Analysis of embankment failure mechanism in reservoirs due to rainfall infiltration during heavy rainfall

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**ABSTRACT**

Many heavy rain disasters have occurred in recent years in Japan. The maintenance of water storage facilities such as dams has become a global issue. The aim is to simulate the failure mechanism of reservoirs in which slip failure occurred on the downstream slope of the embankment due to the heavy rainfall in August 2019 in Japan. This study proposes a simulation method for evaluating embankment stability with respect to the fluctuations in the water level and the rainfall infiltration from the embankment surface using the two-dimensional saturated–unsaturated seepage flow simulation based on the finite element method. Moreover, the safety factors of the slip surface during heavy rain were calculated based on the simulated seepage line. By considering rainfall infiltration, the safety factor of the slip surface decreased significantly, and the occurrence of slip failure in the in-situ position could be reproduced. In conclusion, rainfall infiltration must be considered when evaluating the stability of a reservoir body during heavy rainfall.

**1. Introduction**

Many of the approximately 170,000 reservoirs in Japan were built before 1800 AD. In Japan, agricultural reservoirs have an embankment height of less than 15 m. Most of the old reservoirs were constructed using human power without any heavy machineries. Hence, compaction control and material quality control tests were not performed while designing the embankments. Therefore, the number of reservoir breaches caused by water leakage due to aging and the shrinkage of embankment cross-section owing to erosion is increasing. Maintaining reservoirs is becoming difficult because of the decreasing number of farmers and aging of agricultural workers. Heavy rain occurrences in recent years has caused multiple reservoir related disasters in Japan. Many reservoirs were breached during heavy rains in northern Kyushu in...
July 2017 and July 2018. Moreover, the outflow of stored water caused enormous damage to the downstream area. (Nishimura et al. 2020; Ozaki et al. 2019). The target of this study is the 2019 heavy rainfall disaster, which will be described later.

Damage to the embankment of a reservoir due to heavy rain can be divided into three types. (Penman 1986; Hori et al. 2002; Wrachien and Mambretti 2009; Jiang et al. 2020). ① ‘Seepage failure’ is caused due to the increase in pore water pressure inside the embankment and the formation and expansion of piping holes. ② ‘Slip failure’ results in shear failure on the downstream slope due to the increase in pore water pressure and saturation in the embankment as well as the decrease in the shear resistance of the embankment. ③ ‘Overflow failure’ occurs when stored water overflows from the top of the embankment resulting in embankment erosion. These failures can occur individually resulting in reservoir damage. However, these failure usually occur simultaneously during natural disasters leading to an embankment breach. For example, the cross section of an embankment decreases due to erosion caused by stored water overflow. This results in reservoir breach due to stored water pressure and seepage force. Moreover, a decrease in the cross section of the embankment body and failure on the downstream results in a breach.

Researchers have proposed various methods for predicting breach parameters in case of overflows, such as breach width, depth, breaching start time, and progress speed, based on past dam breach cases, field experiments, and laboratory model experiments. (MacDonald and Langridge-Monopolis 1984; Wahl 1998; Froehlich 2008; Javadi and Mahdi 2014; Dhiman and Patra 2019; Zhong et al. 2019). The purpose of determining the breach parameter is to predict the outflow amount and outflow range of the stored water to the downstream area.

Many field surveys have been conducted to determine the status of damage to reservoirs. However, there are few detailed studies simulating the disaster mechanism for comparison with the in-situ damage of the embankment of the reservoir (Koyama et al. 2020). The impermeability and drainage inside the embankment of many old reservoirs cannot be sufficiently controlled or compared with recently constructed fill dams or reservoirs (Eslamian et al. 2018). Many old reservoirs have different characteristics such as the degree of deterioration, repair history, type of embankment, and soil characteristics. It is believed a comprehensive simulation of the research on embankment damage mechanisms can contribute to the effective maintenance of reservoirs in the future. This study focuses on the mechanism of slip failure on the downstream slope resulting in embankment breach.

The current reservoir design guidelines (MAFF 2015) for assessing the stability of an embankment against slip failure during heavy rainfall are based on the rise of seepage line in the embankment because of the increase in the water level. However, the seepage of rainfall from the surface of the embankment has a significant impact on the formation of the seepage line in the embankment.

Studies have shown that rainfall infiltration destabilizes the slopes. From the FEM seepage flow simulation, it has been reported that the pore water pressure in the slope increases and the soil becomes saturated with rainfall infiltration. In addition, the slope stability during rainfall is evaluated by performing a seepage flow and slip stability simulation with the limit equilibrium method (Lee et al. 2009; Acharya et al.
The effects of rising and falling water levels on the stability of the embankment have been investigated. The mechanism by which the soil particles flow out due to infiltration and leading to the collapse of the embankment was investigated (Zhang and Chen 2006; Chen et al. 2012; Li et al. 2018; Zhong et al. 2019; Xu et al. 2020). However, few studies consider the water storage level changes and infiltration of rainfall simultaneously. Su et al. (2021) evaluated the slip stability of the embankment in the core dam considering rainfall and changing water levels. It is reported that even with a constant water level, the saturation of the downstream slope of the embankment progresses due to rainfall. Li et al. (2018) experimentally show that repeated rainfall wetting and drying causes cracks in the cohesive materials embankment. Park et al. (2019) demonstrated that rainfall infiltration is an important factor in simulating reservoir embankments stability concerning the frequent disasters due to heavy rainfall in recent years. There have been no cases in which the appropriate method was examined for seepage flow and stability simulation in light of the damaged dams and reservoirs.

The purpose of this study is to clarify the validity of seepage flow simulation that consider the water level changes and infiltration of rainfall as well as stability simulation. Moreover, the effect of rainfall infiltration on embankment stability was elucidated. The differences in safety factors depending on the presence or absence of rainfall infiltration were compared. The downstream slope stability of the reservoirs was simulated considering the fluctuation of the water level and the infiltration of rainfall. The damage mechanism of the reservoir was also clarified. The downstream slope slip failure of the embankment of two reservoirs in Takeo City, Saga Prefecture due to heavy rain in August 2019, was simulated. The seepage line in the embankment was determined using a 2-dimension FEM seepage flow simulation.

2. Summary of rainfall hysteresis and reservoir damages in Saga prefecture caused due to the heavy rainfall in August 2019

Highest rainfall was recorded in the northern part of Kyushu from August 26 to 29, 2019, because of the impact of linear rainfall zone. This resulted in the highest rainfall in recorded history in Saga and Nagasaki prefectures leading to flooding of rivers and sediment-related disasters. The Ministry of Agriculture, Forestry, and Fisheries reported heavy damages to agricultural land, agricultural facilities, and 44 agricultural reservoirs in the Fukuoka and Saga prefectures (MAFF 2019). Heavy rainfall was recorded from August 27 to 28, 2019 in the Saga prefecture.

Figure 1 illustrates the course of hourly and cumulative rainfall from August 27 to 29 from the Automated Meteorological Data Acquisition System (AMeDAS) Imari of the Japan Meteorological Agency. Continuous rainfall was observed from the early dawn of August 27 to the morning of August 28. Hori et al. (2002) reported that 20 mm hourly and 200 mm cumulative rainfall are the lower limits that cause damage to reservoirs. Therefore, this study defined a heavy rainfall event as 20 mm/h or more. Three heavy rainfall events occurred with hourly rainfall exceeding 20 mm. One of them was hourly rainfall of approximately 60 mm. The cumulative precipitation over two days was 398 mm.
In Saga Prefecture, 20 reservoirs were damaged, and in Takeo City, the downstream slopes of the four reservoirs were damaged. Figure 2 illustrates a location map of the four damaged reservoirs. The specifications of the four reservoirs are listed in Table 1. The reservoirs have homogeneous embankments composed of uniform soil materials, and the year of construction is unknown. Photos 1 to 4 illustrate the damage to each reservoir. Photo 4 shows that seepage failure presumably occurred due to multiple piping holes found on the downstream slope of the embankment of the Amari reservoir. Other reservoirs did not have visible piping holes. Moreover, slip failure occurred on the downstream slope in the three reservoirs as shown in Photos 1 to 3. The damage caused to Yoshinotani and Fukuida reservoirs was studied. Moreover, the stability of the downstream slopes of both reservoirs was evaluated.

Figure 1. Time data matrix for rainfall from August 27th to 28th, 2019.

Figure 2. Location of damaged reservoir in Takeo city.
based on the fluctuation of the water level in the reservoir due to heavy rainfall and the seepage of rainfall from the surface of the embankment.

3. Details of the simulated reservoir and rainfall data

Figure 3 illustrates the cross-sectional views of the Yoshinotani and Fukuida reservoirs. The full water level for the Yoshinotani reservoir is 6.36 m, the embankment
height is 7.56 m from the measured value, the upstream slope is 1: 1.3, and the downstream slope is 1: 1.4.

The full water level for the Fukuida reservoir is 6.86 m, the embankment height is 7.86 m from the measured value, the upstream slope is 1: 1.6, and the downstream slope is 1: 1.4. Both reservoirs exhibited a slip failure on the downstream slope. A slip failure occurred in the Fukuida reservoir from the vicinity of the shoulder to the upper part of the gabion of the buttoc. Gabion is below the slip surface and it is not accounted for in the simulation.

The physical and mechanical properties of the embankment soil and natural water content \( (w_n) \) are listed in Table 2. Figure 4 illustrates the particle size distribution, and Figure 5 illustrates the compaction curve for the soil samples. Moreover, undisturbed samples were collected from the embankment, and a consolidated undrained (CU) triaxial test was conducted. The compaction degree \( D_{cn} \) of the in-situ embankment obtained from the dry density \( \rho_{dn} \) of the undisturbed sample and the maximum dry density \( \rho_{dmax} \) of the compaction test are listed in Table 2. The embankments of both reservoirs exhibited a compaction degree of approximately 95\% as per the current design guideline of the reservoir (MAFF 2015).

**Table 1.** Outline of the specifications of reservoirs to be surveyed and the damage situation.

| No. | Reservoir | Type of embankment | Height (m) | Top width (m) | Crest length (m) | Water storage (m³) | Damage situation of embankment |
|-----|-----------|---------------------|------------|--------------|-----------------|-------------------|-------------------------------|
| 1   | Yoshinotani | Homogeneous | 7.56       | 2.0          | 75.0            | 12,000            | Slip failure of downstream slope |
| 2   | Fukuida   | Homogeneous   | 7.86       | 3.0          | 55.0            | 13,600            | Slip failure of downstream slope |
| 3   | Kozugawa  | Homogeneous   | 9.75       | 4.5          | 108             | 67,500            | Slip failure of downstream slope |
| 4   | Amari     | Homogeneous   | 6.20       | 1.6          | 20.3            | 4,200             | Seepage failure                 |

Figure 1 illustrates the hourly and cumulative rainfall change data over time from August 27 to 29, 2019, directly above the Yoshinotani and Fukuida reservoirs.
obtained from XRAIN radar. The XRAIN is a real-time rainfall observation system using high-performance weather radar data provided by the Ministry of Land, Infrastructure, and Transport. The XRAIN datasets were provided by the Data Integration and Analysis System (DIAS) developed and operated by the Ministry of Education, Culture, Sports, Science and Technology. The changes over time in the XRAIN rainfall data in the two reservoirs were similar with the maximum hourly rainfall recorded from 4 to 5 am on August 28. The maximum hourly rainfall was approximately 60 mm in the Yoshinotani reservoir and 90 mm in the Fukuida reservoir. The cumulative rainfall was 430 mm in the Yoshinotani reservoir and 477 mm in the Fukuida reservoir. The maximum hourly rainfall data recorded by AMeDAS Imari shown in Figure 1 is approximately similar to the data recorded using XRAIN for Yoshinotani reservoir; however, the recording date and time are different. The AMeDAS rainfall measurement method uses a tipping-down method different from that used by the XRAIN method. It is note that, the XRAIN data directly above AMeDAS Imari and AMeDAS Imari were consistent with each other.

![Figure 3. Cross section of the embankment of Yoshinotani and Fukuida reservoirs.](image)

| Reservoir  | $\rho_s$ (g/cm$^3$) | $F_c$ (%) | $I_p$ | $\rho_{d_{\text{max}}}$ (g/cm$^3$) | $\omega_n$ (%) | $\rho_d$ (g/cm$^3$) | $D_{\text{cn}}$ (%) |
|-----------|----------------------|-----------|-------|----------------------------------|---------------|---------------------|---------------------|
| Yoshinotani | 2.731 | 56.3 | 22.6 | 1.591 | 25.7 | 1.540 | 96.8 |
| Fukuida | 2.721 | 37.0 | 12.3 | 1.544 | 30.5 | 1.446 | 93.7 |
4. Examination of the slip failure mechanisms

The stability of the downstream embankment slope based on the fluctuating water level of the reservoir and the rainfall infiltration from the embankment surface was simulated. FEM seepage flow simulation was performed on the Yoshinotani and Fukuida reservoirs for calculating the seepage line in the embankment. Moreover, the stability of the downstream slope was simulated based on the calculated seepage line. The simulation period was from 00:00 on August 27 to 18:00 on August 28, 2019 (2520 min) on the XRAIN data (Figure 1). Yoshinotani reservoir was used in the description because the seepage flow and stability simulation method for both reservoirs is the same.

4.1. Estimating method of reservoir level fluctuation of reservoir

The fluctuating water levels in the Yoshinotani reservoir during heavy rains in August 2019 were determined using the XRAIN rainfall data. The hourly
Precipitation in the XRAIN rainfall data directly above the reservoir was used for calculating the rainfall intensity required to determine the water level in the reservoir. Multiple peak values of hourly rainfall observed during the target period were selected for calculation (rainfall data marked by a circle in Figure 6). The flood peak flow $Q_p$ was calculated using the equation given in Eq. (1) shown in the dam design standards in Japan (MAFF 2003) for specified hourly rainfall intensity.

$$Q_p = \frac{1}{3.6} \times r_e \times A$$

where $Q_p =$ flood peak flow (m$^3$/s); $r_e =$ mean effective rainfall intensity over the basin (mm/h); $A =$ basin area of reservoir (km$^2$).

The mean effective rainfall intensity $r_e$ was calculated using Eq. (2). The peak runoff coefficient $f_p$ is defined using the Mononobe method (MAFF 2003). $f_p$ is a coefficient based on the rainfall infiltration into the basin from the surface. The reservoir is located in a mountainous area; therefore, this study used a value of $f_p = 0.85$, which applies to mountainous conditions.

$$r_e = r \times f_p$$

where $r_e =$ the mean effective rainfall intensity (mm/h), $r =$ the observed rainfall intensity (mm/h), and $f_p =$ the peak runoff coefficient $= 0.85$.

Eq. (3) by Kadoya and Fukushima (1976) was used for calculating the flood arrival time $t_p$. $t_p =$ the time taken for the rain that falls at the farthest point of the basin to reach the apogee of the basin.

$$t_p = C \times A^{0.22} \cdot 2r_e^{-0.35}$$

where $t_p =$ flood arrival time (min); $C =$ constant governed by land surface conditions $= 290$.

From $Q_p$ and $t_p$, the flood flow $Q_d$ for at a specific time was calculated using Eq. (4) and (5) from MAFF 2003. The duration of water level rise is the period from a specified time till $t_p$. Moreover, the riod after $t_p$. 

![Figure 6. 10-minute rainfall and hourly rainfall used in the seepage flow simulation (Yoshinotani reservoir).](image-url)
period of water level rise \( \frac{t_u}{t_p} = \frac{Q_d}{Q_p^{0.6}} \) \hspace{1cm} (4)

period of water level fall \( \frac{t_d}{t_p} = P \frac{1 - (Q_d/Q_p)}{(Q_d/Q_p)^{0.4}} \) \hspace{1cm} (5)

where \( t_u = \) time from hydrograph rise time (min), \( t_d = \) time from hydrograph peak (min), \( Q_d = \) flood flow (m\(^3\)/s), \( P = \) constant value = 1.

The overflow depth \( H_d \) for the canal inflow type spillway was calculated using Eq. (6) using \( Q_d \) at different times. Moreover, the fluctuation in the water level of the reservoir was determined by adding it to the full water level.

\[ B = \frac{Q_d}{1.704 \times C \times H_d^{3/2}} \] \hspace{1cm} (6)

where \( B = \) spillway channel width = 1.3 m (Yoshinotani), 1.2 m = (Fukuida); \( H_d = \) overflow depth (m); \( C = \) inflow coefficient = 0.821

Equations (1) to (6) are derived from the design guidelines (MAFF 2003 and MAFF 2015).

### 4.2. Seepage flow and slip stability simulations

The unsteady saturated–unsaturated seepage flow simulation was conducted for the Yoshinotani reservoir using the FEM model shown in Figure 7. The surface of the embankment was given a 10-minute rainfall and the water level fluctuation calculated simultaneously (in 10-minute increments) to determine the time course of the seepage line in the embankment. The 1 min rainfall obtained from DIAS was recalculated for 10 min rainfall. Since the analysis time is enormous, the 2520 min rainfall period was separated by 10 min intervals. The boundary conditions are shown in Figure 7. An initial water level of 6.36 m was provided, and the upstream slope was the water level fluctuation boundary. The top of the embankment and downstream slope are the boundaries of rainfall infiltration. The part above the water level on the upstream slope becomes the rainfall infiltration boundary. One week before August 27, as shown in Figure 1, no rainfall exceeding 10 mm/h was recorded. Therefore, it is considered that the influence of the rainfall preceding August 27 was small. Moreover, the program used for the simulation was AC-UNSAF2D (Academic- UNsaturated-Saturated Analysis program by Finite element method-2Dimension), which was developed at Okayama University, Japan. The foundation ground was an impermeable layer. The governing equation in this program is expressed using the continuity equation derived from the conservation of the mass equation and Eq. (7) that extends Darcy’s law to the unsaturated region (Akai et al. 1977).

\[ \frac{\partial}{\partial x} \left( k \frac{\partial \psi}{\partial x} \right) + \frac{\partial}{\partial y} \left( k \frac{\partial \psi}{\partial y} + k \right) = \left( C + \alpha \cdot S_s \right) \frac{\partial \psi}{\partial t} \] \hspace{1cm} (7)
where $x$ = horizontal axis of the cross section of the reservoir; $y$ = vertical axis of the cross section of the reservoir; $k$ = permeability coefficient (m/s); $\psi$ = pressure head (m); $C$ = specific water capacity (1/m); $z = 1$ is the saturation region, $z = 0$ is the unsaturation region, $S_s$ = specific storage coefficient (1/m), and $t$ = time (s). The value of $C$ is determined from the tangential gradient of the correlation between the pressure head $\psi$ and the volume of the water content. In the seepage simulation, when the entire embankment was saturated, the amount of rainfall provided to the model did not infiltrate. Surface flow occurred in the actual embankment; however, the simulation did not consider surface flow.

The time course of the seepage line in the embankment for three cases was obtained. In Case 1, the fluctuation of the water level was considered exclusively. In Case 2, the water storage was maintained constant at the full level, and the fluctuations in rainfall infiltration through the embankment surface was considered. In case 3, fluctuations in the water level and rainfall infiltration were considered simultaneously. The safety factor $F_s$ for slip failure on the downstream slope of the embankment was calculated using the Fellenius method, which was modified by Turnbull and Hvorslev (1967), as shown in Eq. 8 based on the seepage line in the embankment obtained from the seepage flow simulation. The slip surface was fixed as shown in Figure 3. The time course of $F_s$ for each slip surface of both reservoirs with respect to the fluctuations in rainfall infiltration was calculated.

$$F_s = \frac{\Sigma \{ c' l + (W - ub) \cos z \tan \phi' \}}{\Sigma W \sin z}$$  \hspace{1cm} (8)

where $F_s$ is the safety factor, $W$ is the weight of the slice (kN/m), $u$ is the pore water pressure (kPa), $b$ is the slice width (m), $l$ is the slice length (m), $z$ is the slope angle (°), $c'$ is the effective cohesion (kPa), $\phi'$ and is the angle of internal friction (°).

The properties of each reservoir embankment in simulation are listed in Table 3. The results of the variable permeability test for the specimen reconstructed based on the state quantity of the undisturbed sample were used for the saturated permeability coefficient $k_s$. The $CU$ test results for undisturbed samples were applied to the unit volume weight and soil strength parameters. The $\gamma_{sat}$ and $\gamma_t$ were saturated weight and wet weight, respectively.
The specific storage coefficient \( S_s \) was \( S_s = \frac{1}{2C^4} \) (1/m). The unsaturated infiltration characteristics are the parameters of sandy soil calculated using the VG model (Van Genuchten 1980) based on the experimental values obtained by Nishigaki (1991) (Figure 8). This parameter was selected because the saturated volume moisture content calculated from the specimen used in the triaxial test were consistent with the existing experimental values.

### 5. Result and discussion

Figure 9(a) illustrates the changes over time in the water level and 10-minute rainfall over the Yoshinotani reservoir during the simulation period. The water level changed repeatedly after 9:20 am on August 27, 560 min after the start of rainfall. The full water level of 6.57 m was reached at 5:40 am on August 28, 1780 min later. The water level increased by 0.2 m from the full water level of 6.36 m due to heavy rain. Water does not overflow at the reservoir level calculated in this study because the height of the embankment is 7.56 m. This was verified during the field survey. Figure 9(b) illustrates the change over time in the water level of the Fukuida reservoir and the 10-minute rainfall. The water levels in the Fukuida reservoir has repeatedly risen and fallen but has not overflowed like the Yoshinotani reservoir.

Figure 10 demonstrates a comparison of the seepage lines for each simulation 560 min and 1780 min after the start of rainfall in the Yoshinotani reservoir. The fluctuation of the water level in Figure 10(a) begins at 560 min after the start of rainfall. The seepage line at 560 min does not change from the initial state because the water level does not always change from the full water level in Case 1. In cases 2 and 3, the seepage line begins to rise marginally from the downstream slope due to rainfall. The seepage lines for Case 2 and Case 3 coincide at 560 min because the impact of the

Table 3. Soil parameter used for seepage flow and stability analysis.

| Reservoir   | \( k_s \) (m/s) | \( S_s \) (1/m) | \( \gamma_{sat} \) (kN/m³) | \( \gamma_t \) (kN/m³) | \( c' \) (kPa) | \( \phi' \) (°) |
|-------------|----------------|----------------|----------------------------|------------------------|---------------|---------------|
| Yoshinotani | \( 1.7 \times 10^{-6} \) | \( 1.0 \times 10^{-4} \) | 19.3                       | 18.9                   | 5.2           | 32.8          |
| Fukuida     | \( 7.7 \times 10^{-7} \) | \( 1.0 \times 10^{-4} \) | 18.9                       | 17.7                   | 3.5           | 36.3          |
Figure 9. 10-minute rainfall and water level changes over time.

Figure 10. Comparison of seepage lines in each seepage flow simulation case (Yoshinotani reservoir).
The increase in water level is not reflected in the seepage line. The seepage line on the upstream side increased in Case 1 at 1780 minutes at the maximum water level in Figure 10(b). The increase in the seepage line is small because the amount of increase in the water level is 0.2 m. However, seepage lines for Cases 2 and 3 are located above that for Case 1. Moreover, the seepage lines near the upstream and downstream slopes are rising due to the seepage of rainfall from the surface of the embankment. Therefore, the rainfall infiltration has a greater impact on the rise of the embankment seepage line than on the rise of the water level. However, the lower the $k$-value the more difficult it is for the rainfall infiltration. (Huat et al. 2006; Rahardjo et al. 2007; Li et al. 2013). Rahardjo et al. (2007) simulated that, for example, when $k$ is less than $10^{-6}$ m/s, the slope is stable for less than 24 h of rainfall. In the case of this study’s reservoir, the value of $k$ was $10^{-6}$ m/s or less. The embankment destabilized when considering the long-term rainfall for approximately two days. The effect of the rainfall pattern on the stability of the embankment will be described in subsequent sections.

Figure 11 illustrates the contour diagram of the degree of saturation in the Yoshinotani reservoir embankment in Case 3. The entire embankment was saturated.
except for a part on the inside of the embankment at 1780 min because of the seepage of rainfall from the embankment surface.

Figure 12 illustrates the time course of \( F_s \) in the Yoshinotani reservoir for each simulation case. \( F_s \) is the slip surface shown in Figure 3. \( F_s \) at \( t = 0 \) was calculated using the seepage line obtained by steady-state simulation at the full water level. The initial value of \( F_s \) is 1.14 in each simulation case. In Case 1, no discernible change was observed in the seepage line during the simulation as shown in Figure 10. Hence, \( F_s \) did not change significantly. Moreover, \( F_s \) gradually decreased with the start of rainfall in Cases 2 and 3. The 10 minutes rainfall of less than 5 mm continued intermittently up to 1500 min after the start of rainfall. \( F_s \) decreased to 1.02. In both cases, \( F_s \) decreased below 1.0 due to the heavy rainfall after 1500 min. As shown in Figure 10, no significant difference was observed in the seepage lines of both cases. Therefore, the \( F_s \) values for Cases 2 and 3 remained same throughout the simulation. Moreover, a steady-state seepage flow simulation with a maximum water level of 6.57 m and \( F_s \) calculated using the seepage line based on the current reservoir design guidelines (MAFF 2015) was performed. As a result, \( F_s \) was greater than 1.0. The initial value of \( F_s \) in Cases1-3 is F.W.L = 6.36 m and the water level is constantly 6.57 m in steady-state simulation. Therefore, the \( F_s \) of the steady-state simulation is smaller than initial \( F_s \) of in Cases 1-3. Figure 13 illustrates the time-dependent changes in \( F_s \)
of the Fukuida reservoir against slip failure. The time course of $F_s$ in each case is approximately the same as that of the Yoshinotani reservoir.

In the Yoshinotani reservoir, heavy rainfall of up to 10 mm/10 min occurred for approximately 4 h after the preceding rainfall of up to 3 mm/10 min continued for approximately 24 h. The rainfall after 1500 min and the water level fluctuations for Case 3 were used to examine the difference in $F_s$ with respect to the rainfall pattern. Figure 14 illustrates the results of calculating the change over time for $F_s$. From Figure 14, $F_s$ exceeds 1.0 when preceding rainfall is absent. Therefore, the seepage line gradually rose due to the preceding rainfall, and the seepage line rose significantly due to heavy rainfall resulting in the destabilization of the embankment.

The stability simulation method used in this study reproduced the phenomenon of slip failure of the embankment. In the simulation method of this study, the unsaturated seepage characteristics were determined using the existing estimation formula. It is necessary to verify whether the failure can be reproduced when experimental unsaturated seepage characteristics of the embankment soil are applied. However, the focus of our research was to simulate the stability of an embankment with respect to the rainfall infiltration from its surface. This method is not listed in the Japanese reservoir design guidelines (MAFF 2015). The stability simulation results for the embankment were completely different, depending on the presence or absence of rainfall. This study clarified the effects of rainfall infiltration on the stability of embankments under certain conditions. The failure cases of other reservoirs will be collected for future studies. Furthermore, a sensitivity analysis is performed by changing the amount of rainfall and soil parameters, which contributes to the establishment of a method for designing reservoir embankments in consideration of rainfall infiltration. In this study, the slip surface was fixed and the limit equilibrium method was used for stability analysis. In the future, we would simulate the uplift and seepage force due to changes in the seepage line with FEM. Moreover, the failure process from local embankment slope destruction to large-scale slip failure will be examined.

6. Conclusion
This study clarified the mechanism of the slip rupture in an agricultural reservoir caused by heavy rainfall in August 2019. The main findings are summarized as follows:

1. Heavy rainfall exceeding 60 mm/h occurred for approximately 4 h after the continuous rainfall with a maximum of 20 mm/h for approximately 24 h in the region of the Yoshinotani and Fukuida reservoirs. Slip and seepage failure occurred in four reservoirs in Takeo City due to the heavy rainfall. Seepage flow and slip stability simulation were performed based on water level fluctuation and rainfall infiltration in damaged reservoir embankments using long-term rainfall data.

2. The water level during heavy rain was determined using a rational formula. The calculated reservoir level did not exceed the embankment height in both the Yoshinotani and Fukuida reservoirs. This result is consistent with the local situation.
The increase in rainfall infiltration had a greater impact than the rise in water level in the reservoir on the rise in seepage line in the embankment. The safety factor of the slip surface decreased significantly due to the impact of rainfall infiltration, and the occurrence of slip failure in the in-situ position could be reproduced.

Therefore, the proposed stability simulation method can effectively evaluate the stability of embankments during heavy rainfall.

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