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Flash Flood Flow Simulation, the Case of Situ Gintung Dam Failure, Indonesia

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Abstract. The disaster caused by dam failure causes a lot of casualties. The flash flood caused by the failure may propagate downstream and destroy every things along the reach to some distance. In order to understand such flood, a numerical simulation study is conducted. The failure of Situ Gintung Dam that was recently occured can be used for comparison. Simulation for other dams, therefore, can also be conducted for preparing the warming system necessary to be set.

Simulation of flush flood is the peculiar one. This because the degree of non-linearity of the phenomenon is high and the Froude number of the flow is also high. This paper presents an experience in conducting 2D numerical simulation study of the flash flood propagation downstream of the Situ Gintung dam using FESWMS software. A 1D numerical simulation study using HEC-RAS 4.0 Beta version has been done and presented in Rahardjo, et al, 2009.

Keywords. Dam break, collapse, hydrodynamic simulation, numerical model, flash flood, Situ Gintung Dam

1. Introduction

Most of the failure of dams have caused disasters involving significant lost of properties and even there were always lost of lives. In the design stage of any dam, now days, there is a requirement to evaluate the effect of the failure of the dams along the downstream valley in case of the dam collapses. In the region of earthquake prone area, the risk of the collapse is higher. Therefore, the analysis becomes very important. The dam break analysis is focus on finding the area to where the flood triggered by the collapse of the dam will inundate and have high probability causing any kind of destruction. The results of the analysis are used to take preventive measures to minimize losses in case the failure occurs.

Several small dams in the Special Province of Capital City, Jakarta were built decades ago. Some of them were built by Dutch before the independence of Republic of Indonesia. These dams have no such flood analysis until now. Most of them, currently, have densely populated downstream valley. Therefore, the flood hazard become a very serious matter for those areas. This threat is currently becoming to be of the growing concern among the authorities and those people living in such valleys, since the collapses of Situ Gintung Dam in the dawn of 27th, March, 2009. Moreover, within the year of 2009, several earthquake attacks occur at Sumatra, Java, West Papua, and other smaller islands, one of them hit Padang, the capital city of West Sumatra Province. The risk of dam failure becomes a real threat.

2. The Collapse of Situ Gintung Dam

Situ Gintung Dam collapse was preceeding by a heavy storm pouring its catchment area the day before the disaster occurred. It was reported that reservoir water started to spill at about the sun set time. At the midnight, cracks were reported started to be heard by the people living downstream of the dam. At about two o’clok, the bridge above the spillway collapsed. It was also reported that a big pool was created at the toe of the spillway at that time. It seems that the spillway chute and its downstream end that supposed to be the energy dissipator could no longer withstand with the spilled water discharge. Two hours later, a reach of the dam including the spillway of about 30 m long was collapsing. The flood then flushed every things including houses at the downstream of the dam away toward Pesanggrahan River. At the junction with that river, the flood still have enough strength to float and move some cars onto the fence of housing near the bank of the river.

The flood was flowing out from the reservoir until the sun rise time. The inundation reached 3 m depth. There was a secondary landslide occurred which destroyed the house on top of the bank. Some photographs of the disaster are presented in the following pages.

Photo 1 Aerial photograph of Situ Gintung Dam and the downstream valley before the collapse.
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3. Hydrodynamic Simulation and Flood Velocity Profile

The simulation was done by simplifying the condition at the breaking event, since the 2D breaking process involving interaction of land slide, erosion, and water flows is very complicated process. Therefore, the upper boundary condition is taken to be the maximum discharge and the lower boundary condition is taken to be the surface water level at the occurrence of the maximum discharge of the result of 1D simulation. The finite element mesh was constructed from the valley cross-section data surveyed after the disaster. The bed level data between two cross-sections are linearly interpolated. At the current study, the domain of...
computation stretches until about 350 m downstream of the toe of the dam. Therefore, at this stage, the study focuses on the horizontal variation of the depth-averaged velocity on that reach.

The simulation of the flood including the opening of the collapsed dam as sketched in Fig. 1

![Fig. 1 Collapsing dam and the flood propagating downstream](image)

The simulation is aimed to study the cross-sectional flow profile of the flood. In order to reach this aim, a steady 2D flow simulation was conducted. However, the simulation runs the numerical model in dynamic mode (transient) to spin down the low velocity initial condition into the flow state that simulate the peak discharge condition.

The study used the FESWMS Fl02DH Model within the Surfave Water Modeling System (SMS) version 8. The SMS software is basically a pre- and post-processing unit for numerical modeling analysis. The FESWMS Fl02DH Model of the Federal Highway Administration, USA is one of the numerical engines available within the SMS software. The FESWMS (Finite Element Surface Water Modeling System) Fl02DH Model descretizes the 2D depth averaged water flow equations using the Finite Element Methods in the form of convection and diffusion equations of mass and momentum conservations. By implementing the above approach the model is capable implementing the above approach the model is capable of simulating subcritical and supercritical flow conditions. These two flow conditions may occur in the flow state of the flash flood.

The governing equations used by the Fl02DH are as follows (Froehlich, 2002):

\[
\begin{align*}
\frac{\partial c_{v,1}}{\partial t} + \frac{\partial q_{x,1}}{\partial x} + \frac{\partial q_{y,1}}{\partial y} - q_{w,1} &= 0 \\
\frac{\partial q_{x,1}}{\partial t} + \frac{\partial}{\partial x} \left( \beta \frac{q_{x,1}^2}{H} + \frac{1}{2} g H^2 \right) + \frac{\partial}{\partial y} \left( \beta \frac{q_{y,1}^2}{H} + \frac{1}{2} g H^2 \right) + g H \frac{\partial v_{x,1}}{\partial x} + \frac{H}{Q} \frac{\partial P_{a}}{\partial x} - \Omega q_{x,1} + \\
\frac{1}{\rho} \left[ \tau_{x,1} - \tau_{v,1} - \frac{\partial (H \tau_{c,1})}{\partial x} - \frac{\partial (H \tau_{v,1})}{\partial y} \right] &= 0 \\
\frac{\partial q_{y,1}}{\partial t} + \frac{\partial}{\partial x} \left( \beta \frac{q_{x,1} q_{y,1}}{H} + \frac{1}{2} g H^2 \right) + \frac{\partial}{\partial y} \left( \beta \frac{q_{y,1}^2}{H} + \frac{1}{2} g H^2 \right) + g H \frac{\partial v_{y,1}}{\partial y} + \frac{H}{Q} \frac{\partial P_{a}}{\partial y} - \Omega q_{y,1} + \\
\frac{1}{\rho} \left[ \tau_{y,1} - \tau_{v,1} - \frac{\partial (H \tau_{c,1})}{\partial x} - \frac{\partial (H \tau_{v,1})}{\partial y} \right] &= 0
\end{align*}
\]

where \( \beta = \) isotropic momentum flux correction coefficient that accounts for the variation of velocity in the vertical direction, \( g = \) gravitational acceleration, \( \rho = \) water mass density, \( P_{a} = \) atmospheric pressure at the water surface, \( \Omega = \) Coriolis parameter, \( \tau_{x,1} \) and \( \tau_{y,1} = \) bed shear stresses acting in the \( x \) and \( y \) directions, respectively, \( \tau_{c,1} \) and \( \tau_{v,1} = \) surface shear stresses acting in the \( x \) and \( y \) directions, respectively, and \( \tau_{c,1}, \tau_{c,2}, \tau_{c,3}, \) and \( \tau_{c,4} = \) shear stresses caused by turbulence where, for example, \( \tau_{v,1} = \) the shear stress acting in the \( x \) direction on a plane that is perpendicular to the \( y \) direction.

The finite element formulation applied in the Fl02D Model is the Galerkin Method. The method optimizes the coefficients of trial functions by requiring orthogonality of the weighted functions and the residual functions. This is done by selecting the basis functions of the trial functions (interpolation functions) as the weighting functions and setting the integral of the product of weighting functions and the residual functions to zero. The integration is done element by element, and the global matrix is constructed by assembling all element matrices. Fl02D Model uses quadratic elements of both triangular and quadrilateral.

The governing equations of the hydrodynamics can be written shortly as follows.

\[
L \mathbf{u} - \mathbf{f} = 0 \quad (4)
\]

where, \( L \) is the differential operator, \( \mathbf{u} \) is the dependent variables, and \( \mathbf{f} \) is the known function. The dependent variables are the unknown and approximated by the trial functions \( v \). Since \( v \) is the approximating function, therefore, applying the differential operator \( L \) on \( v \) will give a residual \( \mathbf{e} \).

\[
L v - \mathbf{f} = \mathbf{e} \quad (5)
\]

The formulation of the Galerkin weighted residual optimization in each element is then,

\[
\int N_i (x, y) \mathbf{e} (x, y) \, d\Omega^e = 0; \quad j = 1, 2, \ldots, n \quad (6)
\]

Where, \( n \) is the number of nodes in the element. The element trial function is in the form of interpolation functions. The generic formulation of the interpolation function is,

\[
v = \sum_{i=1}^{n} N_i (x, y) \mathbf{v}_i \quad (7)
\]

where, \( N_i (x, y) \) is the basis function of the interpolation function. The Lagrange function is used as the basis function. The element integral is evaluated in the element natural coordinate system \((\xi, \eta)\). The result is then transformed to the \((x, y)\) coordinate system.

The data used for the finite element mesh construction is from the result of topographic survey on the valley after the disaster by the Ministry of Public Works in form of cross-sectional data. Therefore, the values of bed
elevation at nodes located between cross-sections are interpolated linearly from those at the cross-sections. The cross-sections of Situ Gintung reservoir and its downstream valley are shown in Figure 2 (Rahardjo, 2009). The computation domain is limited from upstream of the dam to about 350 m downstream of the dam. The contour and discretization of the computation domain is shown at Figure 3 and 4.

Fig. 2 Cross-sections of Situ Gintung reservoir and its downstream valley.

Fig. 3 Simplified contour of the downstream valley

Fig. 4 The finite element mesh

4. Result and Discussion

The simulation was run using inflow discharge upstream boundary condition of 349.1 m2/s, and water level downstream boundary condition of +85 m (about 3 m of flow depth). This values are the peak of dam break triggered flood hydrograph.

Roughness coefficients are approximated according to Chow, 1992. From the photographs of the field survey and the aerial photograph before the disaster, the land coverage of the downstream valley is grouped into several zones. The approximate values of the Manning roughness coefficient are set as follows. For the housing zone, tree zone, open area zone (fish pond, rice field, or other), main channel zone, and trench zone, the value are of 0.15, 0.11, 0.06, 0.04, and 0.035 respectively.

The wall boundaries are approximated as slip boundary and semi-slip boundary. However, the simulation result are very similar.

Successful runs were achieved after implementing spinning down technique. The initial condition started with a 10 m higher downstream water level, and by using transient simulation and downstream boundary condition setting, the downstream boundary condition was pulled down slowly until reaching +85 m that is 3 m above the bed level. It needs 20 hour simulation time with computation time step of 0.1 hour to spin down the initial condition to get the steady state solution.

Experiment was carried out to study the sensitivity of roughness coefficient values to the cross-sectional flow profile. The result of the study shows that the change in roughness coefficient values of the bed is more sensitive to the change of the cross-sectional velocity profile than that of the slip or semi-slip wall boundary conditions.

The highest velocity in the downstream valley is of about 2.0 m/s. This highest velocity occurs at the middle of the cross-section. At the opening of the collapsed dam, the maximum velocity reaches about 8.4 m/s. At just down stream of the opening, the velocity magnitude of the flash flood peak is about 3 to 4 m/s. The following figures are the result of the simulations.

The diffusion of flow momentum determine how strong current drags it surrounding water, supposed it has lower velocity magnitude. The strength of such process depends on the rate of turbulence at place. The rate of turbulence depends on the bed roughness and the velocity transversal gradient. In Flo2D Model, a diffusion coefficient is provided to adjust the process. During the study, the default value of 5 m2/s is used since its provide stability of the simulation run. At the opening there is area that the bed elevation decrease rapidly and the velocity magnitude changes very significantly. Instability of the simulation always starts from this area, therefore, the diffusion coefficient is increased to 10 m2/s. Decreasing the value of this diffusion coefficient in the area just downstream of the opening gives significant velocity pattern change and velocity magnitude increase in the area. The value of 5 m2/s gives a rapid decrease of velocity magnitude from 8 m2/s to 5 m2/s just in about 30 to 50 m. When the diffusion coefficient is reduced to 1 m2/s, the decrease of velocity magnitude becomes more slower and higher velocity magnitude extends longer. Figure 7 shows that the velocity magnitude of 3 to 4 m/s extends to nearly the downstream end of the computation domain or about 300 m from the opening of the collapsed dam.
5. Conclusion

The effort to simulate dam failure triggered flash flood has been successful. The current achievement is simulating the peak discharge state in a quasi-steady approach. The simulation result shows that velocity magnitude of about 8 m/s occurs around the opening part of the broken dam. In the interval of about 50 m downstream of the opening, the velocity magnitude is decreasing rapidly toward downstream to the value of 2 m/s. The decrease seems to relate with the widening of the width of the main flow discharge. This widening of the main flow depends on the width of the valley and the momentum diffusion process of the flow, especially in transversal direction. In the simulation, the last factor is controlled by setting the diffusion coefficient. Decreasing this coefficient from value of 5 m²/s to 1 m²/s in the downstream of the opening gives significant increase of the velocity magnitude along the middle of the valley.

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Fig. 5 Velocity vectors of the simulation result with the diffusion coefficient of 5 m²/s in the valley. Only corner node vectors are plotted.

Fig. 6 Zone of velocity interval 0-1 m/s, 1-2 m/s, 2-3 m/s, 3-4 m/s, and higher with the diffusion coefficient of 5 m²/s in the valley.
Fig. 7 Velocity magnitude contour using lower diffusion coefficient value, that is of 1 m²/s in the middle of the valley downstream of the opening. Zone of high velocity of 3-4 m/s extends to 300 m from the dam opening.

Fig. 8 Velocity magnitude and direction pattern using lower diffusion coefficient value, that is of 1 m²/s in the middle of the valley downstream of the opening.