapezoidal	0.2	0.3	1	0.35
4	Triangle	0.2	0.3	0.2	0.50

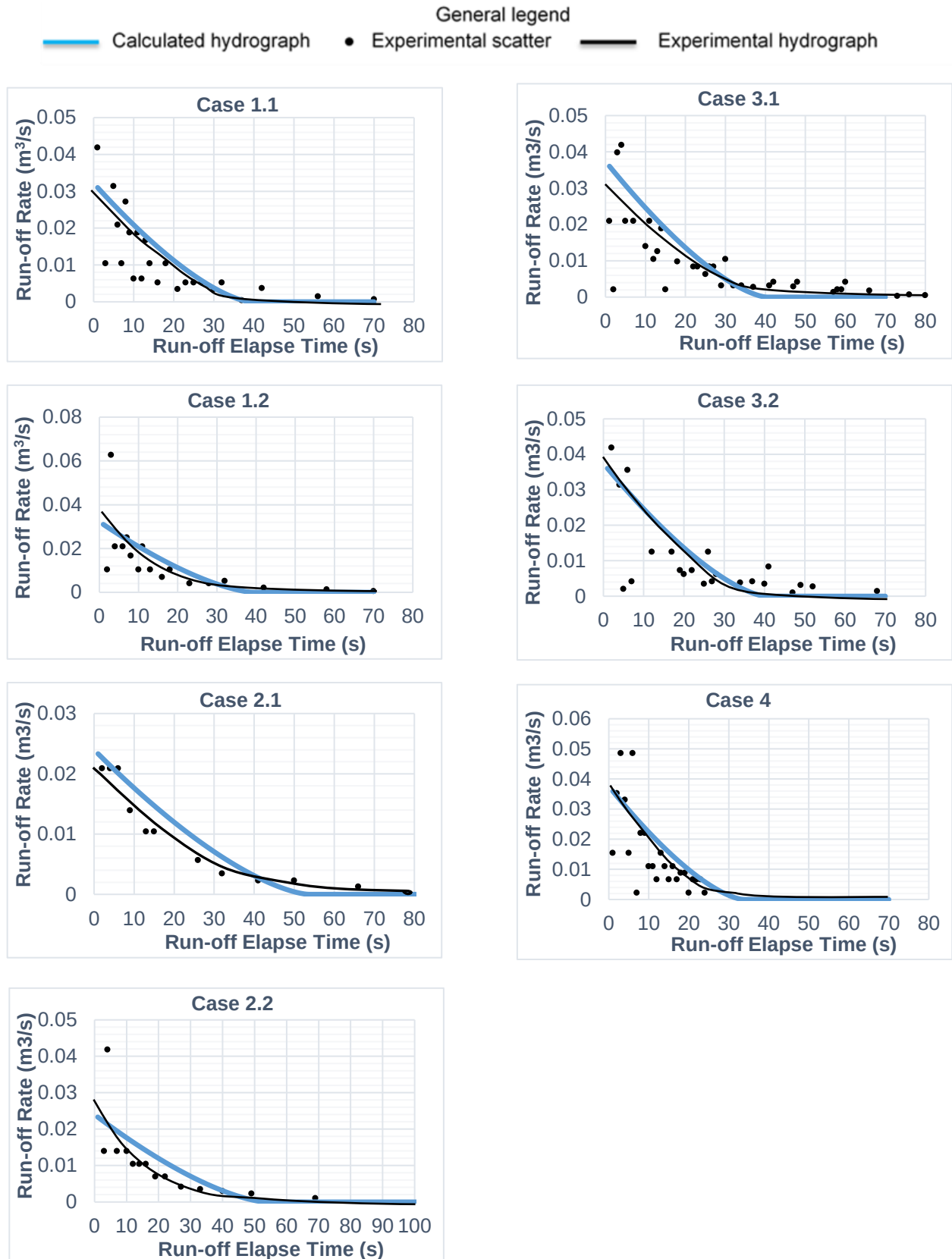


Fig. 8 Dam-break hydrographs from experimental cases and calculated using Eq. 12 (Case 1-4).

4. Verification Simulations

4.1 Hydro-morphodynamics Simulation in Sendai-Natori Coast

Sendai-Natori coast is located in the Miyagi prefecture of Japan. This coast experienced a magnitude 9.0 earthquake and a consequent catastrophic great tsunami in 2011. The authors executed many hydro-morphodynamics simulations to replicate the actual flooding conditions and presented the results on the paper (Ahmadi et al., 2020)¹⁰. The authors also showed in their paper that the results of the simulations are in good agreement with the actual measured data from the area as shown in Fig. 9(a), random spot elevation-change points are selected and their relative data are plotted from both measured data and the reproduction data along line (A) shown on Fig. 9(b). For the locations where measured data is available, the calculated data matches perfectly with the measured data, however, for some locations like the seaside or underwater parts, because there is no digital elevation model data available, one cannot decide whether it differs from the actual topography change situation.

4.2. Dam-break Hydro-morphodynamics Simulation of Amagase Dam

Amagase Dam is located in Uji City in Kyoto Prefecture of Japan shown in Fig. 10. It is an arch dam made of concrete, built on the Uji-Gawa river shown in Fig. 11. The Amagase dam's construction started in 1955 and finished in 1964. This dam is mainly constructed for flood control and It also serves as a great water supply reservoir for the Uji City residents. The catchment area of the dam is 4200 square kilometers and its design volume is 122 thousand cubic meters. The height of the dam is 73 meters with a crest length of 254 meters. The reason behind why this dam is chosen for simulation is, about 200 thousand people live downstream of the dam and if in a worst-case scenario this dam, breaks, it would be catastrophic. Therefore, in this paper, it is intended to prepare a hazard map to depict the inundation height, velocity, scouring, and deposition depths for the possible affected area. For the proposed dam-break simulation, the following information is required as input data sets; (1) Assumed broken width and height, (as listed in Table 2). (2) Proposed dam's reservoir area, (as listed in Table 2). (3) Water head behind the dam, (as listed in Table 2). (4) Calculation meshes size, (as listed in Table 2). Because the proposed model of Ca et al. (2010)⁷ is based on the finite difference method (FDM), the mesh size limitation is always a concern which mainly depends on the capacity of the available computer, size of the proposed dam, and required accuracy. (5) Existing topography contour map as shown in Fig. 12, in this research, a 10-meter accuracy digital elevation map (DEM) is used. (6) The building and trees ratio in a mesh

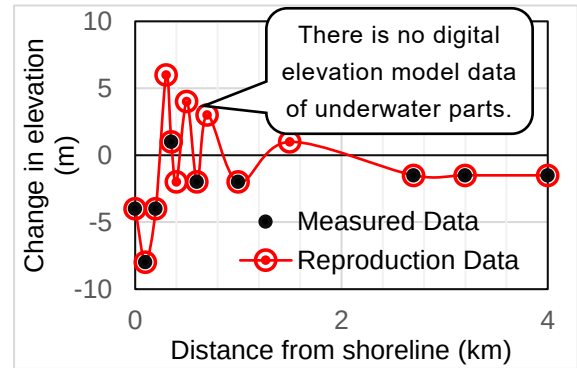


Fig. 9(a) Relative accuracy of the topography-change along line (A) on the measured and reproduced simulation results.

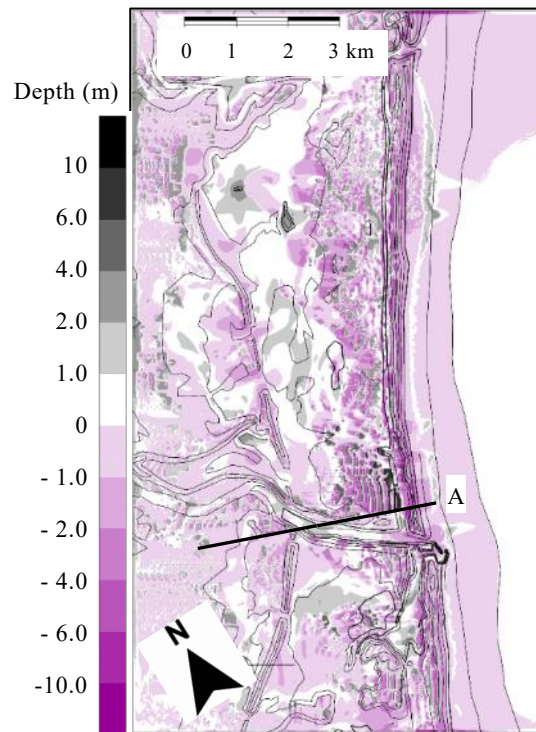


Fig. 9(b) Topography change results (Total bed - load), 48 minutes from start of the tsunami calculations.

as shown in Fig. 13, the light green color (the building ratio is 1 % in a mesh area) is referend to a soil and sand area, the dark green color (5 % in a mesh area) shows a wood area and a forest area, the light brown (30 % in a mesh area) shows a house with a garden, the gray color area (70 % in a mesh area) shows a building area, (7) Grain size distribution in the area, and (8) Calculation limit boundary.

Table 2 Input data for dam-break simulation.

Description	Value
Broken width and height (m)	100 X 50
Broken area (sq. m)	2500
Dam reservoir area (sq. m)	1'880'000
Water height behind the dam (m)	68
Mesh size (m)	20 X 20

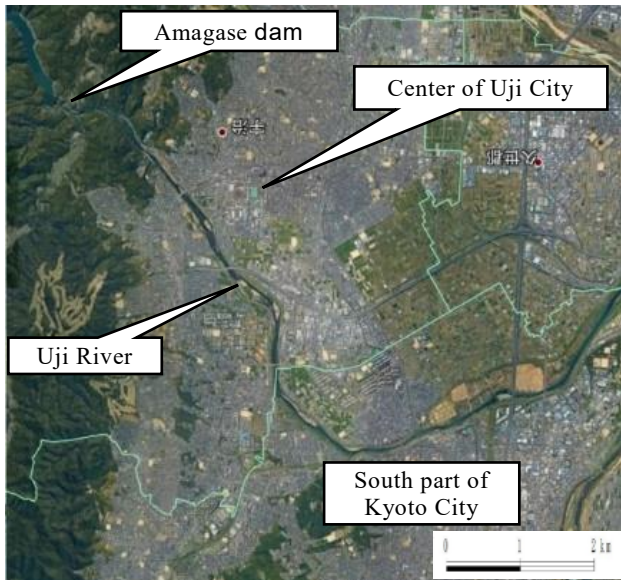


Fig. 10 Locations of Amagase Dam and the high-density zone of the population of Uji City (the gray zone is the circumference of “Center of Uji City”).



Fig. 11 A photo of Amagase Dam (the dam height is 73m, the dam width is 254m).

The results of the Amagase dam-break simulation are shown in Figs. 14-22. Scouring depths and deposition heights are depicted in Figs. 14-16. The maximum scouring depth is about 10 meters close to the dam and it decreases to about 5 meters as the bore reaches further locations from the dam. Forward and return velocity distributions are depicted in Figs. 17-19, the maximum forward velocity is calculated to be about 6 meters per second and the maximum return velocity is found

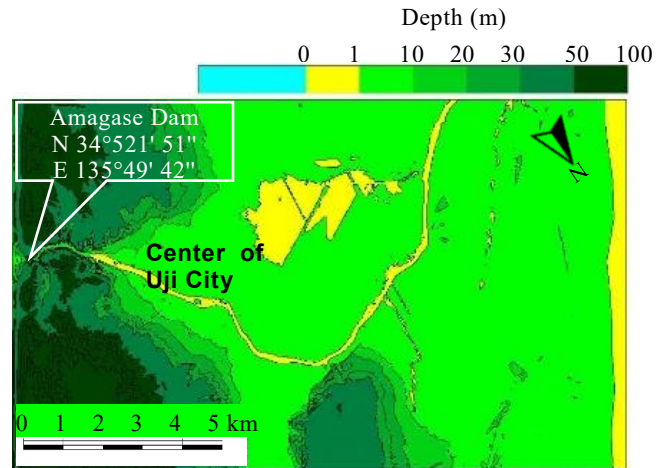


Fig. 12 Existing topography – Downstream of Amagase dam (Uji City).

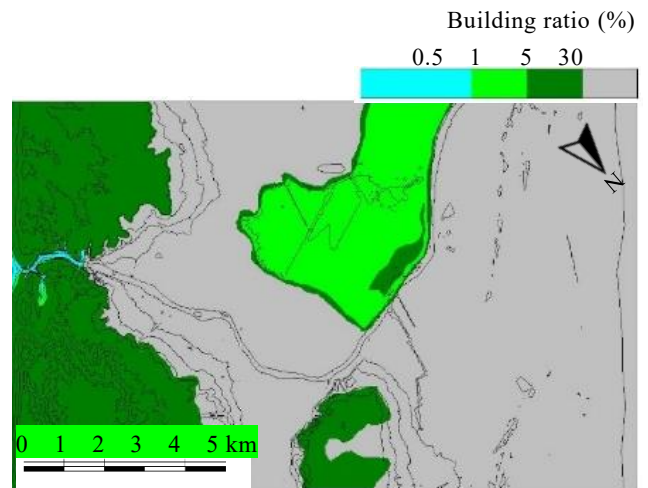


Fig. 13 Building ratio in a calculation mesh (= the area ratio of buildings & trees in a mesh).

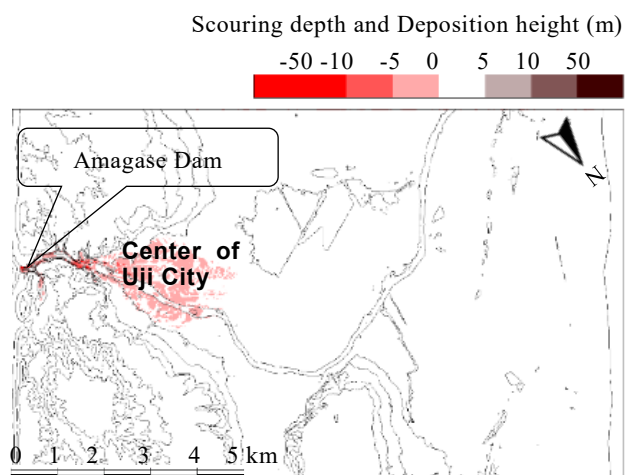


Fig. 14 Topography change, 10 minutes after the start of the calculation.

to be 3 meters per second. Inundation height and extent are depicted in Figs. 20-22.

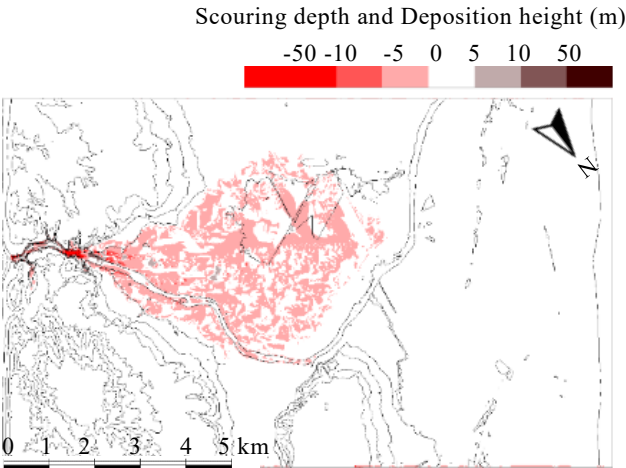


Fig. 15 Topography change, 20 minutes after the start of the calculation.

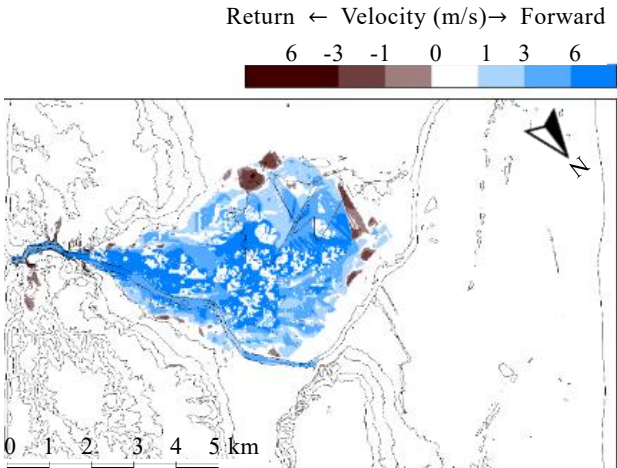


Fig. 18 Inundation velocity 20 minutes after the start of the calculation.

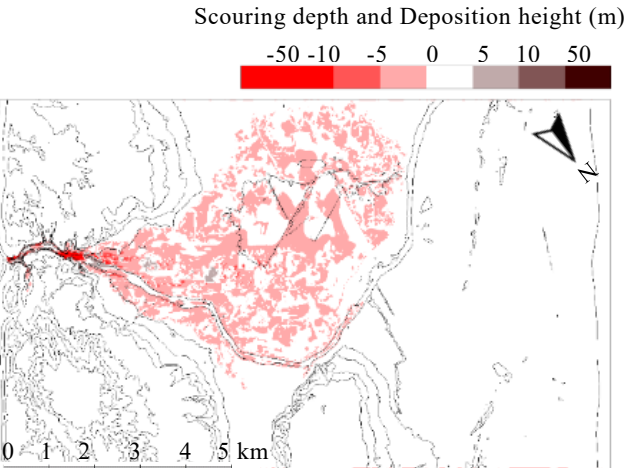


Fig. 16 Topography change, 30 minutes after the start of the calculation.

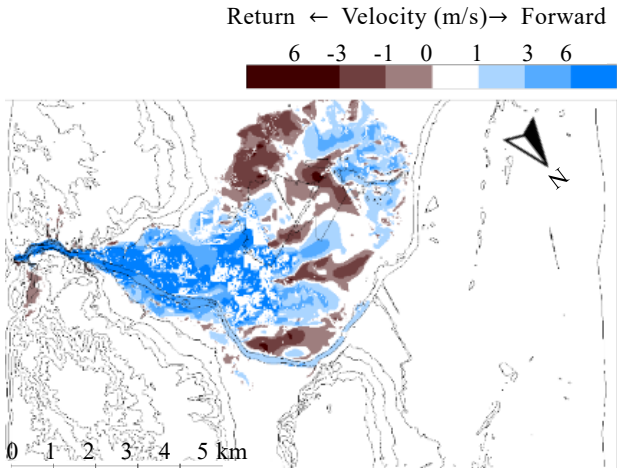


Fig. 19 Inundation velocity 30 minutes after the start of the calculation.

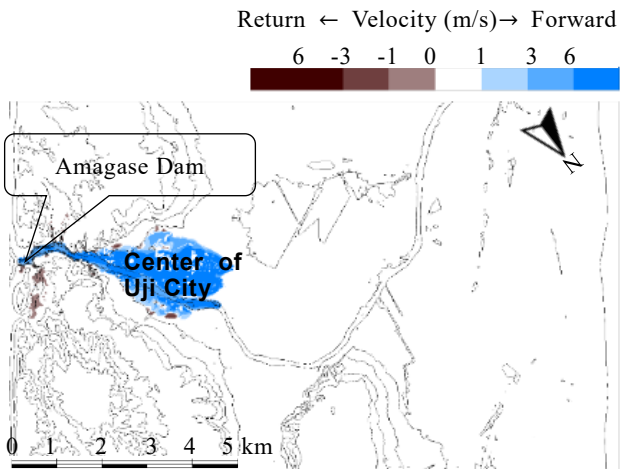


Fig. 17 Inundation velocity, 10 minutes after the start of the calculation.

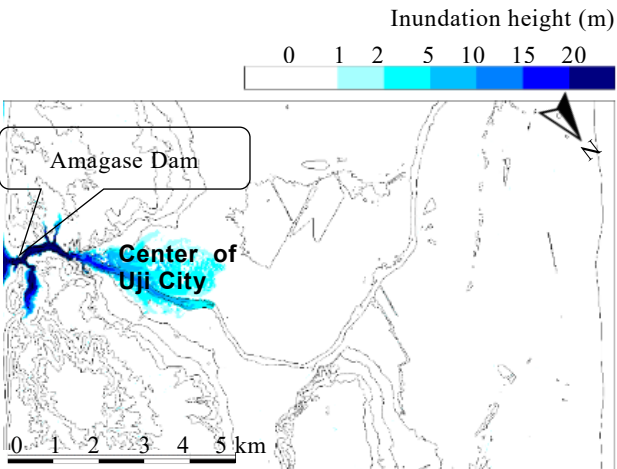


Fig. 20 Inundation height, 10 minutes after start of the calculation.

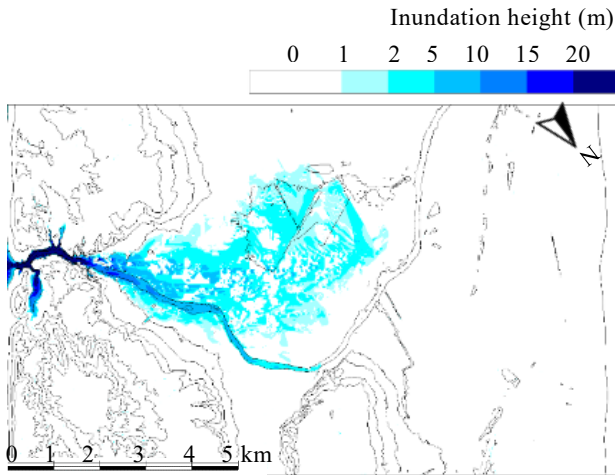


Fig. 21 Inundation height, 20 minutes after start of the calculation.

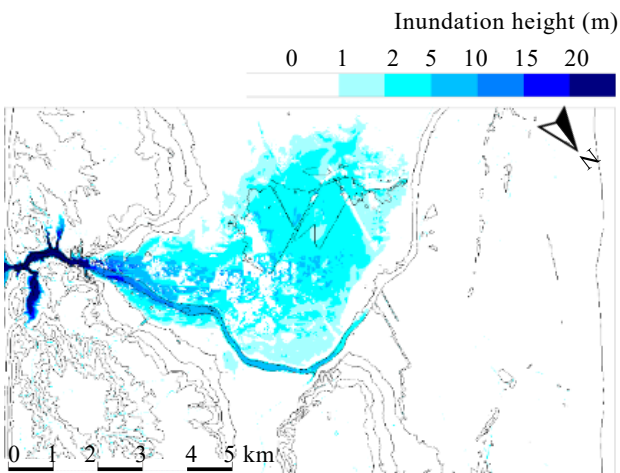


Fig. 22 Inundation height, 30 minutes after start of the calculation.

5. Conclusion

A new approach towards a dam-break hydro-morphodynamics simulation is present with a simple and user friendly manner. Furthermore, a new dam-break outflow rate equation is also presented. The proposed outflow rate equation's output depends on many variants, i.e., bathymetry, dam height, breach shape, and characteristics, etc., out of which, the inference of a dam breach shape (break section's height and width) are evaluated by performing many hydraulic experiments in a rectangular water channel. By comparing the experimental and calculated hydrographs, and trial-error practice, a best fitting curve is acquired with a proper coefficient c , consequently, by integrating the proposed outflow rate equation on a two-dimensional flood simulation model, and simulating the Amagase dam-break, it is perceived that this equation can be potentially useful for the preparation of dam-break flood routing. According to the simulation

results of the Amagase Dam presented in this paper, thousands of lives, as well as a countless amount of economy, is in a high-risk zone in case of a dam-break, therefore, the authors suggest that the households living downstream of the Amagase dam and in the high-risk zone, should be informed of the potential danger. In the case of superannuated dams in the area where the frequency of Mega Earthquakes is high, a brief hazard map would be useful for all dams in the area especially the Amagase dam that a densely populated city is right at its downstream. By widening the Amagase dam and by the improvement of dikes at certain locations, the risk of calamity can be lowered.

References

- 1) L. You, C. Li, X. Min, and T. Xiaolei: Review of Dam-break Research of Earth-rock Dam Combining with Dam Safety Management. *Procedia Eng.* 28 (2012), 382–388, <https://doi.org/10.1016/j.proeng.2012.01.737>.
- 2) US Army Corps of Engineers (USACE): *Using HEC-RAS for Dam Break Studies*, (Institute for Water Resources Hydraulic Engineering Center, 609 Second Street, Davis, Ca 95616-4687, 2014).
- 3) M. Pilotti, M. Tomirotti, G. Valerio, and B. Bacchi: Simplified Method for the Characterization of the Hydrograph following a Sudden Partial Dam Break, *J. Hydraul. Eng.* 136 (2010), 693–704, [https://doi.org/10.1061/\(ASCE\)HY.1943-7900.0000231](https://doi.org/10.1061/(ASCE)HY.1943-7900.0000231).
- 4) O. Saberi and G. Zenz: Empirical Relationship for Calculate Outflow Hydrograph of Embankment Dam Failure due to Overtopping Flow, *Int. J. Hydraul. Eng.* 9 (2015), <https://doi.org/10.5923/j.ijhe.20150403.01>.
- 5) T. A. Basheer, A. Wayayok, B. Yusuf, and R. Kamal: Dam breach parameters and their influence on flood hydrographs for Mosul dam, *J. Eng. Sci. Technol.* 12 (2017), 14.
- 6) H. Hakimzadeh, V. Nourani, and A. B. Amini: Genetic Programming Simulation of Dam Breach Hydrograph and Peak Outflow Discharge, *J. Hydrol. Eng.* 19 (2014), 757–768, [https://doi.org/10.1061/\(ASCE\)HE.1943-5584.0000849](https://doi.org/10.1061/(ASCE)HE.1943-5584.0000849).
- 7) V. T. Ca, Y. Yamamoto, and N. Charusrojthanadech: Improvement of Prediction Methods of Coastal Scour and Erosion due to Tsunami Back-flow, *Proc. 20th Intl. Offshore and Polar Engineering Conference* (2010), 1053-1060.
- 8) J. S. Ribberink: Bed-load Transport for Steady Flows and Unsteady Oscillatory Flows, *Coastal Engineering* 34 (1998), 59-82.
- 9) G. He, J. Chai, Y. Qin, Z. Xu, and S. Li: Evaluation of Dam Break Social Impact Assessments Based on an

- Improved Variable Fuzzy Set Model, *Water* 12 (2020), <https://doi.org/10.3390/w12040970>.
- 10) S. M. Ahmadi, Y. Yamamoto, and V. T. Ca: Rational Evaluation Methods of Topographical Change and Building Destruction in the Inundation Area by a Huge Tsunami, *J. Mar. Sci. Eng.* 8 (2020), <https://doi.org/10.3390/jmse8100762>.
 - 11) Y. Yokoyama, H. Tanabe, K. Torii, F. Kato, Y. Yamamoto, J. Arimura, and V. T. Ca: A quantitative valuation of scour depth near coastal structures, *Proc. ICCE2002* (2002), 1830-1841.
 - 12) H. J. Owen: *Flood Emergency Plans: Guidelines for Corps Dams*, (Fort Belvoir, VA, Defense Technical Information Center, 1980), <http://www.dtic.mil/docs/citations/ADA158679>.
 - 13) S. Yue, E. Damisse, and M. Ansar: Methodology for Earthen Dam Breach Analysis, *World Environmental and Water Resources Congress*, May 22–26, West Palm Beach, Florida (2016).
 - 14) L. Y. Sidek1, F. C. Ros, N.N.N. Ibrahim, A.Z.A. Razad, M.N. Ahmad: Hydrodynamic Dam Breach Modelling of Earthfill Saddle Dam, *Journal of Energy & Environment* 3 (2011) No. 1, 28-32.
 - 15) Ki-Bum Park: Analysis of Hydrograph by Dam Breach Shapes, *Journal of the Environmental Sciences* 16 (2007) No 4, 487-493.