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A Simple Mathematical Model for Flood Control by a Dry Dam

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Abstract. At the same time that the estimated risk of natural disasters is increasing due to global warming, the capacity to prevent disasters in Japan has been weakened due to infrastructure degradation and population aging. To reduce the damage caused by flooding, flood control dams without a slide gate in a spillway, known as "dry dams," have been planned and built on some sites in Japan. In this study, a simple mathematical model for flood control by a dry dam is proposed, aiming to encourage the construction of such dams. An implicit integral equation that is based on the continuity equation and simulates flood control by a dry dam is analytically derived. This equation can be easily solved by a spreadsheet program, making the mathematical models widely accessible, particularly to hydraulic engineers and students. The model's results are compared with laboratory experimental results and the output of a numerical simulation. The results show that the accuracy of the model is quite good, especially in its prediction of the maximum water level of dam reservoirs, meaning that the peak discharge during a flood can be predicted to high accuracy by using this simple mathematical model.

1. Introduction

In recent years, an increase in the frequency and intensity of disaster hazards, including heavy rainfall, drought, and typhoons, has been observed and is likely due to global warming. Various effects of global warming are expected to become apparent in the future. For example, the occurrence of catastrophic disasters due to large-scale flooding is mentioned as a concern by the Science Council of Japan [1]. During earlier periods of strong economic growth, social and disaster-prevention infrastructure in Japan was constructed in many places, but most of it is now aging. Although the Great East Japan Earthquake of 2011 caused disaster resilience and readiness to be seen as pressing issues in Japan, global economic conditions make it unlikely that new construction and major overhauls of large-scale disaster prevention facilities will continue over the long term. Therefore, intelligent disaster prevention measures, such as the effective use of existing facilities, will be increasingly needed in the future.

Given these circumstances, dry dams are arousing renewed interest in Japan [2]. Representative examples include the Masudagawa Dam in Shimane Prefecture and Nishinotani Dam in Kagoshima Prefecture [3, 4, 5, 6]. Although some relatively small dry dams were constructed before the 1970s to protect farmland from flooding, few major dry dams have been built exclusively to protect lives and property against flooding. Issues concerning these dry dams have already been noted, and it is hoped that additional studies will help clarify these issues [7, 8, 9]. Austria, where many small-scale dry dams have been constructed in catchment basins, provides many examples for study [10, 11]. To contribute to solving issues with dry dams and spread their adoption, this study analytically derives a simple mathematical model for flood control with a dry dam. The accuracy of this model was checked by comparison with results from laboratory experiments and numerical simulation.

2. Outline of the mathematical model

2.1. Basic equation

In this research, the following equation (1) is used as a basic equation for flood control by a dry dam, using only a continuity equation for simplicity:

$$\frac{dV(t)}{dt} = Q_{in}(t) - Q_{out}(t) \qquad (1)$$

where t is time, $V(t)$ is the value of water stored in a dry dam reservoir at time t, $Q_{in}(t)$ is inflow discharge to the dam, and $Q_{out}(t)$ is outflow discharge from the dam.

In this model, $Q_{out}(t)$ is expressed by the following equation:

$$Q_{out}(t) = C_a\, a_o \sqrt{2gh(t)} \qquad (2)$$

where $h(t)$ is the reservoir water level of the dry dam, C_a is a coefficient of discharge, a_o is the cross-sectional area of the bottom of a regular spillway, and g is the constant of gravitational acceleration. It is assumed that a_o is very small relative to the flow area just in front of the dam levee and that the outflow velocity from the regular spillway is determined by the water level at that time, based on Torricelli's theorem [12]. If the peak inflow discharge at $Q_{in}(t)$ is less than the designed high-water discharge and the emergency spillway connected to the dry dam is never used, then the dry dam uses only a bottom hole as a regular spillway for flood control.

To simplify notation, a coefficient α is defined by

$$\alpha = C_a\, a_o \sqrt{2g} \qquad (3)$$

which allows rewriting equation (2) as equation (4).

$$Q_{out}(t) = \alpha\, h^{1/2} \qquad (4)$$

2.2. Dry dam modeling

In this study, the cross-sectional shape of a dry dam levee is simply approximated by the following equation:

$$y = C_d \left| x \right|^m \qquad (5)$$

where m (a non-zero real number) and C_d (a positive real number) are dimensionless constants related to the cross-sectional shape of the levee, x is the spatial axis extending in the lateral direction from the center of the levee, and y is the vertical axis from the bottom of the center of the levee. As seen in Eq. (5), the dam shape is assumed to be symmetric in the x direction. Notably, Eq. (5) and C_d are unnecessary for a dam with a rectangular cross section. In this study, the cross-sectional shape of the riverbed is also the same as reflected in Eq. (5) since the river is assumed to be a straight channel.

The slope of the riverbed and reservoir bed is a constant, tan θ, where θ ($0 < \theta < \pi/2$) is the angle between the straight riverbed and the horizontal plane. In this study, cases where there is no slope (i.e., θ = 0) are also considered; in such cases the longitudinal length L of the dam lake is needed when calculating the water volume stored in the dam lake. However, when the riverbed has a positive slope, L is unnecessary because the water storage volume is naturally determined by the water surface level (h) leveling upstream freely.

We have two options (hereinafter, Type-1 and Type-0) for applying this analytical model to cases where multiple dry dams are arranged in series. In Type-1 cases, the distance between the dams is relatively long, and the positive averaged uniform riverbed slope is used. In Type-0 cases, where the distance between the dams is short or the riverbed slope is quite mild, the stored water volume (V) can be restricted by an upstream dam levee. In this condition, the riverbed slope in this model should be approximately 0 and the longitudinal length of the dam lake L must be given so as to make the height of a dam levee (H_d, described below) accord with the maximum water level in the dam lake in order to match the maximum stored water volume with the storage capacity planed in advance.

As an example, **Figure 1** shows a schematic of a dam model in which the dam shape is quadratic (m = 2) and there is a riverbed slope (Type-1). Despite this, the model can be used when the cross section of the dam levee is rectangular, corresponding to infinite m. As another example, **Figure 2** shows a schematic of a rectangular dam model without riverbed slope (Type-0). The symbol H_d used in **Figures 1** and **2** indicates the height of the dam, B_d is the length of top of dam, and $b(h)$ is the water surface width just before the dam levee at water depth h. From the geometry, the relation of C_d to the dam levee shape can be expressed as equation (6).

$$C_d = \left(\frac{B_d}{2}\right)^{-m} H_d \qquad (6)$$

In the case of a rectangular cross section, Eq. (6) is unnecessary because C_d is not used; only H_d and B_d are necessary to describe the cross-sectional shape of the dam levee (see **Figure 2**). As is clear from the above description, there is no water surface gradient in this model, and sediment storage capacity and other concerns are ignored for the sake of simplicity.

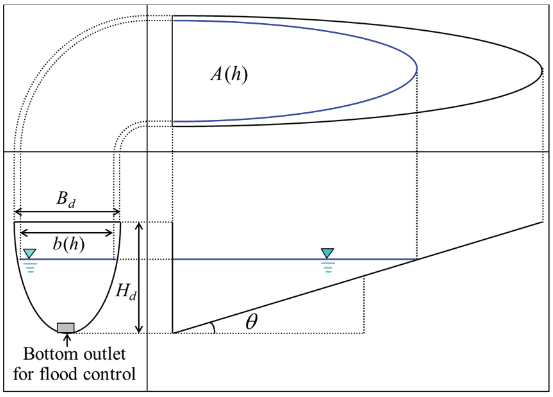

Figure 1. Example of a Type-1 dry dam (m=2, n=3/2).

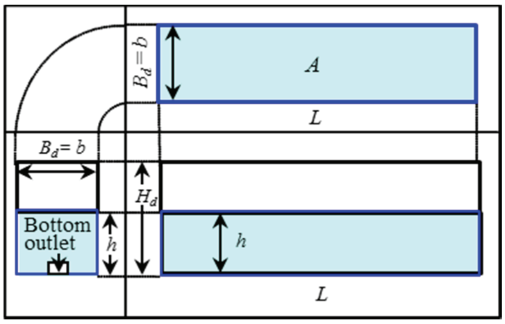

Figure 2. Example of a Type-0 dry dam (m is infinite, n=0).

2.3. Derivation of the analytical solution

The derivative of the stored water volume $V(t)$ is expressed by the following equation involving the water surface area $A(h)$ of the dam lake.

$$\frac{dV(t)}{dt} = A(h)\frac{dh(t)}{dt} \qquad (7)$$

From this, Eq. (1) can be expressed as the following equation by applying Eqs. (4) and (7).

$$A(h)\frac{dh(t)}{dt} + \alpha\, h^{1/2}(t) = Q_{in}(t) \qquad (8)$$

According to the conditions set for this model, the stored water volume $V(t)$ can be described as a function of the water depth $h(t)$. Also, because the water surface area $A(h)$ can be expressed as a power function of the water depth due to the assumption of Eq. (5), $A(h)$ can be expressed as

$$A(h) = \beta\ h^n \quad (9)$$

where β is a positive real constant, and n is a real constant depending on m as described later. Substituting Eq. (9) into Eq. (8) gives equation (10).

$$\beta\ h^n \frac{d\,h(t)}{dt} + \alpha\ h^{1/2}(t) = Q_{in}(t) \quad (10)$$

The above equation is an inhomogeneous ordinary differential equation. Therefore, when $Q_{in}(t) = 0$, the following homogeneous ordinary differential equation is solved.

$$\beta\ h^n \frac{d\,h}{dt} + \alpha\ h^{1/2} = 0 \quad (11)$$

If we let C_c be an integration constant, we have Eq. (12) as the analytical solution for Eq. (11).

$$\frac{1}{n+1/2} h^{n+1/2} = -\frac{\alpha}{\beta} t + C_c \quad (12)$$

Next, replacing C_c with the function of time $C_c(t)$ and solving for the inhomogeneous basic equation (10), $C_c(t)$ is expressed in integral form as Eq. (13).

$$C_c(t) = \frac{1}{\beta} \int Q_{in}\ h^{-1/2}\ dt \quad (13)$$

Therefore, the following equation, Eq. (14), is obtained as a solution.

$$\frac{1}{n+1/2} h^{n+1/2} = -\frac{\alpha}{\beta} t + \frac{1}{\beta} \int Q_{in}\ h^{-1/2}\ dt \quad (14)$$

Note that the superscript $^{n+1/2}$ in the left side denotes an exponent, not a time step (as is often done in descriptions for a numerical simulation). That is, the solution can be written as Eq. (15).

$$h(t) = \left[\frac{n+1/2}{\beta} \left(-\alpha\, t + \int Q_{in}\ h^{-1/2}\ dt \right) \right]^{\frac{1}{n+1/2}} \quad (15)$$

Although the above Eqs. (14) and (15) are implicit integral equations, the analytical solution $h(t)$ can be obtained easily using spreadsheet software. The integral of the second term on the right side in Eq. (14) can be calculated as the sum of discrete quantities from the initial value because $Q_{in}(t)$ is treated as a discrete quantity or a relatively complicated function in actual calculation.
The stored water volume $V(t)$ can be easily obtained by integrating $A(h)$ using Eq. (9), and the following equation, Eq. (16), is obtained.

$$V(t) = \frac{\beta}{n+1} h^{n+1} \quad (16)$$

Here β, A, and V differ depending on the shape of the dam levee and the presence or absence of a riverbed slope, so we consider the two types separately in the next section: Type-0 ignores the riverbed slope, and Type-1 has a positive riverbed slope θ.

2.3.1 Cases with no riverbed slope (*Type-0 dams*). In this type, the exponent n in Eq. (9) can be expressed in terms of the exponent m relating to the dam shape in Eq. (5), as in Eq. (17).

$$n = \frac{1}{m} \quad (17)$$

Also, β is expressed as Eq. (18).

$$\beta = \frac{B_d\ L}{H_d^{\ n}} \quad (18)$$

$A(h)$ from Eq. (9) can be expressed as Eq. (19).

$$A(h) = \frac{B_d\ L}{H_d^{\ n}}\ h^n \quad (19)$$

From these equations, the stored water volume of Eq. (16) can be expressed as Eq. (20).

$$V(t) = \frac{B_d\ L}{(n+1)\ H_d^{\ n}}\ h^{n+1} \quad (20)$$

As described above, n and β depend on the dam shape. Some examples of shapes for Type-0 dams are summarized in **Table 1**. As is apparent from Eq. (14), no solution is found for $n = -1/2$. This is because the solution derived from Eq. (10) or Eq. (11) is exponential. When $n = -1/2$, the left-hand side of Eq. (14) is replaced by $\log_e h$, and the solution is given by Eq. (21).

$$\log_e h = -\frac{\alpha}{\beta} t + \frac{1}{\beta} \int Q_{in}\ h^{-1/2}\ dt \quad (21)$$

Here, e is the base of the natural logarithm. Note that, as in the case of $m = -2$ (corresponding to $n = -1/2$ in this case), cases in which $m < 0$ are not ordinary shapes for a dam levee because the length of the dam top becomes shorter than that of the dam bottom.

Table 1. Example parameter values for cross-sectional shapes with zero riverbed slope (Type-0).

	Dam shape	β	$A(h)$	V
n=0, m=∞	Rectangle	$B_d\ L$	$B_d\ L$	$B_d\ L\ h$
n=1, m=1	Triangle (Linear)	$\frac{B_d\ L}{H_d}$	$\frac{B_d\ L}{H_d} h$	$\frac{B_d\ L}{2\ H_d} h^2$
n=1/2, m=2	Quadratic curve	$\frac{B_d\ L}{\sqrt{H_d}}$	$\frac{B_d\ L}{\sqrt{H_d}} h^{\frac{1}{2}}$	$\frac{2\ B_d\ L}{3\ \sqrt{H_d}} h^{\frac{3}{2}}$
n=1/3, m=3	Cubic curve	$\frac{B_d\ L}{H_d^{1/3}}$	$\frac{B_d\ L}{H_d^{1/3}} h^{\frac{1}{3}}$	$\frac{3\ B_d\ L}{4\ H_d^{1/3}} h^{\frac{4}{3}}$

2.3.2 Cases with uniform positive riverbed slope (*Type-1 dams*). When there is a uniform positive riverbed slope, the power n in Eq. (9) is expressed in terms of m as shown in Eq. (22).

$$n = \frac{1}{m} + 1 \quad (22)$$

β is expressed as Eq. (23).

$$\beta = \frac{B_d}{n \tan\theta\ H_d^{\ n-1}} \quad (23)$$

Also, $A(h)$ from Eq. (9) can be expressed as Eq. (24).

$$A(h) = \frac{B_d}{n \tan\theta \, H_d^{\,n-1}} h^n \qquad (24)$$

From these, the stored water volume described in Eq. (16) can be expressed as follows:

$$V(t) = \frac{B_d}{n(n+1) \tan\theta \, H_d^{\,n-1}} h^{n+1} \qquad (25)$$

Table 2 shows the parameters for some Type-1 examples with uniform riverbed slope and different cross-sectional shapes.

Table 2. Example parameter values for cross-sectional shapes with a uniform riverbed slope (Type-1).

	Dam shape	β	$A(h)$	V
$n=1,\ m=\infty$	Rectangle	$\frac{B_d}{\tan\theta}$	$\frac{B_d}{\tan\theta} h$	$\frac{B_d}{2\tan\theta} h^2$
$n=2,\ m=1$	Triangle (Linear)	$\frac{B_d}{2\tan\theta\, H_d}$	$\frac{B_d}{2\tan\theta\, H_d} h^2$	$\frac{B_d}{6\tan\theta\, H_d} h^3$
$n=3/2,\ m=2$	Quadratic curve	$\frac{2\,B_d}{3\tan\theta\sqrt{H_d}}$	$\frac{2\,B_d}{3\tan\theta\sqrt{H_d}} h^{\frac{3}{2}}$	$\frac{4\,B_d}{15\tan\theta\sqrt{H_d}} h^{\frac{5}{2}}$
$n=5/4,\ m=4$	Quartic curve	$\frac{4\,B_d}{5\tan\theta\, H_d^{1/4}}$	$\frac{4\,B_d}{5\tan\theta\, H_d^{1/4}} h^{\frac{5}{4}}$	$\frac{16\,B_d}{45\tan\theta\, H_d^{1/4}} h^{\frac{9}{4}}$

3. Verification of the Analytical Solutions of the Simple Mathematical Model

The accuracy of the analytical solution derived in the previous chapter is examined by comparing the analytical solution of this model with results from laboratory experiments and numerical simulations.

3.1. Zero riverbed slope in a rectangular dam

For this case, the accuracy of the analytical model was confirmed by performing a simple laboratory experiment with zero riverbed slope in a rectangular dam (see **Figure 2**).

3.1.1. Outline of the experiment (*Case A*). In the experiment, a rectangular water tank with a height of 50.0 cm, a width of 25.0 cm (corresponding to B_d) and another directional horizontal length of 58.1 cm was used for the experiment, Case A. The water tank is divided into two areas by a vertical partition plate (equivalent to an overflow weir) with a height of 36.4 cm and a thickness of 1.1 cm. The area with horizontal longitudinal length of 29.0 cm (corresponding to L) forms a dry dam lake, with an outlet installed in the bottom as a regular spillway for flood control. The other area, whose horizontal longitudinal length is 28.0 {=58.1−(29.0+1.1)} cm, forms a drainage area for water overtopping the weir in order to keep the water surface level (WSL) constant. The shape of the bottom outlet is a circle of diameter 1.16 cm, and the cross-sectional area a_o = 1.06 cm^2.

First, in order to determine the discharge coefficient (C_a) of this experimental tank with the outlet, an experiment with a constant water level was conducted as a preliminary experiment. Enough water was supplied to the part of the dry dam to keep the water depth constant at 36.4 cm, overtopping the weir, and the steady outflow discharge from the outlet was directly measured. The discharge coefficient C_a obtained in the full water condition was 0.70. In this study, this value (C_a = 0.70) was used for convenience in the analytical solution.

As a simple case, Q_{in} = 0 was considered. A free drainage experiment from the outlet was performed by setting the initial water level h_0 at t = 0 to 34.0 cm, and the times until the water level was 32.0 cm, 28.0 cm, 24.0 cm, 20.0 cm, and 16.0 cm were measured as drainage times. Three trials were performed; these are Case A1, Case A2, and Case A3.

3.1.2. Comparison of the experimental results with the analytical solution (*Case A*). When $Q_{in}(t)$ equals 0 (as in this experiment), the equation of the homogeneous form given in Eq. (11) is solved with the initial condition h_0, and the following analytical solution is obtained as Eq. (26).

$$h(t)=\left(\sqrt{h_0}-\frac{\alpha}{2\beta}t\right)^2=\left(\sqrt{h_0}-\frac{C_a a_o\sqrt{2g}}{2B_d\ L}t\right)^2 \qquad (26)$$

Notably, Eq. (26) is similar to Eq. (15) when n=0. Because the shape of this dam lake is a rectangular prism, the stored water volume V is B_dLh, which can be confirmed by substituting n=0 into Eq. (20).
A comparison of the above analytical solution with the experimental results is shown in **Figure 3**. The proposed model has sufficient accuracy under the given conditions because the time series of the WSL agrees well with the experimental results.

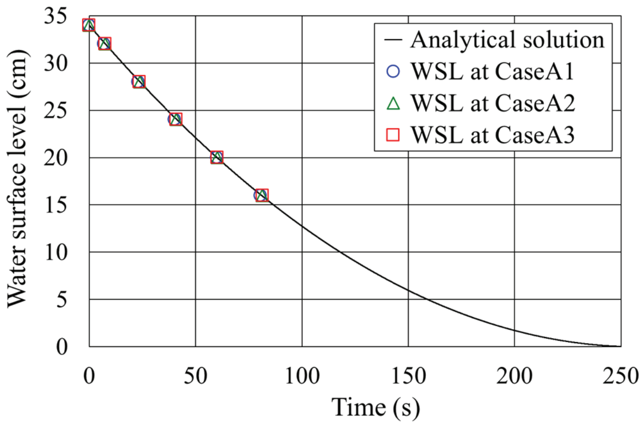

Figure 3. Comparison of experimental results with the analytical solution of Case A (Type-0, n=0).

3.2. Non-zero riverbed slope with a dam having a quadratic cross-sectional shape (*Type-1*)
Performing an experiment with this condition was not straightforward, so the accuracy of the proposed mathematical model was confirmed by comparison with known numerical simulation results [13].

3.2.1 Outline of the numerical simulation (*Case B*). In the numerical simulation, three dry dams with quadratic cross-sectional shapes were used. For each, the levee height was H_d = 100.0 m, and the length of dam top was B_d = 222.2 m. The three were arranged in series [13]. The flood control effect of dry dams was investigated for a flood with peak inflow discharge of 8887 m^3/s, a normal inflow discharge of 219 m^3/s, and peak arrival time of 20 h ("conventional type Case 1-1" in reference [13]). Since tan θ = 0.01, this is a Type-1 dam with m = 2 and n = 3/2, corresponding to **Figure 1**. For the sake of simplicity, the time series of the water surface level and outflow discharge of the first upstream dam were compared with the analytical result as Case B. In both analyses, C_a = 1.0 and a_o = 152.9 m^2.

3.2.2 Comparison between numerical simulation and analytical results (*Case B*). **Figure 4** shows a comparison between the previous numerical simulation results [13] and the analytical results given by the proposed mathematical model using Microsoft Excel 2010 spreadsheet software. The two calculation results are in perfect agreement. This means that the models are ultimately performing the same calculation. Note that the numerical simulation was also based on only the continuity equation, and the basic equation and the computational condition were basically the same as in this analytical solution. Therefore, this perfect agreement is a reasonable result; we expected the solutions would coincide to within truncation error.

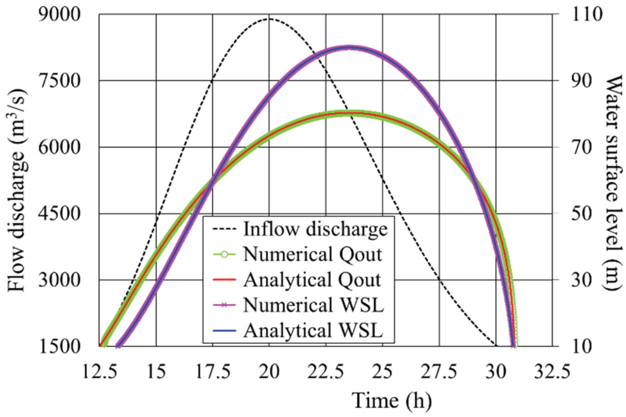

Figure 4. Comparison of numerical simulation with analytical solution in Case B (Type-1, *n*=3/2).

3.3. A riverbed with non-zero slope in a rectangular dam (*Case C*)
In this section, the analytical solution is compared with results from a past laboratory experiment [14] as Case C to examine the accuracy of the proposed model for the case of a rectangular dam with non-zero riverbed slope.

3.3.1. Outline of the experiment (*Case C*). A straight open channel with a length of 1400.0 cm, a width of 60.0 cm, a depth of 60.0 cm, and a channel bottom slope of tan $\theta = 0.04$ was used for the experiment [14] (see **Figure 5**). Three dry dams, which were identical except for the cross-sectional area a_o of the regular spillway, were arranged in series.

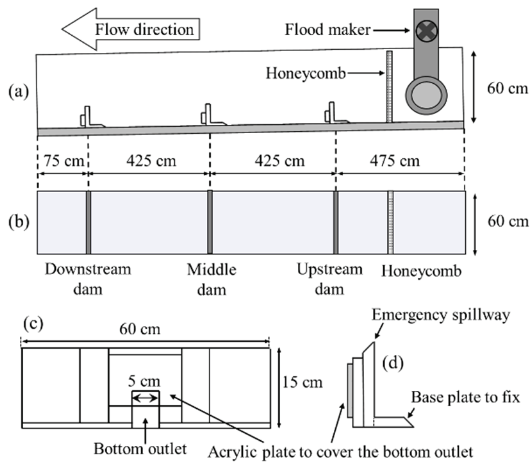

Figure 5. Schematic of the experimental system for Case C [14]: (a) side view of the channel, (b) plane view of the channel, (c) front view of a dam, (d) side view of a dam.

Each dam was a vertical wall with a rectangular cross section with dam levee height $H_d = 15.0$ cm. The cross section of a rectangular bottom outlet corresponding to a regular spillway could be precisely adjusted by changing the height of the rectangular outlet and keeping the width constant at 5.0 cm. Although a crest-free overflow-type gateless dam was adopted for each emergency spillway, the experimental condition in which overflow did not occur is compared with the analytical solution proposed in this study.
The three dams were located at 475.0 cm, 900.0 cm, and 1325.0 cm from the upstream end of the water channel. There was enough space between dams that the reservoir water level in the high water condition was not affected by the upstream dam levee because the bottom slope was sufficiently steep. Therefore,

the dam lake length L can be ignored here. The storage capacity of each dam was 168,750 cm^3 excluding the volume of the dam levee, its base, and other parts (see **Figure 5**).

In this experiment, time-series data of each reservoir level were acquired by taking images from the side of each dam. Flow discharges in the steady states were separately measured for each reservoir level condition, and a stage-discharge curve (the so-called H–Q curve) was created for each dam from the reservoir level and measured flow discharge. After the experiment, the time series data of water level were converted to flow discharge rates.

In this study, for the sake of simplicity, the accuracy of the analytical model was examined by comparing the time series of the reservoir level of the first dam in the experiment (Case A1 in reference [14]) with the analytical result. The cross-sectional areas a_o of the dams in the referenced experiment were 30.375 cm^2, 23.35 cm^2, and 18.725 cm^2, in order from the upstream side.

The discharge coefficient C_a in the model experiment was determined from the H–Q curve as follows. Oshikawa et al. [14] noted when the water levels were below the levee height then they were proportional to the square root of the water depth, as in Torricelli's theorem [12]. Therefore, $C_a = 0.82$ was obtained as the discharge coefficient of the experiment by finding the coefficient of the curve proportional to the square root of the water depth (the least squares method was used to find this coefficient).

3.3.2 Comparison of the experimental result and the analytical solution (*Case C*). As mentioned above, the analysis targeted the dam farthest upstream. To compare the proposed method's results with the experimental results, the analytical conditions were set to $n = 1$, $H_d = 15.0$ cm, $B_d = 60.0$ cm, $a_o = 30.375$ cm^2, $C_a = 0.82$, $\tan\theta = 0.04$, and g=980.665 cm/s^2, and $Q_{in}(t)$ was the same as in the experiment with peak inflow discharge of 5982 cm^3/s.

Figure 6 shows the inflow discharge hydrograph generated by the flood maker at the upstream end; the WSL from the experiment; and the WSL and flow discharge as calculated by the proposed method. It can be seen that the peak value of the WSL given by this model matches the WSL from the experiment fairly well. However, the experimental WSL is time lagged relative to the analytical WSL. This is because the analytical model is based on the continuity equation only and neglects equations of motion (i.e., the motion of the flood, which produces deformation in the hydrograph). Also, it can be seen that the proposed model's accuracy for the time at which the reservoir level reaches a low after the peak is not good and has a different curvature. This is why discharge from the dam was determined by Torricelli's theorem [12] in the model. Therefore, in Case C, the peak of the reservoir level can be analytically reproduced with high accuracy, and relatively high prediction accuracy can also be expected for the peak flow discharge. Note, however, that the time lag in the analytical model is in the safe direction for disaster prevention because it forecasts an early peak time.

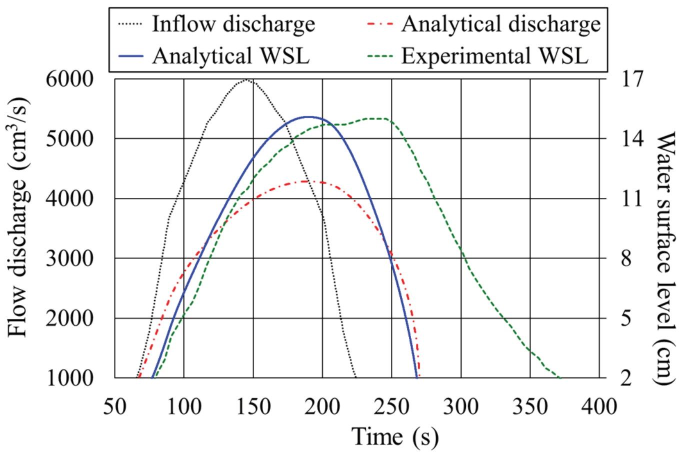

Figure 6. Comparison of experimental results with the analytical solution for Case C (Type-1, n=1).

4. Conclusions

Large-scale water disasters are of increasing concern due to global warming, and dry dams have been attracting attention as a powerful means of achieving both environmental and disaster prevention in recent years. However, additional research results are useful in promoting the adoption of dry dams in Japan, where few such dams are in use.

In this study, a simple mathematical model that can be used to easily predict a time series of reservoir water level and outflow discharge from a dry dam was proposed. The accuracy of the model was examined by comparison with results from laboratory experiments and numerical simulation. The model was found to have high prediction accuracy on peak outflow discharge, which is a particularly important quantity for flood control planning because the peak value of the reservoir water level can be accurately reproduced from it.

The proposed model will be very useful in appropriate cases. Although the reservoir water level is expressed by an implicit integral equation, a solution can be easily obtained by spreadsheet software, and the predictions are easy to interpret because equations of motion are not used, meaning that a sufficient theoretical understanding of hydraulics is not necessary for application. This benefits both civil engineers and students, among others. As can be seen from the verification in experiment Case A, this mathematical model can also be used for the analysis of outflow from an outlet.

Acknowledgments

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