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Drainage and Inundation Analysis in a Flat Low–lying Paddy–cultivated Area of the Red River Delta, Viet Nam

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The Red River Delta (RRD) is located in the northern Viet Nam. The central part of the RRD is markedly flat, and 56% of the central part is less than 2 m above sea level. In the flat low–lying area of the RRD, many pumping stations for irrigation and drainage have been constructed as a result of the government's adoption of the Doi Moi (renovation) policy in 1986. Most of the excess water in the rainy season is now removed by pumped drainage. However, inundation still occurs in paddy fields, damaging summer rice cultivated in the rainy season. This paper presents a mathematical model of drainage system for calculating inundation and excess water removal during the rainy season in Phu Lam Commune, which is a typical flat low–lying paddy–cultivated area in the RRD. This model was developed from a flat low–lying area tank model to identify potential problems in the further development of the drainage system. The present situation of excess water removal and inundation was then evaluated by numerical simulations using the model. The results indicated that the inundation occurred in the paddy fields apart from the pumping stations. Also considered was the low flowing capacity of drainage facilities between the first and the second canals, resulting from inadequate maintenance of drainage facilities such as sluggish, weedy canals and culverts; complicated drainage networks; and extremely flat topography. The results also suggested that in the further study the following information should be collected through field surveys, including interviews with farmers: accurate drainage network and topographic data, accurate capacity and arrangement of pumping machines, actual operation of pumping machines, rainfall time series data at one–hour intervals or shorter, and actual situations of inundation during past floods.

INTRODUCTION

The Red River Delta (RRD) is located in the northern Viet Nam and covers the area enclosed by the northernmost and the southernmost distributaries of the Red River (Nguyen *et al.*, 2001). The RRD has a total area of 14,789 km^2; 58% is used for agriculture, and 45% for paddy field (MARD, 2002). The central part of the RRD is markedly flat, with an elevation of 0.4 to 12 m above sea level, and 56% of the central part is less than 2 m above sea level. During the rainy season in the flat low–lying area, much of the excess rainwater is removed by pumped drainage, although in some parts it is removed by

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hand-operated waterproof scoops and gravitational drainage (Hai and Egashira, 2002).

In Viet Nam, the market-oriented economy has developed with the effect of the government's adoption of the Doi Moi (renovation) policy in 1986. Renovation in the agricultural sector was politically initiated in 1988. After that, the agricultural production system shifted from cooperative-based to farm-household-based. Before the start of the renovation, cultivated land was arranged in large parcels suitable for the operation of farm machines and instruments. The drainage networks and the drainage facilities in the flat low-lying agricultural areas of the RRD were designed and constructed in those days. After the renovation, cultivated land was allocated to farmers, with each parcel divided into many small plots. Many pumping stations have been also constructed in the flat low-lying areas since the renovation. However, inundation and submergence still occurs due to complicated drainage networks, inadequate maintenance and low flowing capacity of drainage facilities, insufficient capacity of pumping stations, and extremely flat topography. As a result, summer rice cultivated in the rainy season is damaged (Hai and Egashira, 2002).

To identify potential problems in the further development of the drainage system, accurate and spatial consideration must be given to the present situation of excess water removal and inundation during the rainy season in the flat low-lying areas in the RRD. In order to execute a study aimed at this purpose, it is necessary to clarify which information should be collected early. This paper presents a mathematical model of drainage system in the rainy season in Phu Lam Commune, which is located in the lowest part of the RRD and is a typical flat low-lying paddy-cultivated area in the RRD. This model was based on a flat low-lying area tank model (Shikasho and Tanaka, 1985). The present methods of excess water removal and inundation were then evaluated by numerical simulations using the flat low-lying area tank model. Finally, information necessary for further study of the flat low-lying areas in the RRD was discussed, based on the results of the modeling and the simulations.

Drought damage has also occurred in the RRD during the dry season due to an inadequate irrigation system, and most of the irrigation and drainage systems in the RRD are dual-purpose canal systems. However, this paper discusses only the present excess water removal and inundation during the rainy season, from the standpoint of the drainage system's characteristics.

MATERIALS AND METHODS

Study area

Phu Lam Commune, 21 km northeast of Hanoi City, depicted in Fig. 1, is located in Tien Du District in Bac Ninh Province in the northern part of the RRD. It is surrounded by the Cau River to the northeast and the Duong River to the south and west. Over half of the cultivated area in Tien Du District is flat and low-lying. Tien Du District has a long history of agricultural production. However, since Phu Lam Commune is located in the lowest part of the RRD, it is frequently inundated by excess water in the rainy season, resulting in the lower yield of summer rice there than in other areas in Tien Du District (Hai and Egashira, 2002).

The irrigation and drainage systems in Tien Du District have been constructed since

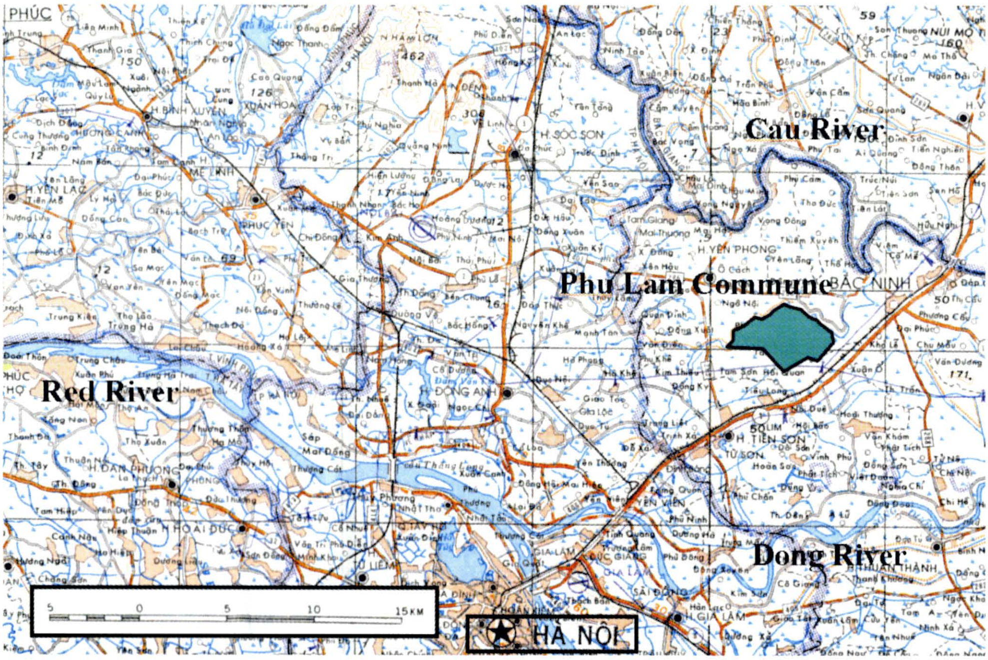

Fig. 1. Location of Phu Lam Commune situated at 21 km northeast of Hanoi City in Tien Du District.

the 1960s. The three levels of different administrative management are large–scale, medium–scale, and small–scale. In the large–scale management, main pumping stations and the systems of first canals to supply or drain water are administered at the district level by the Irrigation and Drainage Management Company (IDMC). In the medium–scale management, pumping stations and the systems of the second canals are managed at the commune level by the IDMC. In the small–level management, small pumping stations and the systems of the third canals are managed at the commune or village level (Hai and Egashira, 2002).

Phu Lam Commune has a drainage area of 13.5 km^2. About 75% of the area is used for paddy fields, the second and the third canals, and ponds; about 2% for the first canals; and about 23% for other land use such as residential areas, public facilities and roads, as Fig. 2 depicts. At Phu Lam Commune, most of the paddy fields were inundated in the rainy season before 1985, resulting in one rice cultivation per year. After that, the construction of pumping stations permitted the area to cultivate rice twice a year. From the field survey carried out in 1999 (Hai and Egashira, 2002), excess water from 74.2% of the drainage area was removed by pumped drainage and 14.2% by hand–operated waterproof scoops. In 11.6% of the drainage area, excess water could hardly be removed during the rainy season because of gravitational drainage. The pumped drainage capacity in Phu Lam Commune was approximately 16,000 m^3 h^{-1} with the operation of 16 pumping machines.

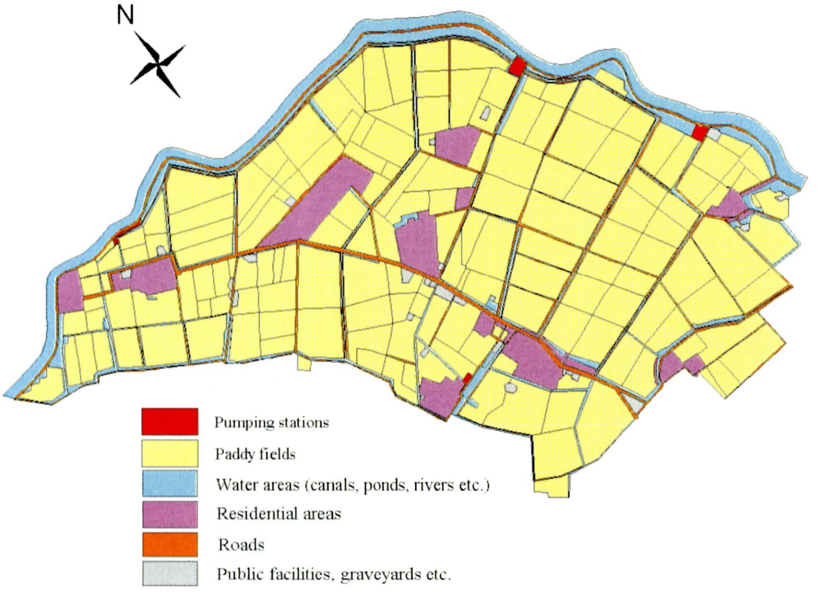

Fig. 2. Land use map in Phu Lam Commune. About 75% of the area is used for paddy fields, the second and the third canals, and ponds; about 2% for the first canals; and about 23% for other land uses.

Figure 3 depicts the general features of the canal and the paddy field during the rainy season in Phu Lam Commune together with the ordinary water level. The dimensions in Fig. 3 are based on the authors' field survey executed in September 2003. The elevations are based on the datum line on a road. The first canal (Fig. 4) was connected to the sec-

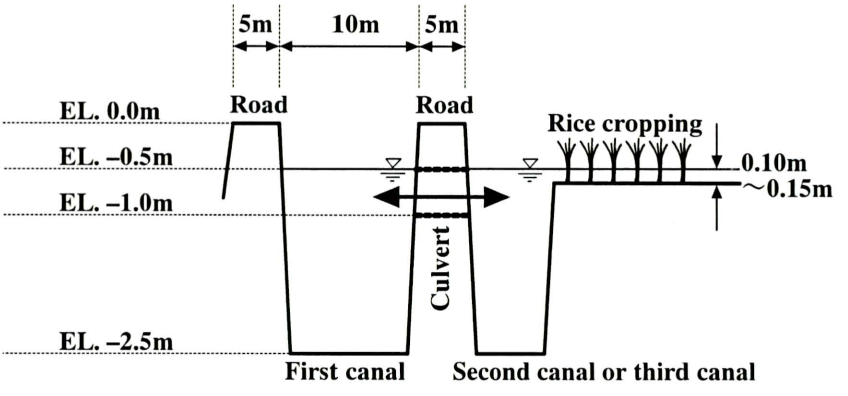

Fig. 3. General features of the canal and paddy field together with the ordinary water level in the rainy season in Phu Lam Commune. The elevations are based on the datum line on a road.

Fig. 4. A photograph of the first canal in the rainy season in Phu Lam Commune.

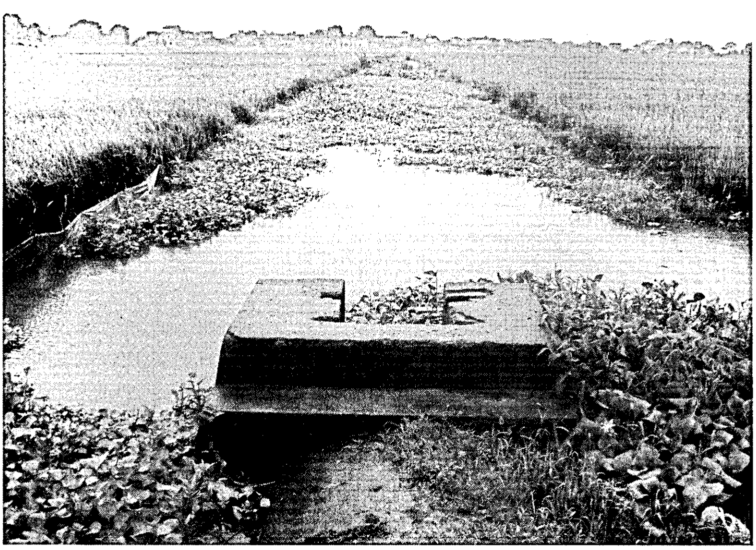

Fig. 5. A photograph of the second canal and the concrete box culvert in the rainy season in Phu Lam Commune.

ond canal or the third canal by a concrete box culvert, as shown in Fig. 5. There was no levee or dike around a plot of paddy field, and a plot of paddy field and the second and/or the third canals around the plot existed in a ponding water area.

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 inundation area and depth in time and space.

Because short–term inundation, not resulting 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 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 Phu Lam Commune

Considering the features of the above–mentioned models 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, paddy fields and ponds were divided into "paddy tanks," and canals were divided into segments called "river tanks." Connecting the paddy tanks and

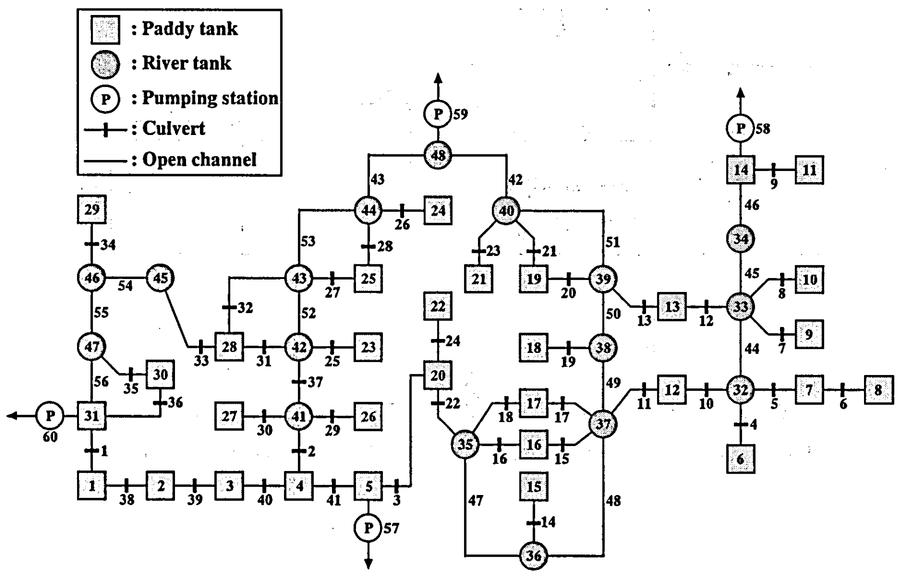

Fig. 6. Drainage network model of Phu Lam Commune obtained by dividing the water areas of paddy fields, ponds, and canals into paddy tanks and river tanks.

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, ponds, and canals into paddy tanks and river tanks considering the topography and land use in Fig. 2, the drainage network model of Phu Lam Commune was obtained, as depicted in Fig. 6. The drainage network model had 31 paddy tanks, 17 river tanks, and 4 pumping stations. During the rainy season in Phu Lam Commune, a plot of paddy field and the second and/or the third canals around the plot existed in the ponding water area; therefore, the second and/or the third canals were included in neighboring paddy tanks. The average area of a paddy tank was approximately 25 ha. From the field survey carried out in 1999 (Hai and Egashira, 2002), hand–operated waterproof scoop and gravitational drainage were used together with pumped drainage. However, because of the lack of detailed information about where hand–operated waterproof scoop and gravitational drainage were used in Phu Lam Commune, the excess water was assumed to be removed by pumped drainage through the four pumping stations.

Assuming that the water level in each tank is uniformly varied, the variation of the water level H_i in tank i is described by the following continuity equation:

$$A_i \frac{dH_i}{dt} = R + Q\,. \tag{1}$$

Here A_i represents the area of the tank i; R, the rainfall intensity; Q, the inflow rate from

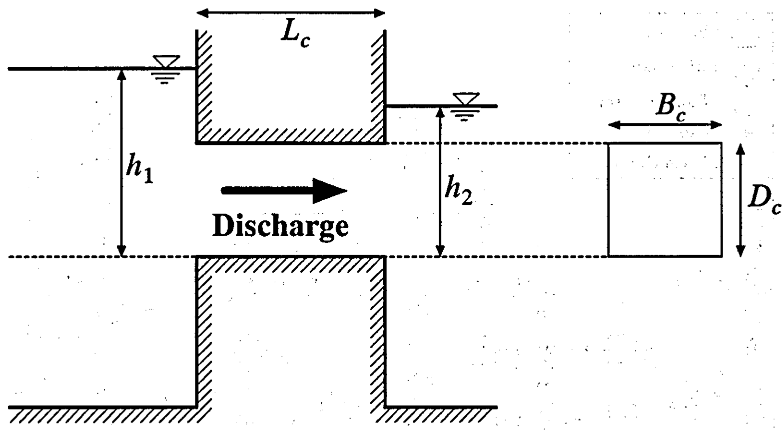

Fig. 7. Sketch of the box culvert at the connection branch between two tanks.

neighboring tanks. The inflow rate Q has a negative value in the case of outflow to neighboring tanks. The drainage network model in Phu Lam Commune had 48 tanks, resulting in the simultaneous first-order ordinary differential equations with 48 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. Box culvert and open channel were assumed for hydraulic treatment of the connection branch, as shown in Fig. 6.

In the case of a box culvert, the flow rate Q was calculated by the following equations (Fig. 7):

(a) Submerged flow: $h_1 > h_2 \geq D_c$

$$Q = B_c D_c \sqrt{\frac{2g(h_1 - h_2)}{1 + 0.5 + \dfrac{2gL_c n^2}{R_c^{4/3}}}}, \quad R_c = \frac{B_c D_c}{2(B_c + D_c)}, \tag{2}$$

(b) Critical flow: $h_1 \geq D_c > h_2$

$$\left.\begin{array}{l} Q = \mu_1 \times \dfrac{2}{3} B_c \sqrt{2g}\left\{(h_1 - h_2)^{3/2} - (h_1 - D_c)^{3/2}\right\} + B_c h_2 \sqrt{\dfrac{2g(h_1 - h_2)}{1 + 0.5 + \dfrac{2gL_c n^2}{R_c^{4/3}}}} \\ R_c = \dfrac{B_c h_2}{2(B_c + h_2)} \end{array}\right\}, \tag{3}$$

(c) Subcritical flow: $D_c > h_1 > h_2$

$$Q=\mu_2 \times \frac{2}{3} B_c \sqrt{2g}\,(h_1-h_2)^{3/2} + B_c h_2 \sqrt{\frac{2g(h_1-h_2)}{1+0.5+\dfrac{2gL_c n^2}{R_c^{4/3}}}}\,, \quad R_c=\frac{B_c h_2}{2(B_c+h_2)}\,. \quad (4)$$

Here h_1 and h_2 represent the water depths in the upstream and downstream tanks above the bottom of the culvert; L_c, B_c and D_c, the length, width and height of the culvert; n, the Manning's coefficient of roughness; R_c, the hydraulic radius; μ_1 and μ_2, the coefficients of discharge. A value of 0.62 was assumed for μ_1 and μ_2 in this study.

In the case of an open channel, assuming non–uniform flow and rectangular cross–section, the flow rate Q was estimated by Manning's formula using the water level gradient between two river tanks:

$$Q=\frac{1}{n}\,\frac{h_1+h_2}{2}\,B_o h_c^{2/3}\,\frac{H_1-H_2}{\sqrt{L_o\,|H_1-H_2|}}\,, \quad R_c=\frac{B_o}{B_o+h_1+h_2}\,\frac{h_1+h_2}{2}\,. \qquad (5)$$

Here H_1 and H_2 represent the water levels in the upstream and downstream tanks; B_o and L_o, the width and length of the open channel.

Because the detailed information about the basin topography and the dimensions of drainage facilities in Phu Lam Commune has not been obtained up to now, those values necessary for calculations using the flat low–lying area tank model were approximately estimated, with the assumption that the Phu Lam Commune was completely flat. The basin topography and the dimensions of drainage facilities in Fig. 3 were approximately adopted in the modeling (Tables 1, 2, 3 and 4). A relatively large value of 0.07 $m^{-1/3}s$ was assumed for the Manning's coefficient of roughness n because of sluggish and weedy canals, as shown in Fig. 5 (Chow, 1986). An n–value of 0.035 $m^{-1/3}s$ was assumed for cul-

Table 1. The areas of the paddy and river tanks (see also Fig. 6).

Tank number	Tank type	Area (m^2)	Tank number	Tank type	Area (m^2)	Tank number	Tank type	Area (m^2)
1	Paddy	565,440	17	Paddy	80,000	33	River	5,600
2	Paddy	248,600	18	Paddy	138,000	34	River	3,000
3	Paddy	365,750	19	Paddy	536,800	35	River	6,800
4	Paddy	295,460	20	Paddy	273,900	36	River	3,200
5	Paddy	723,690	21	Paddy	421,960	37	River	7,000
6	Paddy	612,370	22	Paddy	51,400	38	River	2,800
7	Paddy	282,040	23	Paddy	249,700	39	River	8,400
8	Paddy	117,590	24	Paddy	286,900	40	River	11,000
9	Paddy	217,140	25	Paddy	165,700	41	River	3,500
10	Paddy	261,250	26	Paddy	44,000	42	River	5,500
11	Paddy	291,850	27	Paddy	103,620	43	River	7,800
12	Paddy	413,820	28	Paddy	522,060	44	River	7,900
13	Paddy	361,800	29	Paddy	367,300	45	River	8,800
14	Paddy	240,800	30	Paddy	310,200	46	River	8,000
15	Paddy	85,800	31	Paddy	155,320	47	River	6,500
16	Paddy	97,200	32	River	8,500	48	River	10,000

Table 2. The dimensions of the box culverts. The elevations are based on the datum line on a road in Fig. 7 (see also Figs. 6 and 7).

Branch number	Bottom elevation (m)	Height (m)	Width (m)	Length (m)	Manning's n ($m^{-1/3}s$)	Number of culvert
1	−1.5	1.0	1.0	5.0	0.035	1
2	−1.5	1.0	1.0	5.0	0.035	1
3	−1.5	1.0	1.0	5.0	0.035	1
4	−1.5	1.0	1.0	5.0	0.035	1
5	−1.0	0.5	0.5	5.0	0.035	2
6	−1.0	0.5	0.5	5.0	0.035	1
7	−1.0	0.5	0.5	5.0	0.035	1
8	−1.0	0.5	0.5	5.0	0.035	1
9	−1.0	0.5	0.5	5.0	0.035	2
10	−1.0	0.5	0.5	5.0	0.035	2
11	−1.0	0.5	0.5	5.0	0.035	2
12	−1.0	0.5	0.5	5.0	0.035	2
13	−1.0	0.5	0.5	5.0	0.035	2
14	−1.0	0.5	0.5	5.0	0.035	1
15	−1.0	0.5	0.5	5.0	0.035	1
16	−1.0	0.5	0.5	5.0	0.035	1
17	−1.0	0.5	0.5	5.0	0.035	1
18	−1.0	0.5	0.5	5.0	0.035	1
19	−1.0	0.5	0.5	5.0	0.035	1
20	−1.0	0.5	0.5	5.0	0.035	2
21	−1.0	0.5	0.5	5.0	0.035	1
22	−1.0	0.5	0.5	5.0	0.035	2
23	−1.0	0.5	0.5	5.0	0.035	2
24	−1.0	0.5	0.5	5.0	0.035	1
25	−1.0	0.5	0.5	5.0	0.035	2
26	−1.0	0.5	0.5	5.0	0.035	1
27	−1.0	0.5	0.5	5.0	0.035	1
28	−1.0	0.5	0.5	5.0	0.035	1
29	−1.0	0.5	0.5	5.0	0.035	1
30	−1.0	0.5	0.5	5.0	0.035	1
31	−1.0	0.5	0.5	5.0	0.035	2
32	−1.0	0.5	0.5	5.0	0.035	2
33	−1.0	0.5	0.5	5.0	0.035	1
34	−1.0	0.5	0.5	5.0	0.035	2
35	−1.0	0.5	0.5	5.0	0.035	1
36	−1.0	0.5	0.5	5.0	0.035	1
37	−1.5	1.0	1.0	5.0	0.035	1
38	−1.0	0.5	0.5	5.0	0.035	2
39	−1.0	0.5	0.5	5.0	0.035	2
40	−1.0	0.5	0.5	5.0	0.035	2
41	−1.0	0.5	0.5	5.0	0.035	2

verts. The total drainage capacity of 16,000 m^3 h^{-1} by 16 pumping machines was allocated to the four pumping stations in proportion to an approximate drainage area of each pumping station. The pumping stations in Phu Lam Commune were considered to be manually

Table 3. The dimensions of the open channels. The elevations are based on the datum line on a road in Fig. 7 (see also Figs. 6 and 7).

Branch number	Width (m)	Length (m)	Bottom elevation (m)	Manning's n ($m^{-1/3}s$)
42	15	800	–2.5	0.035
43	15	645	–2.5	0.035
44	10	705	–2.5	0.035
45	10	430	–2.5	0.035
46	10	300	–2.5	0.035
47	10	500	–2.5	0.035
48	10	510	–2.5	0.035
49	10	490	–2.5	0.035
50	10	560	–2.5	0.035
51	10	970	–2.5	0.035
52	10	665	–2.5	0.035
53	10	785	–2.5	0.035
54	10	840	–2.5	0.035
55	10	725	–2.5	0.035
56	10	650	–2.5	0.035

Table 4. The capacities of the pumping stations (see also Fig. 6).

Branch number	Number of machines	Total capacity ($m^3 h^{-1}$)
57	3	3,000
58	5	5,000
59	6	6,000
60	2	2,000

operated; insensitive control was therefore assumed for all pumping stations. In insensitive control, by continuously monitoring the water level in the nearest neighbor tank to each pumping station, the pumping station was made to start operating when the water level rose and reached −0.45 m, and to stop operating when the water level dropped to −0.5 m. A water level of −0.5 m was given to all tanks at the beginning of each calculation.

Rainfall data

Tien Du District belongs to the subtropical monsoon climate with two distinct seasons: the dry season from November to March, and the rainy season from April to October (Nguyen *et al.*, 2001; Hai and Egashira, 2002). Based on these climatic characteristics and this study's focus on the approximate estimation of current drainage and inundation in Phu Lam Commune during the rainy season, two patterns of monthly rainfall time series in Fig. 8 were used for numerical simulations. A virtual time series with a one–hour continuous rainfall at regular intervals, 5 mm h^{-1} from 16:00 p.m. to 17:00

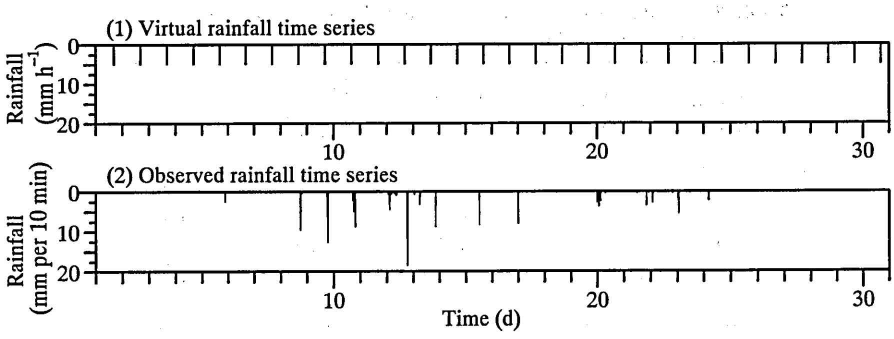

Fig. 8. Monthly rainfall time series used for the numerical simulations: (1) the virtual time series with a one-hour continuous rainfall at regular intervals and (2) the actual time series observed about 50 km northwest of Hanoi City.

p.m. every day, was assumed for the monthly rainfall, as shown in Fig. 8 (1), in order to estimate the approximate flow direction in an ordinary drainage condition. The total monthly rainfall is 150 mm. The actual rainfall time series, depicted in Fig. 8 (2), was observed about 50 km northwest of Hanoi City at ten-minute intervals in July 2000 (Kurosawa *et al.*, 2004) and was used for evaluating the flow direction and discharge, as well as the inundation duration in storm drainage conditions. The observed rainfall time series revealed that the total monthly rainfall of 308 mm was almost the same as the average monthly rainfall in July in Hanoi City (Nguyen *et al.*, 2001; Hai and Egashira, 2002). The maximums of ten-minute, hourly, and daily rainfall amounts in the observed rainfall time series were 18.5 mm, 49.8 mm and 83.1 mm, respectively.

Evapotranspiration was not considered in the simulations. The vertical percolation was not incorporated in the model because subsurface drainage due to percolation does not contribute to the removal of excess water during the rainy season in the study area.

RESULTS AND DISCUSSION

Applicability of the model

Figure 9 depicts the drainage discharges at the four pumping stations and the variation of the water level in the nearest neighbor tank to each pumping station. Figure 9 reveals that each pumping station started to operate when the water level in the nearest neighbor tank rose to −0.45 m and stopped operating when the water level dropped to −0.5 m. Since the excess water was assumed to be removed only by pumped drainage through the four pumping stations in the study area, there was no water movement in the whole drainage area when none of the four pumping stations were not in operation. After one or more pumping stations started to operate, the excess water started to move toward the pumping stations. Figure 10 depicts the flow directions at every connection branch obtained by the numerical simulation using the virtual rainfall time series in Fig. 8 (1). Figure 10 shows the flow directions when all of the pumping stations were in operation. The whole drainage area could be divided into four drainage sub-areas, as shown in Fig. 10.

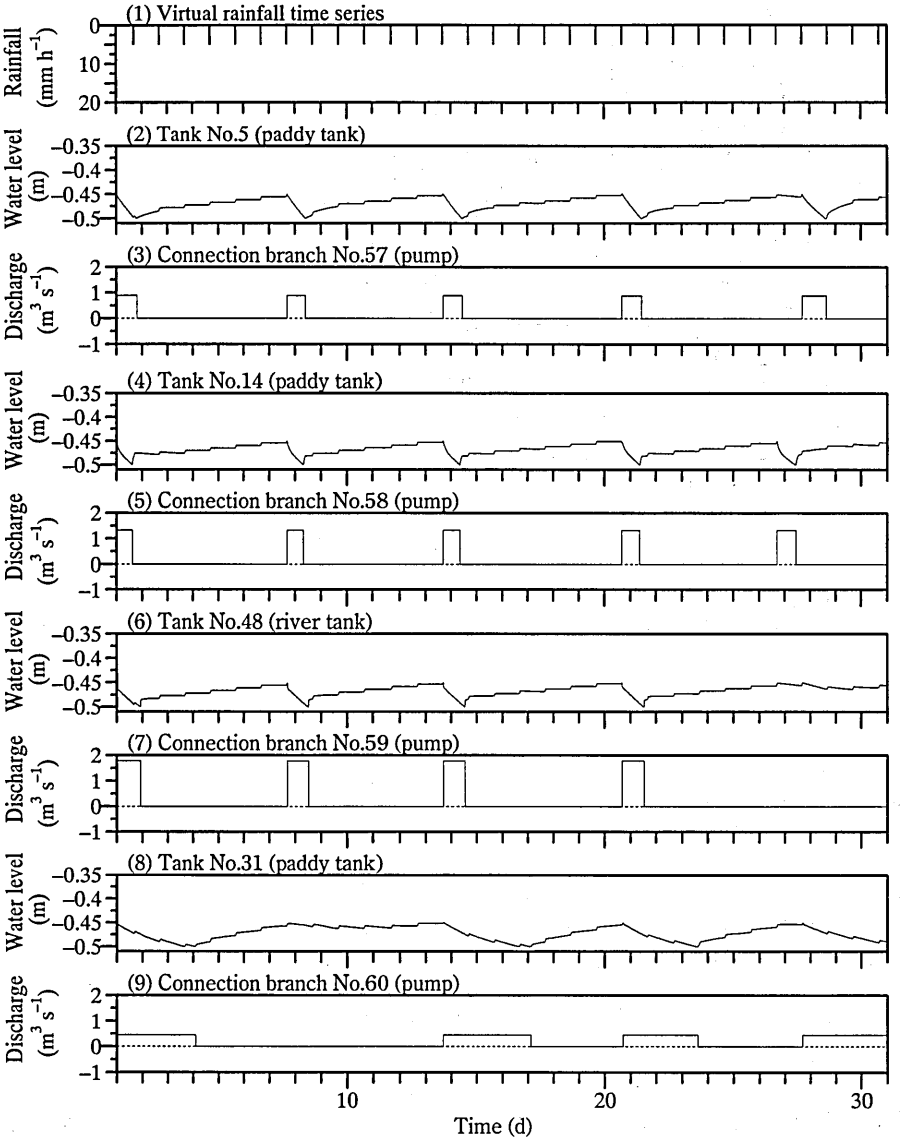

Fig. 9. Variation of the drainage discharges at the four pumping stations and the water levels in the nearest neighbor tanks to the pumping stations (see also Fig. 6).

The flow directions obtained by the simulation using the observed rainfall in Fig. 8 (2) were identical to those using the virtual rainfall. In the simulation using the observed rainfall, prolonged inundation occurred at the end area apart from the pumping stations. Exemplifying prolonged inundation at the end area, Fig. 11 depicts the simulation results

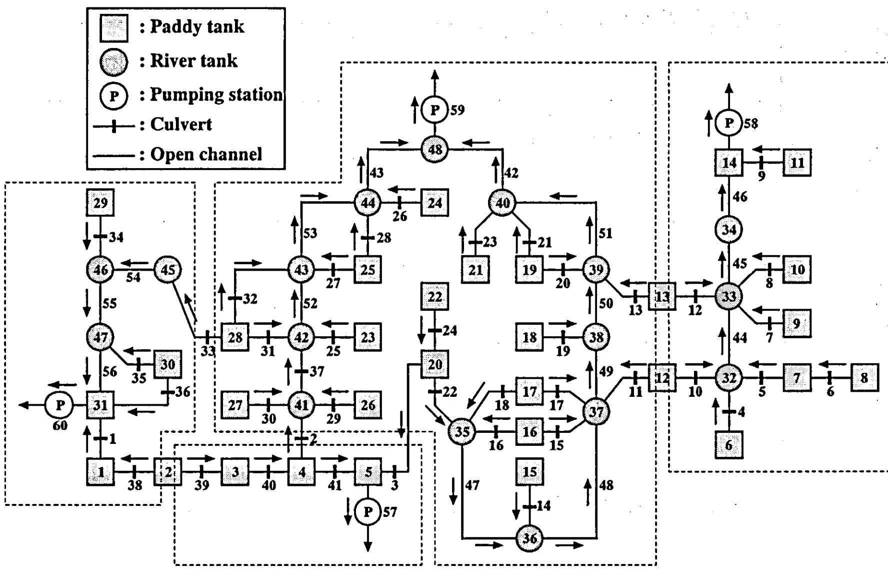

Fig. 10. Flow directions at every connection branch obtained by the numerical simulation using the virtual rainfall time series.

at the eastern part of the study area. In paddy tanks 11 and 14, the excess water was rapidly removed by pumping station 58 nearby. The water levels in river tanks 32 and 33 hardly exceeded the upper limit of the insensitive control of pumping station 58 because the excess water in the first canal moved quickly to pumping station 58. In paddy tanks 7, 8, 9 and 10 located at the end area apart from pumping station 58, inundation was prolonged and the inundation duration reached two days in paddy tank 10. Phu Lam Commune had 16 pumping machines and a total drainage capacity of 16,000 m^3 h^{-1} (Hai and Egashira, 2002), corresponding to the ability to remove the gross rainfall of about 35 mm in the whole drainage area within 24 hours. Consequently, inundation in the paddy tanks at the end area may be due to the low flowing capacity of drainage facilities between the first and the second canals resulting from inadequate maintenance of drainage facilities; sluggish, weedy canals and culverts (shown in Fig. 5); complicated drainage networks; and extremely flat topography in the study area. The simulation results indicated that the model presented here had good applicability, since the results obtained were in accordance with the characteristics of the inundation and excess water removal in Phu Lam Commune.

For further study

Because of the lack of information about the basin topography and the dimensions of drainage facilities in Phu Lam Commune, this paper could not address the present situation of excess water removal and inundation during the rainy season. This study's results

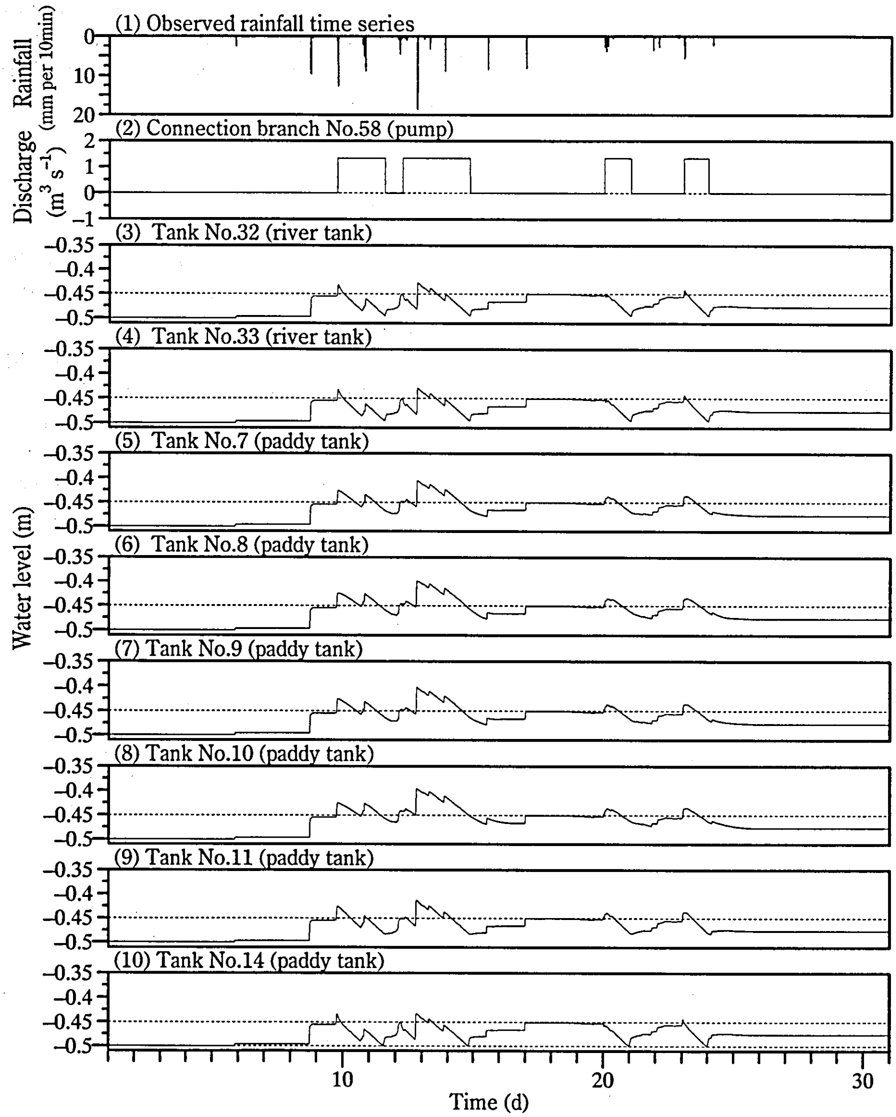

Fig. 11. Variation of the drainage discharge at pumping station 58 and the water levels in the neighboring tanks (see also Fig. 6).

of modeling and numerical simulations confirm the need for accurate and spatial evaluation of the present drainage situation in Phu Lam Commune during the rainy season in order to enhance the model applicability and then to identify the potential problems in the further development of the drainage system in Phu Lam Commune. For further study, the following information should be collected by field surveys including interviews with farmers: accurate drainage network and topographic data, accurate capacity and arrangement of pumping machines, actual operation of pumping machines, rainfall time series data at one–hour intervals or shorter, and actual situations of inundation during past floods.

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