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Optimal Gate Operation of a Main Drainage Canal in a Flat Low–lying Agricultural Area using a Tank Model Incorporated with a Genetic Algorithm

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Chiyoda basin is located in Saga Prefecture in Kyushu Island, Japan. The total basin area is about 566 ha, which is a typical flat low–lying paddy–cultivated area. The main problem in this basin is the appropriate operation of drainage structures during flood events in order to avoid the local inundation that seriously damages the crop yield. This paper presents a mathematical model of a drainage system in Chiyoda basin for calculating the flood inundation and optimizing the operation of gates in a main drainage canal. The optimization of a gate operation was carried out by a genetic algorithm. The simulation process was executed using the rainfall with the return periods of 20, 50 and 100 years. The simulation results indicated that before the optimization of a gate operation, the inundation concentrates locally at some paddy tanks with a prolonged inundation time. The same situation occurred in river tanks. In the main drainage canal, there are some gates that play an important role in the drainage system. By optimizing the operation of these gates, the above issue can be solved. The result showed that the total inundation time in the paddy tanks decreased and the inundation distributed more equally.

INTRODUCTION

In flat low–lying areas, the appropriate operation of drainage structures during flood events is very important. However, there are problems because each drainage structure should be operated not only by considering the flood condition near the structure but also by considering the present flood condition over the wide area in the basin including upstream and downstream. The local concentration of inundation should be avoided in the operation of flood drainage structures in flat low–lying areas. From this point of view, this paper presents a mathematical model of a drainage system in Chiyoda basin for calculating the flood inundation and optimizing the operation of gates in a main drainage canal. The optimization of a gate operation was carried out by a genetic algorithm (Coley, 2003).

MATERIALS AND METHODS

Study area

Chiyoda basin is located in Saga Prefecture in Kyushu Island, Japan. It is surrounded by the Jobaru River to the west and the Tade River to the East. It is a typical flat and low–lying agricultural area. The total basin area is about 566 ha, which is covered mostly by paddy fields. The excess water during flood events is removed from the main drainage canal, which has a length of 7.5 km, to the Chikugo River through the flap gate and pumped drainage. However, it is assumed in this paper that the excess water is removed only by the flat gate as shown in **Fig. 1**.

Models for drainage and inundation analysis

A number of models have been proposed for drainage and inundation analysis in flat low–lying paddy–cultivated areas (Shikasho and Tanaka, 1985). The following section reviews several models, considering that this study should clarify the variation of the inundation area and depth in time and space.

Because short–term inundation, which does not result in severe damage of rice production, is permissible to some extent in paddy fields, the main concern in the early stage of the research was not the accuracy of the hydrograph but the amount of time required for the removal of the total rainfall in a drainage basin. Relatively simple models, which could be used with poor computer resources, were proposed in the 1950s and 1960s.

More complex models with high–performance have been proposed since 1970, with the rapid progress of computer calculation capabilities. Fukushima (1971) divided a drainage basin into unit areas and modeled the relationship between the outflow and the storage in a unit area by an exponential function with two parameters of conveyance capacity and initial storage. The outflow from each unit area was calculated using the function together with the continuity equation. His model was a parametric model because the two model parameters must be identified by the comparison of observed and calculated discharges. Suzuki and Nishihata (1969) replaced a drainage basin with four tanks of paddy field, lateral canal, main canal, and inundation area. The

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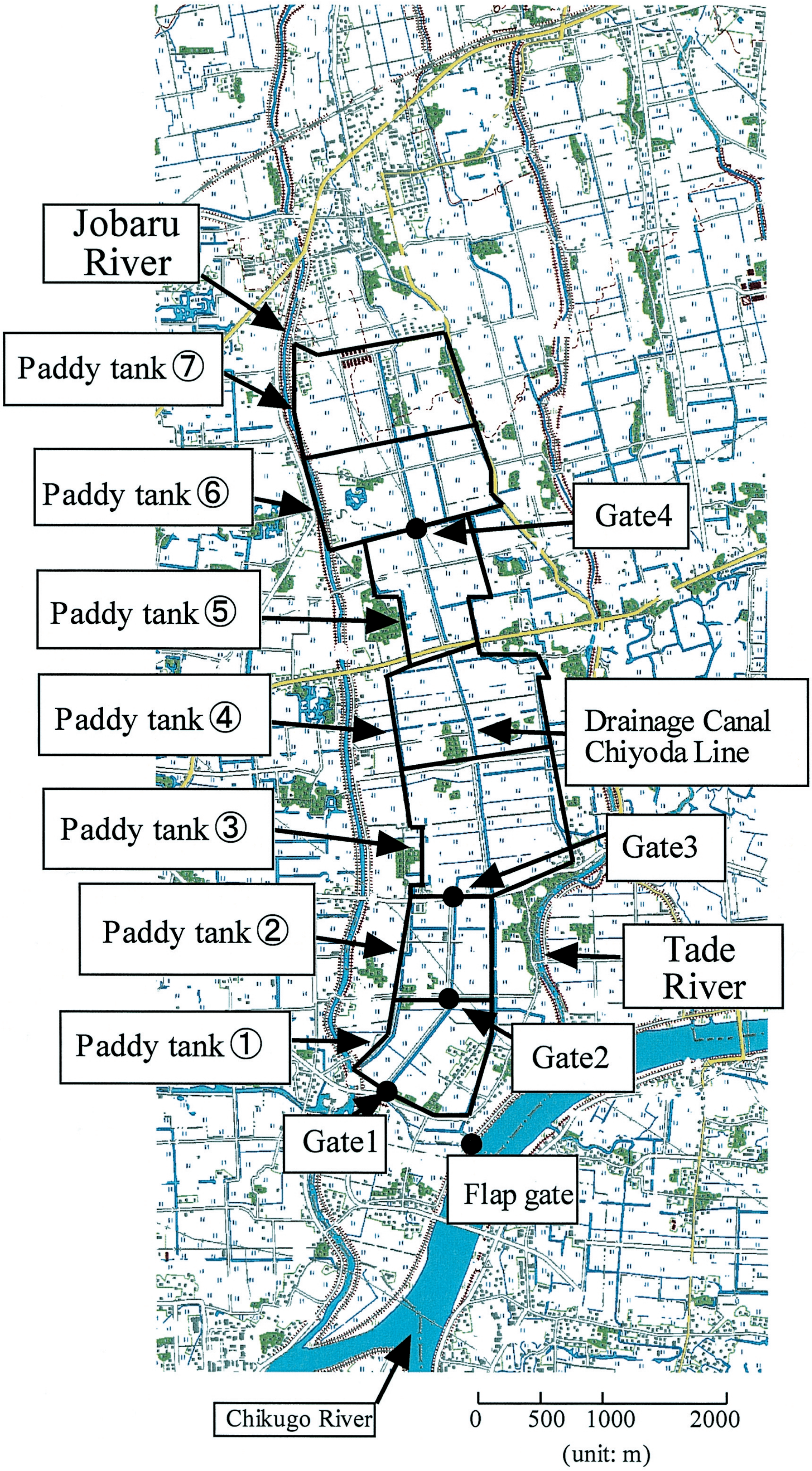

Fig. 1. Location of Chiyoda drainage basin.

discharges were calculated by the differences between water levels in the tanks, considering the storage effects in the paddy field and the drainage canal. Their model may be especially valid in extremely flat low–lying areas. Toyota (1977) proposed the "backwater–type storage model." His model structure was similar to that of Suzuki and Nishihata, except that the parameters were determined from observed data. Ueda (1972) divided a drainage basin into grids and conducted water–level analysis by regarding the grids as sequential storage tanks. His model was structurally similar to that of Suzuki and Nishihata. Hayase and Kadoya (1978) proposed a model similar to that of Ueda. However, Ueda calculated simultaneous nonlinear first–order differential equations to determine the water levels in tanks, whereas Hayase and Kadoya determined tank water levels by calculating simultaneous first–order equations resulting from approximating the flow equations between tanks with linear expressions using tank water levels. Shiraishi (1971), Shiraishi *et al.* (1972), and Toyota and Naruse (1974) interpreted the rainfall–runoff phenomenon in flat low–lying areas as hydraulic phenomenon of unsteady flow with lateral inflow, and proposed models based on unsteady flow equations.

Modeling of the drainage system in Chiyoda

The models for drainage and inundation analysis in flat low–lying paddy–cultivated areas are constructed based on the features, purposes and other related factors of the drainage basin. Considering the features and the purpose of this study, Ueda's model was selected for this study because of its simplicity and high performance. Ueda divided a drainage basin into grids, each of which was regarded as a storage tank. In the authors' model (Hiramatsu *et al.*, 2004), paddy fields were divided into "paddy tanks," and canals were divided into segments called "river tanks." Connecting the paddy tanks and the river tanks, the flow discharge and direction at each connection branch were calculated by hydraulic equations using the differences between water levels in the tanks at both ends of the branch.

By dividing the water areas of paddy fields and canals into paddy tanks and river tanks considering the topography and land use, the drainage network model of Chiyoda was obtained as depicted in **Fig. 2**. The basin was analyzed by this model in which the paddy fields were divided into 7 tanks and the main drainage canal was divided into 8 river tanks. Interfaces between paddy tanks and river tanks are drainage structures including 11 weirs, 4 gates, 3 open channels, and 1 flap gate at the downstream end. The longitudinal section of the drainage canal is depicted in **Fig. 3** in which 8 segments correspond with 8 river tanks. In order to meet the irrigation requirement during the irrigation time, weirs and gates are maintained at ordinary water level, which is the initial water level. In **Fig. 3**, for instance, in paddy tank No.1, 2.530 m is the average bottom eleva-

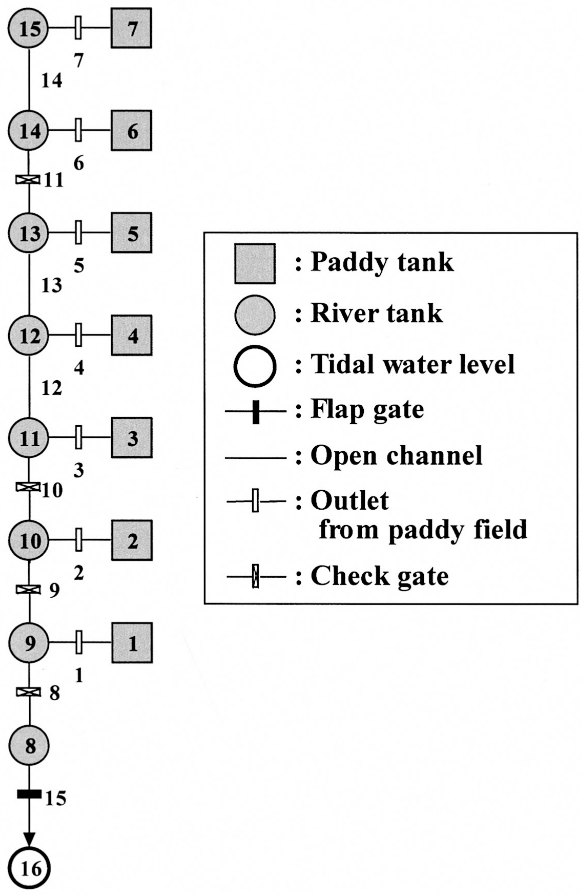

Fig. 2. Drainage network model of Chiyoda basin.

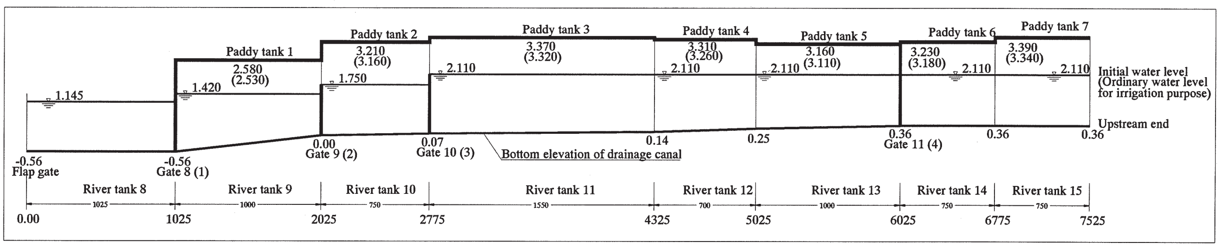

Fig. 3. Longitudinal section of Chiyoda drainage canal (unit: m).

Table 1. Dimensions of paddy tanks

Tank number	Initial water level (m)	Area (m^2)	Average bottom elevation (m)
1	2.580	643000	2.530
2	3.210	490000	3.160
3	3.370	1077000	3.320
4	3.310	921000	3.260
5	3.160	683000	3.110
6	3.230	999000	3.180
7	3.390	866000	3.340

Table 2. Dimensions of river tanks

Tank number	Initial water level (m)	Elevation of bottom at downstream end (m)	Area (m^2)	Length (m)	Bottom width (m)
8	1.145	-0.560	26650	1025	14
9	1.420	-0.560	26000	1000	14
10	1.750	0.000	19500	750	14
11	2.110	0.070	40300	1550	14
12	2.110	0.140	18200	700	14
13	2.110	0.250	26000	1000	14
14	2.110	0.360	19500	750	14
15	2.110	0.360	19500	750	14

Table 3. Dimensions of weirs

Weir number	Width (m)	Crest height (m)	Weir Number	Width (m)	Crest height (m)
1	64.3	2.580	7	86.6	3.390
2	49.0	3.210	8	5.75	1.420
3	107.7	3.370	9	4.0	1.750
4	92.7	3.310	10	5.0	2.110
5	68.3	3.160	11	10.0	2.110
6	99.9	3.230			

Table 4. Dimensions of open channels

Open channel number	Width (m)	Manning coefficient	Channel length (m)
1	14	0.020	1125
2	14	0.020	850
3	14	0.020	750

tion and 2.580 m is the elevation of crest at weir No. 1. The initial dimensions of paddy tanks, river tanks, open channels and weirs in **Fig. 2** were adopted in the modeling (**Tables 1, 2, 3** and **4**). The average area of a paddy tank was approximately 82 ha. The model is the combination of two processes, including the Runge–Kutta–Gill method to determine the water level and flow rate, and a genetic algorithm to optimize the gate operation.

Determination of water level and flow rate

This model was numerically solved to determine the water level in the paddy tanks and the river tanks, and the flow discharges at every connection branch.

Assuming that the water level in each tank is uniformly varied, the variation of the water level $Z_{(i)}$ in tank i is described by the following continuity equation, which is the basis equation for calculating the flood inundation.

$$\frac{dZ_{(i)}}{dt} = \frac{1}{A_{(i)}}(Q_{(i)\,in} - Q_{(i)\,\mathrm{out}}) + R_{(i)} \tag{1}$$

In equation (1), $Z_{(i)}$, $A_{(i)}$, $Q_{\mathrm{in}\,(i)}$, $Q_{\mathrm{out}\,(i)}$ and $R_{(i)}$ are water level, water surface area, inflow to tank No.i, outflow from tank No.i, and rainfall intensity. The drainage network model in Chiyoda basin had 15 tanks, resulting in the simultaneous first–order ordinary differential equations with 15 unknown variables. These equations were numerically calculated by the Runge–Kutta–Gill method with a time step size of 1 s. The flow rate Q at each connection branch was calculated by hydraulic equations using the differences between water levels in the tanks at both ends of the branch. Weirs or gates and open channels were assumed for hydraulic treatment of the connection branch as shown in **Fig. 2**.

In the case of a weir or gate, the flow rate Q at each connection branch was calculated by the following equations (**Fig. 4**):

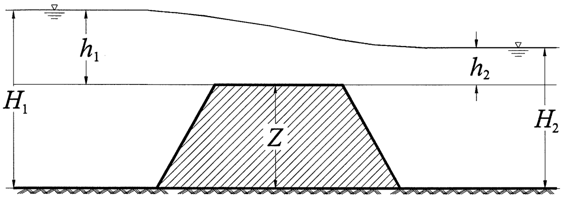

Fig. 4. Cross–section of weir.

(a) Submerged flow: $h_2 \geq 2h_1/3$

$$Q=4.0258Bh_2\sqrt{h_1-h_2} \quad (2)$$

(b) Critical flow: $h_2 < 2h_1/3$

$$Q=1.5495Bh_1^{3/2} \quad (3)$$

Here B is the width of weir; h_1 and h_2 represent the water depths in the upstream and downstream tanks above the crest Z of the weir; $h_1=H_1-Z$, $h_2=H_2-Z$.

In the case of an open channel, assuming that uniform flow and trapezoidal cross–section (**Fig. 5**), the flow rate Q was estimated by Manning's formula (Chow, 1986) and the water level gradient So between two river tanks:

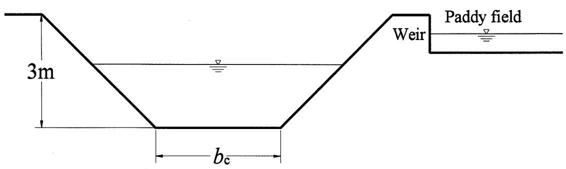

Fig. 5. Cross–section of open channel.

$$Q=\frac{1}{n}AR^{2/3}\sqrt{S_0} \quad (4)$$

Here A, R, S_0 and n represent the cross–sectional area, the hydraulic radius, the gradient and Manning's coefficient of roughness. These parameters are calculated by the following equations:

$$A=\frac{1}{2}\left\{b_c+(b_c+2\times\frac{h_1+h_2}{2}\times m)\right\}\times\frac{h_1+h_2}{2};$$

$$P=b_c+2\sqrt{1+m^2}\times\frac{h_1+h_2}{2};\ R=\frac{A}{P}$$

$$S_0=\frac{H_1-H_2}{L};\ h_1=H_1-z_1;\ h_2=H_2-z_2$$

where H_1, z_1, H_2, z_2 are the water levels and bottom elevations at the upstream and downstream of an open channel. L and m are the length and the side slope of an open channel.

In the case of a flap gate, the flow rate was determined by the following equations (**Fig. 6**):

(a) Full occupied: $H \geq d_a$

$$Q=yBd_a\sqrt{2g\delta} \quad (5)$$

If $x \geq 0.88$ then $y=1.02$; If $x<0.88$ then $y=1.02-2.42$ $(0.88-x)^3$,
here; $x=3\sqrt{H_e\delta}/D$; $H_e>D \Rightarrow H_e=D$

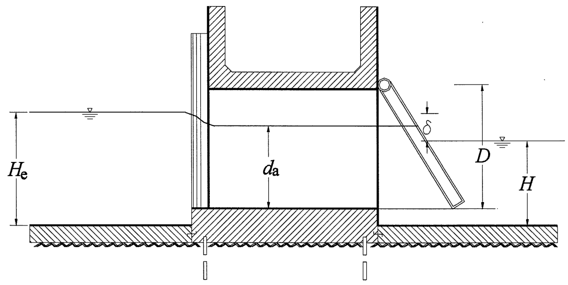

Fig. 6. Cross–section of flap gate.

(b) Sub–critical: $d_a>H>2H_e/3$

$$Q=yBH\sqrt{2g\delta} \quad (6)$$

If $x \geq 0.88$ then $y=0.98$; If $x<0.88$ then $y=0.98-3.41$ $(0.8-x)^3$, here $x=3\sqrt{H_e\delta}/D$; $H_e>D \Rightarrow H_e=D$

(c) Critical: $H<2H_e/3$

$$Q=1.7yBH_e^{3/2} \quad (7)$$

If $H_e>0.2D$ then $y=0.88 \sim 0.94$; If $H_e \leq 0.2D$ then
$y=0.98-3.41\ (0.8-x)^3 \leq 0.88$,
here $x=1.732H_e/D$.

B and D are the width and the height of the flap gate. H_e and H are the water head at upstream and downstream. δ is the different water head between upstream and downstream. Parameters of B and D are $B=5$ m, $D=3.5$ m.

Optimization of gate operation

In this study, a simple genetic algorithm (SGA) was applied to optimize the operation of 4 gates in the main drainage canal including gate No. 8 through gate No. 11. GA is an optimization technique modeled after the biological processes of natural selection and evolution. The GA operates on the population of decision variable sets. Through the application of three specialized genetic operators of selection, crossover and mutation, the GA population evolves toward an optimal solution. The GA has been used by Hiramatsu *et al.* (1998; 1999) to determine the optimal network structure of artificial neural network models for predicting the water–stage in tidal river and by Hiramatsu *et al.* (2000) to search for optimal model parameters of fish schooling of Japanese Medaka Fish. The outline of a simple genetic algorithm (SGA) is shown in **Fig. 7**.

A detailed description of the genetic operations used in GA to search for the optimal gate operation, which was under consideration for avoiding the local concentration of inundation in drainage basin, is as follows.

Step 1: Generation of an initial population

An initial population of coded strings representing the parameters is randomly generated population size $n_p=50$. This coded string of 24 binary bits $\{b_i, i=1, 24\}$ and four substring represent the value of d_8, d_9, d_{10} and d_{11}.

$$d_8=d_{min}+(d_{max}-d_{min})\times\frac{\sum_{i=1}^{6}b_i 2^{(i-1)}}{2^6-1} \quad (8)$$

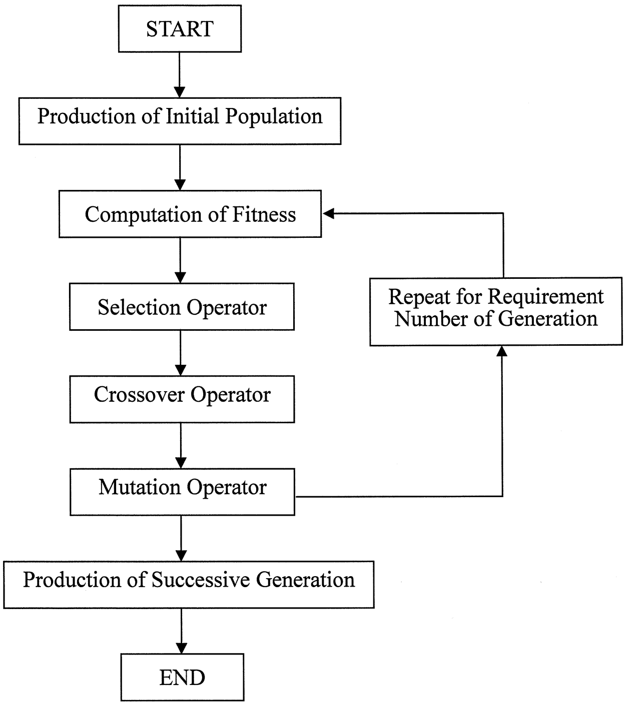

Fig. 7. Flow diagram of a simple genetic algorithm (SGA).

$$d_9 = d_{min} + (d_{max} - d_{min}) \times \frac{\sum_{i=1}^{6} b_{6+i} 2^{(i-1)}}{2^6 - 1} \quad (9)$$

$$d_{10} = d_{min} + (d_{max} - d_{min}) \times \frac{\sum_{i=1}^{6} b_{12+i} 2^{(i-1)}}{2^6 - 1} \quad (10)$$

$$d_{11} = d_{min} + (d_{max} - d_{min}) \times \frac{\sum_{i=1}^{6} b_{18+i} 2^{(i-1)}}{2^6 - 1} \quad (11)$$

where d_{min} and d_{max} are the minimum elevation (ordinary height −1.5 m) and the maximum elevation (ordinary height +1.5 m); b_i is the value of chromosome of each individual (0 or 1).

Step 2: Computation of the fitness

For each n_p string, a coded string is decoded into parameters of d_8, d_9, d_{10} and d_{11} by Eqs. (8) through (11). The fitness function f_t, which is used in the genetic operation, is defined by equation (12).

$$f_t = \frac{1}{|C_{v,p}| + |C_{v,c}| + \beta T} \quad (12)$$

In this equation, $C_{v,p}$ is the coefficient of variance of excess water depth in paddy tanks; $C_{v,c}$ is the coefficient of variance of excess water depth in river tanks. T is the total time of inundation in paddy fields and β (=0.01) is a weight for the inundation time comparing with the coefficients of variance. Here the excess water depth $h_{p(i)}$ in paddy tanks and the water level $h_{c(i)}$ from the top of the bank of river tanks were calculated by equations (13) and (14), respectively.

$$h_{p(i)} = H_{p(i)} - z_{(i)} \quad (13)$$

$$h_{c(i)} = H_{c(i)} - z_{(i)} - 3.5 \quad (14)$$

$H_{p(i)}$ and $H_{c(i)}$ are the water level in paddy tank No.i and the water level in river tank No.i and $z_{(i)}$ is the bottom elevation in the tank. The base height of the bank of river tanks was assumed h=3.5 m.

In equation (12), the inundation concentration is evaluated by the coefficient of variance (relative standard deviation) C_v=(standard deviation)/(mean). This factor is affected by the operation of drainage structures in the drainage system. It should be minimized in order to make the inundation distribute more equally; otherwise, some areas would be seriously inundated.

Step 3: Generation of a new population using the selection operator

New numbers of the next generation are determined by roulette wheel selection with an elite preservation strategy.

Step 4: The crossover operator

Single–point crossover is generally used in SGA. Hiramatsu *et al.* (1998), however, used uniform crossover because of its good convergence. The uniform crossover is then performed with the probability of a crossover p_c=0.5 for each pair of parent strings selected in step 4, which means that on average, 100 p_c% of pair undergo crossover.

Step 5: The mutation operator

Mutations are introduced into the population on a bit–by–bit basis at every generation to prevent the algorithm from converging to local optimum. The mutation is performed with the probably of mutation p_m=0.05 for each bit in the strings.

Step 6: Production of successive generations

Using steps 2 to 5 described above, a new generation is produced. Steps 2 to 5 are repeated until the number of generations exceeds the pre–assigned number of generation n_G=100.

Step 7: Optimal parameters

The string, which has maximum fitness in the final population obtained after n_G generation, is decoded into d_8^{max}, d_9^{max}, d_{10}^{max} and d_{11}^{max} by eqs. (15) through (18). Finally, the optimal crest elevation of four gates in the main drainage canal, including gate No. 8 through gate No. 11, are determined by the following equations:

$$\Delta_8 = d_8^{max} + H_{d8} \quad (15)$$

$$\Delta_9 = d_9^{max} + H_{d9} \quad (16)$$

$$\Delta_{10} = d_{10}^{max} + H_{d10} \quad (17)$$

$$\Delta_{11} = d_{11}^{max} + H_{d11} \quad (18)$$

where H_{d8}, H_{d9}, H_{d10}, H_{d11} are the crest elevation of gates at which the ordinary water level is maintained in order to meet the requirement of irrigation purpose.

The water level at downstream end

$$z_{(t)} = 1.94 + 0.8\cos\frac{2\pi(t+\theta)}{T} \quad (19)$$

where $Z_{(t)}$ is the water level, which is affected by variation of tide. Parameters of θ and T are θ=6 h, T=12.42 h

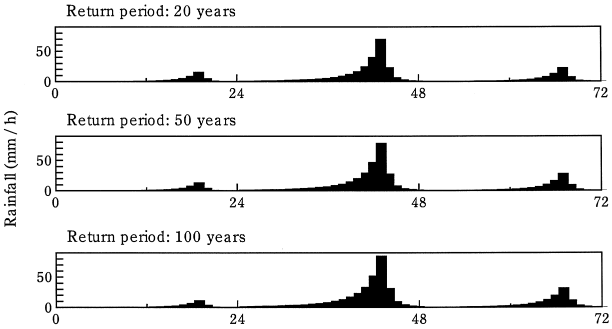

Fig. 8. Hourly rainfall time series with the return periods of 20, 50 and 100 years.

Rainfall data

The simulation process was carried out using the rainfall with the return periods of 20, 50 and 100 years. These rainfall values were statistically calculated from the observed data of the Saga Local Meteorological Observatory in 90 years by the Iwai method. The total calculation time of simulation is 6 days, including 3 days with rainfall and the following 3 days without rainfall. A time series with three–day continuous rainfall at the regular interval of 1 hour was depicted in **Fig. 8**. The maximum of hour rainfall amounts in the rainfall time series with the return periods of 20, 50,100 were 70.1 mm, 78.7 mm and 84.9 mm, respectively. Evapotranspiration was not considered in the simulations. The vertical percolation was also not incorporated in the model because subsurface drainage due to percolation does not contribute to the removal of excess water during flood events in the study area.

RESULTS AND DISCUSSION

Applicability of the model

The result of maximum water depth, the inundation time, the coefficient of variance in the paddy tanks, the maximum water level from the top of the bank of river tanks, and the coefficient of variance in the river tanks with the return periods of 20, 50 and 100 years are presented in **Tables 5**, **6** and **7**, respectively. It is assumed that the inundation in the paddy tank occurs as the water level in the paddy tank is 0.3 m higher than the weir crest. The variations of water depth in paddy tanks and river tanks under the rainfall with the return period of 100 years with and without the optimization are

Table 5. Results of calculation using the rainfall with the return period of 20 years

Tank No.	Paddy tank				Tank No.	River tank	
	Without optimization		Optimization			Without optimization	Optimization
	Max. water depth (m)	Inundation time (hr)	Max. water depth (m)	Inundation time (hr)		Max water level (m)	Max. Water level (m)
1	0.312	2.084	0.306	0.890	1	−0.167	−0.148
2	0.196	0.000	0.196	0.000	2	−0.098	−0.104
3	0.196	0.000	0.196	0.000	3	−0.169	−0.286
4	0.212	0.000	0.196	0.000	4	−0.099	−0.188
5	0.362	11.673	0.277	0.000	5	−0.168	−0.254
6	0.299	0.000	0.290	0.000	6	−0.278	−0.363
7	0.196	0.000	0.196	0.000	7	−0.381	−0.390
Total		13.758		0.890	8	−0.380	−0.389
$\|C_v\|$	0.254		0.202		$\|C_v\|$	0.494	0.392

Table 6. Results of calculation using the rainfall with the return period of 50 years

Tank No.	Paddy tank				Tank No.	River tank	
	Without optimization		Optimization			Without optimization	Optimization
	Max. water depth (m)	Inundation time (hr)	Max. water depth (m)	Inundation time (hr)		Max water level (m)	Max. Water level (m)
1	0.339	4.746	0.357	3.057	1	−0.162	−0.130
2	0.218	0.000	0.215	0.000	2	−0.071	−0.054
3	0.215	0.000	0.215	0.000	3	−0.133	−0.250
4	0.257	0.000	0.215	0.000	4	−0.055	−0.172
5	0.407	26.897	0.294	0.000	5	−0.123	−0.237
6	0.345	8.689	0.345	7.297	6	−0.233	−0.346
7	0.215	0.000	0.215	0.000	7	−0.335	−0.335
Total		40.332		10.354	8	−0.335	−0.334
$\|C_v\|$	0.254		0.229		$\|C_v\|$	0.568	0.431

Table 7. Results of calculation using the rainfall with the return period of 100 years

Tank No.	Paddy tank: Without optimization: Max. water depth (m)	Paddy tank: Without optimization: Inundation time (hr)	Paddy tank: Optimization: Max. water depth (m)	Paddy tank: Optimization: Inundation time (hr)	Tank No.	River tank: Without optimization: Max water level (m)	River tank: Optimization: Max. Water level (m)
1	0.360	8.992	0.382	3.822	1	−0.158	−0.120
2	0.240	0.000	0.228	0.000	2	−0.050	−0.028
3	0.228	0.000	0.228	0.000	3	−0.107	−0.219
4	0.288	0.000	0.228	0.000	4	−0.024	−0.111
5	0.439	36.649	0.354	4.599	5	−0.092	−0.177
6	0.376	18.938	0.352	6.803	6	−0.201	−0.286
7	0.228	0.000	0.228	0.000	7	−0.304	−0.328
Total		64.579		15.224	8	−0.304	−0.328
$\lvert C_v \rvert$	0.252		0.235		$\lvert C_v \rvert$	0.651	0.516

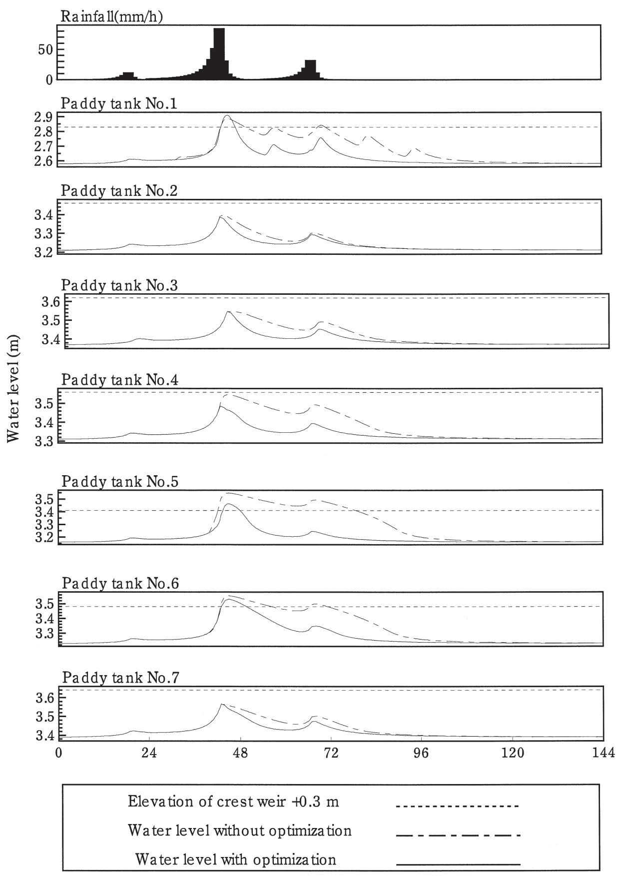

Fig. 9. Variation of water level at paddy tanks.

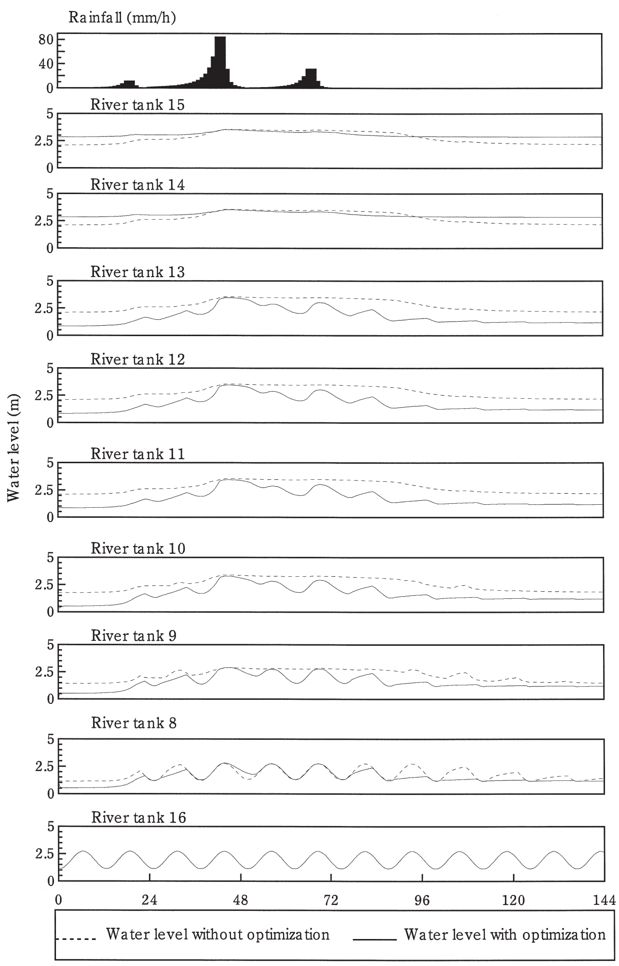

Fig. 10. Variation of water level at river tanks.

depicted in **Figs. 9** and **10**. The simulation results indicate that before the optimization of a gate operation, the inundation concentrates at paddy tanks No. 5 and No. 6 with a prolonged inundation time (**Fig. 9**). The total inundation time increases up to 13.8 hours, 40.3 hours, and 64.6 hours, and the coefficient of variance stands at 0.255, 0.254 and 0.252, corresponding with the rainfall with the return periods of 20, 50 and 100 years. The same situation occurs in river tanks and the coefficient of variance stands at 0.494, 0.568 and 0.651. This local concentration of inundation, which damages the crop yield especially in downstream areas, should be avoided by the optimization of the operation of a drainage system. In fact, four gates, including gate No. 8 through gate No. 11, play an important role in the drainage system. By optimizing the operation of these gates, the above issue can be solved. The results of calculation under the rainfall with the return periods of 20, 50 and

Table 8. The ordinary crest heights and the optimal operations of weirs with the return periods of 20, 50 and 100 years

Gate No.	8	9	10	11
Ordinary crest height (m)	1.420	1.750	2.110	2.110
Optimal operation (m) (T=20 years)	–0.08	0.869	1.229	2.943
Optimal operation (m) (T=50 years)	–0.08	0.393	0.610	2.991
Optimal operation (m) (T=100 years)	–0.08	0.345	0.848	2.848

100 years show that the total inundation time in the paddy tanks decreases to 0.9 hour, 10.4 hours, and 15.2 hours, and the coefficients of variance of paddy tanks and river tanks also decrease to 0.202 and 0.229, 0.235 and 0.392, and 0.431 and 0.516, respectively, which means that the inundation distributes more equally. The optimal heights of weirs are presented in **Table 8**. The ordinary crest height of gates 8 through 11 are 1.42 m, 1.75 m, 2.11 m and 2.11 m, respectively, in which the ordinary water level is maintained in order to meet the requirement of irrigation purposes. However, in order to avoid the local concentration of inundation when the food events occur, the crest height of gates 8, 9 and 10 should be lowered; the crest height of gate 11 should be raised.

Conclusion

The tank model is a very useful tool to simulate the water flow in flat low–lying agricultural areas. In Chiyoda basin, it was applied to determine the water levels in the paddy field and in the main drainage canal, and the flow discharges at the main drainage canal and connection structures. Here, the Runge–Kutta–Gill method with a time step of 1s was used to compute the model. By optimizing the gate operation, the most important point here is that we can evaluate and minimize the inundation concentration in the entire drainage basin. In order to meet this purpose, the SGA was used to optimize the crest height of weirs in the drainage canal. Actually, the inundation concentration was quantified by the coefficients of variance and inundation time. These values were included in the fitness function. By the genetic operators of decoding, calculation of fitness function, selection with elite preservation strategy, and uniform crossover and mutation, the inundation concentration and inundation time were optimally minimized. Finally, the optimal heights of weirs were determined in the range from the minimum elevation (ordinary height −1.5 m) to the maximum elevation (ordinary height +1.5 m). For further study, the information should be collected by field surveys such as accurate drainage network and topographic data.

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