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Large-Scale Experiment and Numerical Modeling of Riverine Levee Breach

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6 Abstract: This study aims to clarify the mechanism of riverine levee breach and propose a new 7 numerical model for that phenomenon. We performed large-scale experiments of overtopping breach using an experimental flume located on the floodway of an actual river channel. By taking advantage 8 of the scale of the flume, we monitored the levee breach process with state-of-the-art observation 9 devices under highly precise hydraulic conditions. We performed four test cases with variations of 10 inflow rate, levee material and levee shape, and monitored the levee breach quantitatively using 11 12 acceleration sensors installed in the levee body. From the results of the experiments, we categorized the breach process into four stages, focusing on the breach progress and hydraulic characteristics. 13 We determined that the correlation between the breached volume and the hydraulic quantities of 14 velocity, water level and Shields number can be expressed by an equation similar to that for bed load 15 transport. Finally, we proposed a two-dimensional numerical model by integrating the experimental 16 results into geomechanics, and we obtained a fine reproduction result. 17

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24 Introduction

Recent years have seen a considerably increased incidence of typhoons, torrential rainstorms and other 25 extreme meteorological phenomena due to climate change, thereby raising the risk of large-scale 26 disasters caused by riverine floods. The flood damage is particularly severe when levee breaches occur, 27 so estimating the flood magnitude and providing hazard maps are crucial for risk management. In light of 28 this, we identified an urgent need to clarify the mechanism of riverine levee breach. Most of the riverine 29 levees are earthen embankments, and a variety of research on earthen embankment breaches has been 30 31 done for decades. That research can be classified as field case studies, laboratory experiments and breach 32 models.

Firstly, field case studies: These are among the most important ways of studying embankment breaches; however, despite considerable effort, only a limited number of real-life, real-time breach events have been investigated because of observation difficulties and safety concerns.

Next, laboratory experiments: Wahl (2007) reviewed a large number of laboratory embankment breach 36 experiments performed over the course of several decades. Most of these experiments were found to be 37 small-scale, except for a few experiments such as the European IMPACT project (Morris and Hassan 38 2005), the USDA-ARS project (Hanson et al. 2005; Hunt et al. 2005) and Sattar et al. (2008) who 39 performed a mobile bed experiment of the 17th street canal breach by hurricane Katrina and stated the 40 bed of breach was variable. Embankment breach, which depends on interaction among flows, sediment 41 transport and corresponding morphological changes, is so complex that laboratory experiments, 42 especially small-scale ones, encounter scale effects and simplifications that make it difficult to 43 understand the breach processes and to collect reliable data toward developing embankment breach 44 models. Wahl's review also clarified that most of the experiments, including the aforementioned two 45 projects, addressed dam embankment breaches, whereas very few experiments addressed riverine levee 46 breaches. 47

Thirdly, breach models: Many models have been proposed, and the ASCE/EWRI Task Committee on Dam/Levee Breaching (2012) reviewed such models and classified them as parametric models, simplified physically based models or detailed physically based models. According to the review, most of these models addressed overtopping dam breaches, and only a few models addressed riverine levee breaches.

Here we attempt to clarify the different characteristics of the dam embankment breaches and riverine levee breaches. Morphologically, the dam breaches are characterized by vertical progress, in contrast to the horizontal advancement of riverine levee breaches. Hydraulically, an overflow direction perpendicular to the cross section characterizes dam breaches, in contrast to oblique overflow for riverine levee breaches. Moreover, for dam breaches, the inflow decreases rapidly as the breach advances and the reservoir becomes empty, whereas for the riverine levee breaches, the inflow continues unless the upstream flood recedes.

60 Considering the aforementioned different characteristics of dam breach versus riverine breach, we assume that the results from the dam breach experiment can be applied only to the very initial stages of 61 riverine levee breach and not to the later stages. This is because the horizontal scale versus the vertical 62 scale for riverine levee breaches may exceed 100 or more, and such scales are beyond the scope of 63 previous dam breach experiments. Therefore we realized that proper experiments were needed to 64 65 reproduce the riverine levee breach, which is characterized by lateral breach widening normal to river flow. To obtain reliable data, we performed large-scale experiments using the Chiyoda Experimental 66 Flume (hereinafter: "the flume"). 67

The flume is in the floodway of the Tokachi River in Hokkaido, Japan, and these facilities are attached to the fume, such as the inflow control gate and observation bridges. In the experiments, we simulated riverine levee breaches by erecting a model levee with an overtopping notch on the right side of the flume, and we performed four test cases with varied inflow discharges, levee materials and levee structural configurations. We studied general characteristics of the riverine breach, and also obtained hydraulic data as well as morphological data during the breach process. We also applied a twodimensional numerical model and modified it by integrating the experimental results into the geomechanics.

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77 Description of Experiment

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79 Experiment facilities and test conditions

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Figure 1 is an aerial photo of the flume. The flume specifications are shown in Figure 2. We 81 constructed the flume on the floodway channel of the Tokachi River, with a separation levee on the 82 83 right side and vertical steel sheet piles on the left side. The bed slope was approximately 1/500. We also created an overflow area with a width of 80 m or more on the right side using part of the floodway 84 channel. In the breach experiment section, we replaced a portion of the separation levee with a model 85 levee that is made of homogeneous material and with a bare surface (an embankment with no turf) and 86 a rectangular notch, 0.5 m in depth and 3 m in width, to trigger a breach. We constructed a dam-up 87 88 facility at the downstream end of the flume to maintain the proper water level. To prevent bank erosions, 89 we placed revetment works on the upstream slope side of the levee breach experiment section.

90 The test cases have variations in inflow discharge, levee configuration (crest width) and levee 91 materials, as shown in Table 1. The grain size distribution curves of the levee materials are shown in 92 Figure 3. The bed material was roughly similar to the materials in Cases 1 and 2.

93

94 Measurement method

We placed water level sensors every 25 m in the Flume and every 40 m in the overflow area along 95 the levee. We also placed flow meters in the channel at 50 m upstream and 120 m downstream from the 96 overtopping notch, respectively, so that the overflow rate could be calculated from the balance of these 97 flow rates. We applied Particle Image Velocimetry (PIV) to measure the surface flow rate distribution 98 at the levee breach section. A unique feature of the experiments was that we placed accelerometers 99 100 inside the levee body and substrata to monitor the breach process under overflowing water. The 101 accelerometers were installed at intervals of 1.5 m in the cross-sectional and vertical directions, and 2 m in the longitudinal direction. 102

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- 104 Test Results
- 105

106 Water level observation

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The results of water level observations in the flume and the overflow area are shown in Figure 4 (top). 108 Dotted lines indicate the notch height, and dashed lines indicate the time at which the gate closing 109 began. In each case, the water level rose until it reached the target height and remained constant for a 110 111 while. When the breach began to widen laterally, the water level fell and then kept constant. The tail water level in the overflow area began to rise when the head water level fell, and the difference between 112 these water levels decreased. As indicated by the correlation between the head water level and the 113 breach width, when the width was approximately 10 m in Cases 1 and 2 or 30 m in Cases 3 and 4, the 114 head water began to fall and the water flowed quickly into the levee breach opening. 115

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117 Flow rate observation

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Figure 4 (middle) shows the flow rates in the upstream and the downstream of the levee breach section, as well as the overflow rates, which we calculated from the balance of the upstream and the downstream flow rates with consideration of increase or decrease of the water storage quantities in the breach section of the flume. The overflow rate increased a little at the initial overtopping stage in each case and began to increase significantly when breach widening was initiated. After the overflow rate peaked, it remained roughly constant until the gate was closing and the inflow began to decrease.

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126 Levee breach observation

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Figure 4 (bottom) shows the time history of the breach width along the center of the levee crest as measured by video images taken from above. The vertical axis indicates the distance from the notch, where plus means downstream direction and minus means upstream direction. The longitudinal axis indicates the time elapsed from the beginning of initial overtopping. The levee breach process was monitored by the accelerometers, and Figure 5 shows a time series of breach opening shapes for Case 1, which we estimated from the accelerometer results. The black dots indicate the position of the sensors that flowed out.

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136 Characteristics of Breach Process

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138 Beginning of levee breach

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As shown in Figure 5, the initial breach stage began with downstream slope erosion. The erosion retrograded from the top of the downstream slope to the top of the upstream slope. After the erosion reached the top of the upstream slope, the breach opening began to widen gradually in the upstream and downstream directions. A similar process was seen in the other cases. As shown in Figure 4 (bottom), this initial stage took longer in Cases 3 and 4 than in Case 1. We assumed that the factor causing this for Case 3 was the relatively higher soil cohesion, which may have slowed the erosion processes, and that the factor causing this for Case 4 was the larger cross-sectional volume of the levee to be eroded.

Here we must mention that the notch placed on the levee top might have affected the processes in the beginning stages. So further studies are needed to clarify the experiments could reproduce the early stages correctly or not.

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151 Lateral widening of breach opening

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As shown in Figure 4 (bottom) and mentioned in the previous chapter, the breach opening began to widen gradually in the upstream and the downstream directions. Then, the opening began to widen in the downstream direction very rapidly. The rate of widening speed kept almost constant for a while. In Case 2 where the inflow was low, the rate of widening speed was lower than other cases. In Case 3 where the levee material was relatively fine, the levee collapsed in bulk repeatedly and the rate of breach widening was higher than for other cases of coarse material.

Next, we examined the process of side breaching in detail. From the observations, we noticed that the lower part of the levee, which was hit by the water first, failed and subsequently the upper part lost support and collapsed. Comparing the timing of breaching at the crest (measured by video image from above) and below the crest (measured by accelerator sensors) at the same cross-section, we realized that 163 the whole structure in every case breached almost simultaneously. Also, we realized that in Case 4 the 164 slightly remained lowest structure and the substrata were degraded later than the breach.

Another characteristic we found was that the downstream slope side failed before the upstream slopeside.

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168 Levee breach and sedimentation process

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Comparing Figure 4 (middle) and Figure 4 (bottom), even when the overflow rate was nearly 170 constant in the later stage, the breach widening continued. We estimate the reasons as follows. Figure 6 171 172 shows the surface flow rate distributions for Case 3 from PIV observation. A narrow band of highvelocity flow, 4 m/s or more, appeared and struck the side of the breached levee. When the breach 173 width reached approximately 30 m, the band became nearly half the entire breach width. Even when the 174 breach width reached approximately 50 m, the width of the band remained roughly the same as before, 175 176 and a dead-water area appeared near the upstream end of the breach opening where deposition may have occurred. We summarize this process as follows. The high-velocity band erodes the levee side and 177 178 moves in the downstream direction, then, sedimentation occurs in the upstream area. This repeated process makes the breach widen in the downstream direction as the band moves downstream, keeping 179 almost constant width. 180

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182 Summary of breach process stage

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In consideration of the breach processes observed in the tests, we categorized the levee breachprocess into the four stages shown in Figure 7.

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- 187 Quantitative Evaluation of Breach Process
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189 Formula for breach morphology

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As mentioned in Chapter 1, the main purpose of this study is the evaluation of the breach lateral 191 widening process toward developing a numerical riverine breach model. Therefore we focused on the 192 3rd and 4th breach stages for analyzing the breach widening process. Although we mentioned in 193 194 Chapter 4.2 that the breach process of the upper part of the levee differs from that of the lower part, as shown in Figure 8, we assumed that both parts would be subject to the same mechanism, because the 195 upper part collapses onto the foot of the lower part and then washes out by the flow, which is the same 196 mechanism as for the lower part. Therefore we also assumed that the total volume of breached levee 197 transported per unit time and width (hereinafter: "breached load transport") could be expressed by a 198 199 kind of the sediment load transport formula, and we proposed eq. (1) with reference to the Meyer-Peter 200 and Müller equation

201
$$q_B = \alpha (\tau_* - \tau_{*c})^\beta \sqrt{sgd^3} \quad (1)$$

where q_B = breached load transport; α and β are coefficients; τ_* = Shields number; τ_*_c = critical Shields number; *s*= submerged specific sediment density; *g*= gravitational acceleration; and *d*= mean diameter of the levee material. Then q_B can be calculated from the experimental results as

205
$$q_B = (1 - \lambda) \frac{dV}{dt} \frac{1}{L}$$
(2)

 $206 \qquad \frac{dV}{dt} = \frac{dV_1}{dt} + \frac{dV_2}{dt} (3)$

where V= breached volume; $V_1=$ breached volume for the lower part; $V_2=$ breached volume for the

- 208 upper part; t= time; λ = porosity of the levee material; and L= characteristic length.
- **209** Then we obtained α and β through the comparison of eq. (1) and (2).
- 210

211 Analysis of test results

We applied the experimental results to eq. (1) and (2), to obtain α and β as follows. We assumed λ should be 0.4, *d* should be d_{50} , and *s* should be 1.65. Then we calculated dV/dt (= dV_1/dt + dV_2/dt) every 5 minutes from the acceleration sensor data. The results are shown in Figure 9. Shields number τ_* is calculated as

216
$$\tau_* = \frac{N_m^2 u_m^2}{sdh^{\frac{1}{3}}}$$
 (4)

where N_m = manning's roughness coefficient (=0.023); u_m = depth-averaged overflow velocity; and h= water depth. We set a value for the flow velocity u_m as the surface flow rate measured by PIV near the downstream slope of the breach opening end where the flow struck and eroded the levee. We also set a value for the water depth h calculating the balance of the water surface elevations measured by 3D analysis and the initial bed elevations. (Bed level change should be considered but we did not have such data near the downstream slope.) We set a value for the critical Shields number τ_{*c} as 0.05 by applying the method of Iwagaki (1956). Lastly we applied the levee bottom width as the characteristic length.

We plotted the test results as shown in Figure 10. We observed different breach characteristics for each test case; however, the plotted results showed a correlation expressed by the following equation

226
$$(1-\lambda)\frac{dV}{dt}\frac{1}{L}\frac{1}{\sqrt{sgd^3}} = 18(\tau_* - \tau_{*_c})^{1.5}$$
 (5)

We found that the coefficient β in eq. (1) had the value of 1.5, which was similar to the value appearing in sediment load transport formulas such as those of Meyer-Peter and Müller. This finding is expected to be useful in the development of a levee breach model based on such a simple equation for simulating the lateral widening breach process.

231

232 Numerical Simulation

233

234 Conventional numerical model

235

236 Faeh (2007) developed a numerical model based on shallow-water equations considering both bed load and suspended load. The model also has the ability to incorporate slope stability. He evaluated the 237 ability of the model to simulate levee breaches on the Elbe River. The model was able to simulate the 238 main characteristic of the flow field, but it was found to be still limited. He also pointed out that the 239 most sensitive parameters were critical angles describing the bank failure mechanism and, thus, 240 implicitly the ratio between vertical and horizontal erosion. For the simulation of the experimental 241 242 result (Case 4), we assumed that suspended load could be neglected because the main levee materials were sand and gravel. Therefore we employed the two-dimensional model "Nays" (Shimizu 1996, Jang 243 and Shimizu 2005, iRIC 2013), which is based on shallow-water flow and bed load transport. The 244 model also has the ability to incorporate slope stability (repose angle) for bank erosion. The governing 245 equations for the model in the orthogonal coordinate system are given as follows, 246

247 [Continuity equation for flow]

248
$$\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0$$
 (6)

249 [Momentum equation for flow]

250
$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} = -g \frac{\partial H}{\partial x} - \frac{C_f u}{h} \sqrt{u^2 + v^2} + v_t \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right)$$
(7)

251
$$\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} = -g \frac{\partial H}{\partial y} - \frac{C_f v}{h} \sqrt{u^2 + v^2} + v_t \left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right)$$
(8)

252 [Sediment transport equation]

253 $q_b = 8(\tau_* - \tau_{*_c})^{1.5}$ (9)

254
$$q_b^x = \frac{u}{\sqrt{u^2 + v^2}} q_b$$
, $q_b^y = \frac{v}{\sqrt{u^2 + v^2}} q_b$ (10)

255 [Continuity equation for sediment]

256
$$\frac{\partial Z_b}{\partial t} + \frac{1}{1 - \lambda} \left(\frac{\partial q_b^x}{\partial x} + \frac{\partial q_b^y}{\partial y} \right) = 0 \quad (11)$$

where *x*, *y* are orthogonal coordinates; *h*=water depth; *t*=time; *u*, *v* are velocity in the *x* and *y* directions, respectively; *H*=water level elevation; C_f =riverbed shear coefficient; v_t =eddy viscosity coefficient; q_b =total bed load transport per unit width; q_b^x , q_b^y are bed load transport per unit width in the *x* and *y* directions, respectively; Z_b =riverbed elevation; λ =porosity of the levee material.

It is useful to write the sediment transport equation in the general coordinate system because we modifythe equation in the general coordinate system later in this paper.

263
$$\frac{\partial}{\partial t} \left(\frac{Z_b}{J} \right) + \frac{1}{1 - \lambda} \left\{ \frac{\partial}{\partial \xi} \left(\frac{q_b^{\xi}}{J} \right) + \frac{\partial}{\partial \eta} \left(\frac{q_b^{\eta}}{J} \right) \right\} = 0$$
(12)

264 where ξ , η are general coordinates; q_b^{ξ} , q_b^{η} are bed loads transport in the ξ and η directions, 265 respectively; *J*= jacobian of the coordinate transformation.

Figure 11 shows the calculation area and the boundary conditions. In the simulations, we made a set of elevation data of the flume, the notched levee, and the overflow area for geographical input data to reproduce Case 4. We also simulated inflow as the experimental result of Case 4. Then we made the calculation settings shown in Table 2. We set the grid size as 1 m by 1 m for Run 1, 2, and 3. 270 Figure 12 shows the time history of the breached volume for each calculation run and the experimental result. The experiment took longer time in the early stage which could not be reproduced 271 by calculation, so we shift the experimental result so that the data starts from the beginning of breach 272 widening. Focusing on the breach widening speed, Run 1 (repose angle of 30 degrees) reproduced 273 better results but was still slower than the experimental result. Numerical models using critical failure 274 angles are known to depend strongly on a fine grid size, therefore we calculated Run 4, in which the 275 grid size was 0.5 m by 0.5 m. Run 4 showed almost the same result as Run 1, so we recognized that the 276 277 grid size of 1 m by 1 m was reasonable.

Next, we analyzed the simulation results and observed breach mechanisms. As observed in the experiment, the lower part of the levee hit by the water was eroded laterally and the levee side surface of the collapsed part is very steep (almost 90 degrees). However, the numerical model neither includes a lateral erosion mechanism nor does the repose angle reflect the observation results. In the breach widening stage, the lateral erosion process becomes the primary mechanism of breach. Therefore we suppose that improved lateral erosion formulas need to be developed.

284

285 Proposed model

286

We modified only formulas relating morphodynamics in the aforementioned model. Instead of using the sediment transport formula and the slope stability model, we brought an empirical formula delivered from the experimental results. First, we separate the calculation cells of the levee (except near the notch) from other cells. We call the former "levee cells" and the latter "normal cells". For the levee cells, we propose a breach rate formula which is different from the sediment transport formula for the normal cells. Referring to the eq. (5), we assume the breached loads to be collapsed in the orthogonal direction of the streamline as eq. (13) and (14). The breached loads are to be placed on adjacent normal cells and to be transported using the original transport formulas from the next time step. Figure 13 shows the streamline and coordinate systems.

296
$$q_{v}^{n} = \frac{1-\lambda}{L} \frac{dV}{dt} = 18\sqrt{sgd^{3}} (\tau_{*} - \tau_{*c})^{1.5}$$
 (13)

297
$$q_v^s = 0$$
 (14)

where q_v^n = breached loads orthogonal to the streamline; and q_v^s = breached loads along the streamline. As for the characteristic length L, it can be represented by the calculation cell width.

Here we describe the detailed calculation processes for breached loads. Referring to the governing equations developed by Jang and Shimizu (2005), we transform q_v^s and q_v^n into the general coordinate system as

$$q_{\nu}^{\xi} = \left(\frac{\partial x}{\partial s}\frac{\partial \xi}{\partial x} + \frac{\partial y}{\partial s}\frac{\partial \xi}{\partial y}\right)q_{\nu}^{s} + \left(\frac{\partial x}{\partial n}\frac{\partial \xi}{\partial x} + \frac{\partial y}{\partial n}\frac{\partial \xi}{\partial y}\right)q_{\nu}^{n} = \left(\cos\theta_{s}\xi_{x} + \sin\theta_{s}\xi_{y}\right)q_{\nu}^{s} + \left(-\sin\theta_{s}\xi_{x} + \cos\theta_{s}\xi_{y}\right)q_{\nu}^{n} \quad (15)$$

$$q_{\nu}^{\eta} = \left(\frac{\partial x}{\partial s}\frac{\partial \eta}{\partial x} + \frac{\partial y}{\partial s}\frac{\partial \eta}{\partial y}\right)q_{\nu}^{s} + \left(\frac{\partial x}{\partial n}\frac{\partial \eta}{\partial x} + \frac{\partial y}{\partial n}\frac{\partial \eta}{\partial y}\right)q_{\nu}^{n} = \left(\cos\theta_{s}\eta_{x} + \sin\theta_{s}\eta_{y}\right)q_{\nu}^{s} + \left(-\sin\theta_{s}\eta_{x} + \cos\theta_{s}\eta_{y}\right)q_{\nu}^{n} \quad (16)$$

where q_v^{ξ} = breached loads in the ξ direction; and q_v^{η} = breached loads in the η direction; θ_s = angle of the streamline to the x-axis as given by eq. (17) and (18); and $\xi_x = \partial \xi / \partial x$, $\xi_y = \partial \xi / \partial y$, $\eta_x = \partial \eta / \partial x$, and $\eta_y = \partial \eta / \partial y$.

308
$$\sin \theta_s = -\frac{\partial x}{\partial n} = \frac{\partial y}{\partial s} = \frac{v}{U}, \cos \theta_s = \frac{\partial y}{\partial n} = \frac{\partial x}{\partial s} = \frac{u}{U}$$
 (17)

309
$$U = \sqrt{u^2 + v^2}$$
 (18)

where U= composite velocity. From eq. (13), (14), (15), and (16), we obtain the breached loads of the general coordinates as

312
$$q_{\nu}^{\xi} = \left(-\sin\theta_{s}\xi_{x} + \cos\theta_{s}\xi_{y}\right)q_{\nu}^{n} = \left(-\sin\theta_{s}\xi_{x} + \cos\theta_{s}\xi_{y}\right) \left\{18\sqrt{sgd^{3}}\left(\tau_{*} - \tau_{*c}\right)^{1.5}\right\} (19)$$

313
$$q_{\nu}^{\eta} = \left(-\sin\theta_{s}\eta_{x} + \cos\theta_{s}\eta_{y}\right)q_{\nu}^{n} = \left(-\sin\theta_{s}\eta_{x} + \cos\theta_{s}\eta_{y}\right) \left\{18\sqrt{sgd^{3}}\left(\tau_{*} - \tau_{*c}\right)^{1.5}\right\} (20)$$

314 The breach rate formula is given as

315
$$\frac{\partial}{\partial t} \left(\frac{Z_L}{J} \right) + \frac{1}{1 - \lambda} \left\{ \frac{\partial}{\partial \xi} \left(\frac{q_v^{\xi}}{J} \right) + \frac{\partial}{\partial \eta} \left(\frac{q_v^{\eta}}{J} \right) \right\} = 0 \quad (21)$$

where Z_L = levee cell elevation; J= jacobian of the coordinate transformation. When applying this formula, Shields numbers are set as the largest value within 4 m from the levee cell in the same way of the experiment analysis. When the elevation of a levee cell comes below the riverbed, the cell becomes a normal cell.

320 Table3 shows the calculation conditions. Figure 14 shows the calculation results of time-series breached volume by the proposed model (Run 5). The model could not reproduce the early stage, but 321 when comparing the widening stage, the proposed model is found to be improved in terms of breach 322 widening speed and total volume. Next, we conducted some other comparisons of the test result and the 323 calculation result. Figure 15 shows comparisons of levee breach shapes and the flow distributions at the 324 325 same stage, and they are found to be well reproduced. Figure 16 shows comparisons of overflow 326 discharges and stage hydrographs at the channel center near the notch, and they are reasonably reproduced except discharge in the early timing. 327

Lastly, we checked the sensitivity by the constant values. Figure 14 also shows the calculation results using various values of α and β . We found that the value of β affect greater than the value of α , which can be supposed from the eq. (13).

331

332 Limitations and future improvement of modeling

333 The scope of our study is to reproduce the breach widening stage, so the poor performance by the

model in the early stage is not focused on. However, we recognize the importance to simulate the early stage of the breach. In the early stage, the flow and sediment interaction is significant like dam breaks, so the dam break models can be appropriate to use for the initial stage. Xia et al. (2010) used a coupled approach to solve simultaneously the flow and sediment transport processes induced by dam breaks. To applying coupled approach for the initial stage may be a future challenge. Furthermore we need investigations to find parameters for various levee materials or levee shapes other than the experiment conditions we performed in order to apply the model to real rivers.

341

342 Conclusions

343

We performed large-scale levee breach experiments and identified several characteristics of the
 breach process using sensors placed in the levee structure.

We categorized the breach processes into four stages. The breach began with downstream slope erosion (1st stage). After the erosion reached the top of the upstream slope, the breach opening began to widen gradually (2nd stage). The breach widened in the downstream direction rapidly and the overflow rate became maximum (3rd stage). The overflow rate kept constant and the breach rate decreased (4th stage).

351 3. We identified the breach widening mechanism, focusing on sedimentation and overflow
352 distribution. The narrow and strong overflow produced by combination of breach and deposition
353 eroded the levee side and moved in the downstream direction.

4. We identified a correlation equation to estimate the volume of breached levee.

355 5. We understand the importance of spiral flow effect, which could be crucial to determine the lateral356 breaching process. However, we proposed modifying two-dimensional numerical model which

357	cannot replicate such effect physically, and thus it has to be parameterized in some way, therefore
358	we integrated the empirical correlation equation delivered from the experimental result. The
359	modified model could not reproduce the early stage but well reproduced the breach widening stage.
360	
361	Notation
362	
363	The following symbols are used in this paper:
364	q_B = breached load transport
365	α , β = coefficients of breached load transport formula
366	$\tau_* =$ Shields number
367	τ_{*c} = critical Shields number
368	s = submerged specific sediment density
369	g = gravitational acceleration
370	d = mean diameter of the levee material
371	V = breached volume
372	V_1 = breached volume for the lower part of the levee
373	V_2 = breached volume for the upper part of the levee
374	t = time
375	λ = porosity of the levee material
376	L = characteristic length.
377	N_m = manning's roughness coefficient
378	u_m = depth-averaged overflow velocity
379	<i>h</i> = water depth

- q_v^n = breached loads orthogonal to the streamline
- q_v^s = breached loads along the streamline
- q_{y}^{ξ} = breached loads in the ξ direction
- q_v^{η} = breached loads in the η direction
- ξ, η = general coordinates
- x, y = orthogonal coordinates
- θ_s = angle of the streamline to the x-axis
- u = velocity in the *x* direction
- v = velocity in the *y* direction
- U = composite velocity
- Z_L = levee cell elevation
- J = jacobian of the coordinate transformation
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breach section

F1c

Overflow area

Case1, 2, 3(Case4)











Levee breach widening width $B \doteq 27 \text{ m}$



Levee breach widening width $B \rightleftharpoons 51 \text{ m}$

Legend for PIV flow rates 0_10m/s





















Legend for PIV flow rates ; \rightarrow 5m/s



Table 1.	Test case condition	
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Case	Levee Material	Crest	Inflow
	Size d ₅₀	Width	Discharge
	(mm)	(m)	(m^{3}/s)
1	5.4	3	70
2	4.9	3	35
3	0.2	3	70
4	0.7	6	70

Run	Maning's Roughness Coefficient	Material Size (mm)	Repose Angle (degrees)	Grid Size (m)
1	0.023	0.7	30	1×1
2	0.023	0.7	45	1×1
3	0.023	0.7	90	1×1
4	0.023	0.7	30	0.5×0.5

 Table 2.
 Calculation settings (conventional model).

Run	Maning's Roughness	Material Size	α	β
	Coefficient	(mm)		
5	0.023	0.7	18	1.5
6	0.023	0.7	9	1.5
7	0.023	0.7	36	1.5
8	0.023	0.7	18	1.0
9	0.023	0.7	18	2.0

 Table 3.
 Calculation settings (proposed model).