5. セミナー資料

セミナー講演者であり、第1執筆者である農業工学研究所吉野室長にご了解を頂き、スーダン、エジプト両国でのセミナーの資料を掲載する。

Study on the Effects of Operating Method for Offtake Regulators Considering Flow-Arrival Time

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I. Introduction

In recent years, agriculture and related fields have under gone large changes not only in Japan but throughout the world. Paddy fields which had been the main object of irrigation up to now have been provided with farm irrigation ditches so as to enable farmers to control water freely from the plot-to-plot irrigation, and upland field has begun to be irrigated recently. Modernization of such irrigation systems has brought increased water requirement and greater fluctuation of it. In addition, the demand for city and industrial-use water has grown with the development of industries and changes in life-style, thus water resources have come to be crucial. These situations demanded an unsteady water supply which could balance water requirement with the control of wasteful discharges when construction or improvement of the irrigation canal system and water management.

Facilities such as check gates, regulating reservoirs, small ponds called farm ponds, manipulating type offtake regulators, etc., have been adopted as irrigation facilities to realize these demands.

An irrigation canal system with an appropriate buffer function makes it possible to efficiently conduct an unsteady water supply that balances the fluctuating water requirement with the prevention of surplus water. However, in an irrigation canal system with little buffer function, the systematic operation of facilities can be considered for accomplishing a certain degree of water management as mentioned above.

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This report covers a project in which we constructed an unsteady flow simulation model for the main canal in the Huay Luang Project, which is located in the northeastern region of the Kingdom of Thailand. This Huay Luang Project had no regulating reservoirs along the main and little buffer function. The operations of check gates and offtake regulators are simulated in order to study the facility operation method for complying with the changes in demand and for controlling wasteful water.

II. Outline of the Huay Luang Project and Outline of the Unsteady Flow Simulation Model

1. Outline of Huay Luang Project

The Huay Luang Project, the project intended for research, is located in the Udon Thani Prefecture in northeastern Thailand, with an irrigation area that covers about 13,000 ha. (See Figure 1.) The main water source is dependent on the Huay Luang Dam built on the Luang River, a branch of the Mekong River. The crest height of this dam is 12.5 m, the crest length is 4.9 km, it has an effective water storage capacity of 1.13 hundred million m³, and its

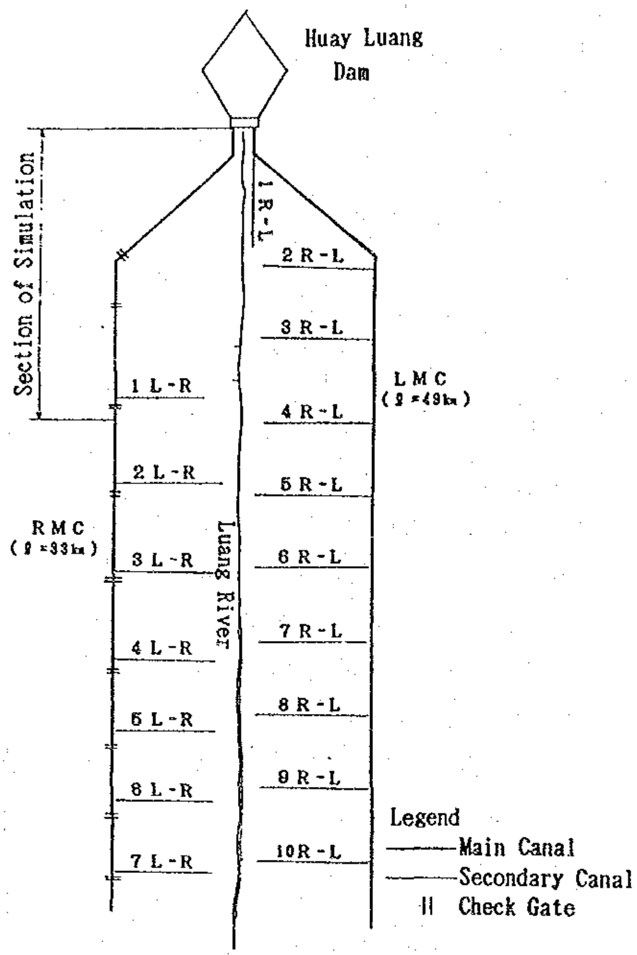

Fig. 1 Outline of Huay Luang Project

catchment area is 664km². Moreover, this irrigation canal system consists of the left main canal (total length is 49 km, maximum flow rate 6.3 m³/s) and right main canal (total length is 33 km, maximum flow rate 6.1 m³/s) which hold the Luang River in between, and secondary canals branching from the main canal mentioned above. In addition to this, it also consists of tertiary canals branching form these main

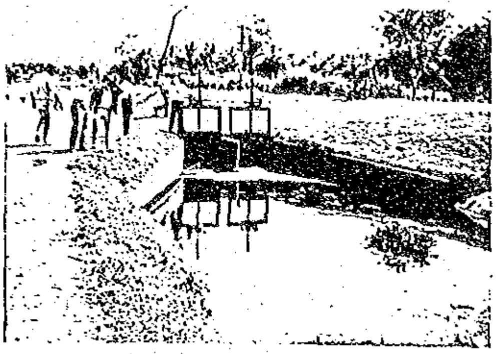

Photo-1 Check Gate No.1(5+198)

Photo-2 Constant head orifice offtake regulator

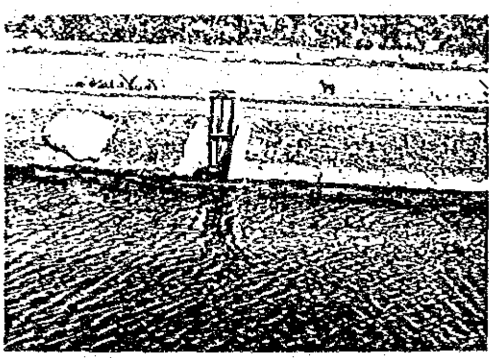

Photo-3 Single gated-offtake regulator

canals and secondary canals. Many of the main canal are trapezoid-shaped, which are made of a three-sided concrete lining section and a two-sided concrete lining section. The bottom gradient is from 1/4,000 to 1/16,000 for the right main canal. Hand-operated check gates have been installed intermediately in the main canal and staffs of the Royal Irrigation Department (RID) operate these check gates. (See Photo 1.) In addition, secondary canals and many tertiary canals diverge from the main canal, and the type of offtake regulators for the former is a constant head orifice offtake regulator, and that of the latter is a single gated-offtake regulator. (See Photos 2 and 3.) The main canal have concrete linings that have slipped midway in several places, and water plants have grown thickly, especially in the gentle gradient sections of the midstream and downstream parts, It can not be said that good maintenance have been conducted.

2. Outline of Unsteady Flow Simulation Model

Since the objective is to study an appropriate check gate and offtake regulators operation, the flow condition in main canal is the unsteady flow. Therefore, it was decided that construction of a numerical simulation model of an unsteady flow would be the best way to evaluate the various operations quantitatively. The simulation, model was decided to compare with on-site water level observation to judge the appropriateness of our unsteady flow simulation model.

Therefore, The simulation model was decided to construct the simulation model up to the 13 km point (2 km downstream from the check gate No. 3) from the intake work of this right main canal from Huay Luang Dam by taking into consideration the observation labours, etc.

The main structures in this area are the chute, syphon, drop, check gates (3 places), structures crossing rivers (2, rectangular aqueducts), single gated-offtake regulators (21), constant head orifice offtake regulator (1), junction work (1) and irregular form of stone masonry fixed weir reportedly constructed by farmers. (See Table 1.)

It was decided that the analysis method was to be of the unsteady flow analysis method utilizing the central differences method developed by Dr.M.Nakamura and Dr.H.Shiraishi[1). The handling methods for the following structures in the unsteady flow analysis are as follows.

Fixed Weir : Instead of solving the equation of motion, The fixed weir is positioned at the flow velocity calculation point (i) and obtain the mean velocity of flow (V_i) and passing discharge (Q_i) from water level H_{i-1} at the upstream water depth calculation point and water level H_{i+1} at the downstream water depth calculation point by using the rectangular weir formula, one of Honma's formulas[2). The equation of continuity is solved by using the passing discharge, and the H_{i-1} and H_{i+1} after delta t are obtained.

Check Gate : The check gate is positioned at the velocity calculation point (i) as in the fixed weir. By judging from the upstream and downstream water level and gate opening, the mean velocity (v_i) and passing discharge (Q_i) are obtained from the water level difference between the upstream and downstream by the formula of the submerged orifice or incomplete orifice[3). The equation of continuity is solved by using the passing discharge, and the water depth and water level of the upstream and downstream after delta t are obtained.

Syphon : As with other structures mentioned above, the syphon was positioned at the velocity calculation point (i), the following equation from the rigid water column

Table 1 Structures analysized in Right Main Canal

Name of Structure	Distance from upstream end (km)	Main Dimensions,etc.	Size of Main Canal	Remarks
Drop work	0.355	length : 35m, fall : 2.7m		i means bed slope s means side wall slope b means bed width
Fixed weir	1.16	Irregular shape made by stones height : about 0.7m	i=1/4,000, s=1:1.5, b=2.5m side walls and bed is lined	
Syphon	1.456-1.546	length : 90m, Double, Square 1.2 × 1.2m		
Check Gate No.1	5.203	width 1.2m × 2 leaves, manual	i=1/8,000, s=1:1.5, b=2.5m side walls and bed is lined	
Check Gate NO.2	6.035	width 1.5m × 1 leaf, manual		
Chute works	6.035-6.280	Rectangle, width 1.55m, fall 7.1m length 245m	i=1/16,000, s=1:1.5, b=5.0m Only side walls are lined	
Junction of old canal	6.584	width 1.55m, height 1.25m Gate type		
Aquaduct	7.730-7.750	Rectangular, length 20m, width 3.25m		
Aquaduct	10.720-10.740	Rectangular, length 20m, width 3.3m	i=1/16,000, s=1.5, b=9.0m Only side walls are lined	
Check Gate No.3	11.105	width 1.2m × 2 leaves, manual		

theory based on the water level difference between the upstream and downstream is solved by the Runge-Kutta method, and the velocity and discharge in the syphon are obtained

$$\frac{\ell}{g}\frac{dv}{dt} = H_{i-1} - H_{i+1} + \frac{n^2 \mid v \mid v}{R^{4/3}}$$

Here, ℓ : siphon length,

g : acceleration of gravity,

n : roughness coefficient of Manning,

v : velocity in a syphon,

R : the hydraulic Radius

Drop : The drop is positioned at the velocity calculation point (i). The critical depth h_{ci-1} of the (i-1) point is obtained from the discharge of the upstream velocity calculation point (i-2) and the canal shape of (i-1), which is the water depth calculation point.

The water depth H_{i-1} obtained from the equation of continuity and the critical depth h_{ci-1} are compared, and when it reaches $h_{ci-1} > H_{i-1}$, H_{i-1} is regarded as the critical depth, v_i is regarded as the critical flow velocity, and it is made ${}_nQ_i = {}_nQ_{i-2}$.

Single gated-offtake regulator: The single gated-offtake regulator is positioned at the water level calculation point, the velocity and discharge through the pipe of offtake regulator are obtained by Manning's formula based on the pipe length and water level difference between the water level H_1 of the main canal before delta t and the water level in tertiary canal (since there were no data this time, the water level was assumed as being at the center of the offtake pipe), and the equation of continuity was solved by using this discharge through offtake regulator as the lateral outflow discharge.

Constant head orifice offtake regulator: this offtake regulator is positioned at the water level calculation point, as was the gated-offtake regulators, and the discharge through the offtake regulator is calculated from the water level in main canal before delta t, water level of stilling pool (assumed as 183.772 m), coefficient of velocity (0.60) and gate opening. The equation of continuity is calculated by using this discharge through the offtake regulator as the lateral outflow discharge.

The identification of the roughness coefficient, which is a field constant, is an important factor for conducting an unsteady flow simulation conforming to the actual site.

Water level observations were conducted at 18 places for identification of roughness coefficient and at 16 places for testing the simulation model. The water level observations were conducted at 5-minute intervals from 9:00 A.M. to 16:00 P.M. on November 3, 1989 by visual observation of staff gauges fixed to side walls, etc. In addition, the discharge gate of the dam was operated for 2 minutes and 24 seconds from 9:20 A.M. on the same day and the discharge was changed from 2.2 m^3/s to 3.4 m^3/s.

Junction work and all offtake regulators were closed during the observation period.

The simulation model was constructed, mainly dependent on the design books such as the longitudinal drawings of main canal, structural drawings of structures, etc. (See Figure 2.) Moreover, the discharge-time relationship was given to the upstream end as boundary condition, and by taking into considering the simulation of Chapter 3, a boundary condition was given to the downstream end so that flow water depth became uniform. Then, Manning's coefficient of roughness was estimated based on the initial discharge, final

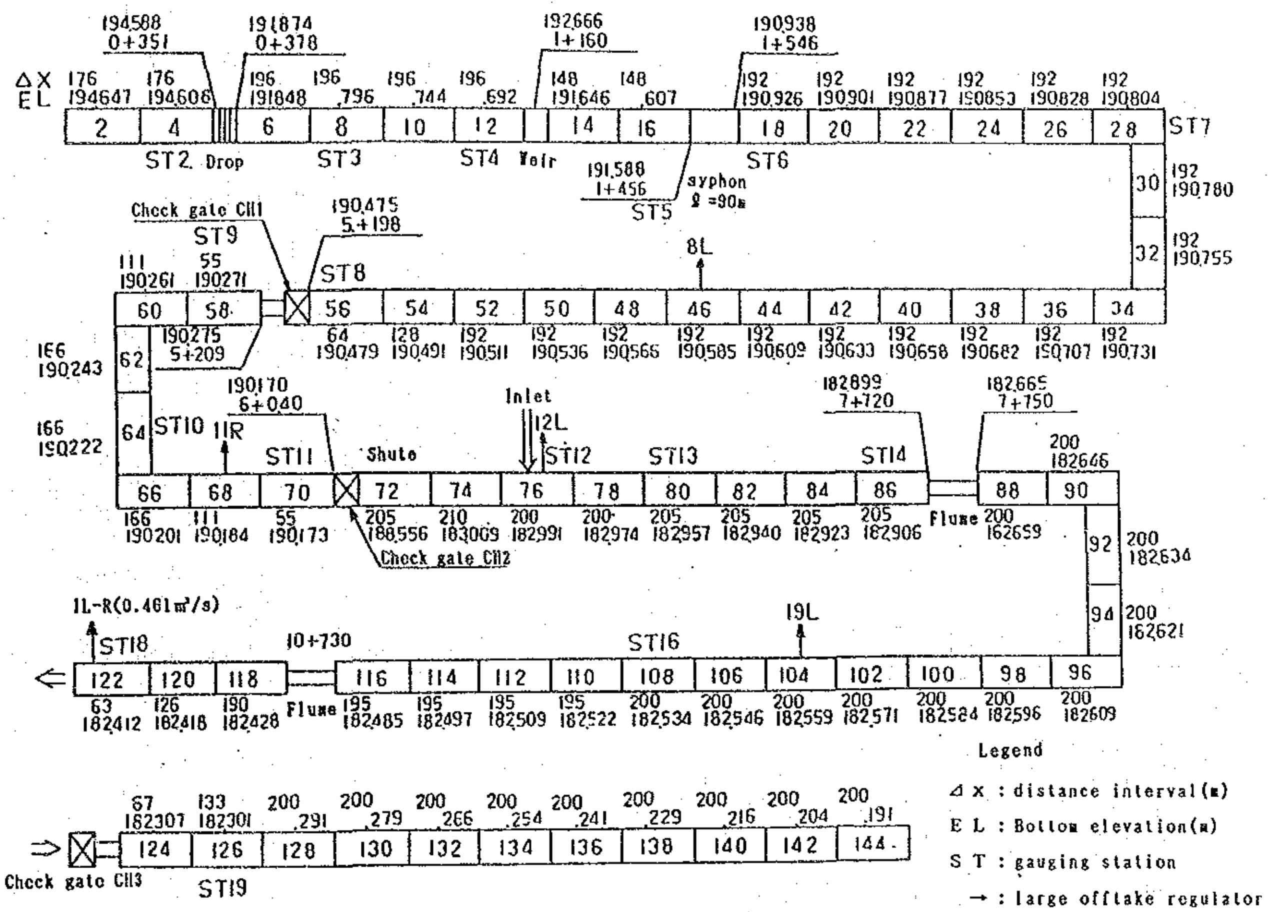

Fig. 2 Unsteady Flow Simulation model

discharge, initial and final water levels at each gauging point. Manning's coefficient of roughness obtained from the two discharge conditions was within the range of 0.020 - 0.070.

In observing the assumed coefficient of roughness, an extremely high value was shown in the gentle gradient canal of the middle reach, which is located more downstream than the junction work from the old canal.

Although the declining of water passing efficiency due to the slope collapse and the thick growth of water plants can be considered the causes, the roughness coefficient of the lower reach is not so great. Therefore, although the canal bottom elevation of the design book has been used in this analysis, the actual cause can be considered primarily that the canal bottom elevation has become high through sediment inflow from the old canal.

However, the canal cross section was not surveyed because of the enormous costs and time involved. Therefore, we regarded these values as the characteristic values of the simulation model in this research and decided to simulate check-gates and regulators operations, by the model based on this value.

The elevation of water level gauging points were leveled prior to the water level observation. In addition, although it had not completely reached the steady condition in the lower reach of the main canal after changing the discharge to 3.4 m³/s, it was regarded as a steady flow and we estimated of the roughness coefficient. Then an unsteady flow was simulated under the same discharge condition as the water level observation. The result, shown in Figures 3 and 4, compares the observations and the unsteady flow simulation. As it is clear from these drawings, the constructed simulation model can be considered sufficiently accurate for pursuing the actual hydraulic phenomenon.

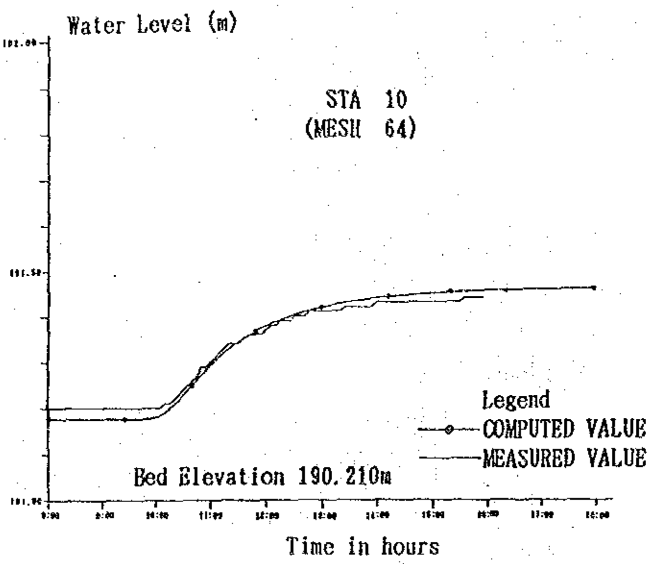

Fig. 3 Simultaneous Measurement (1)

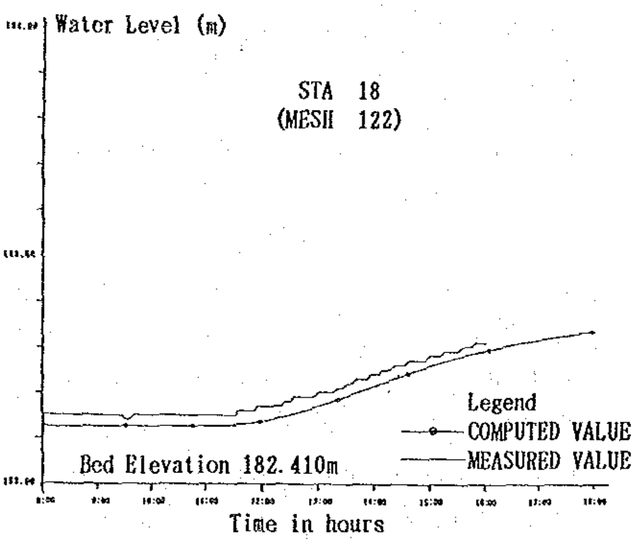

Fig. 4 Simultaneous Measurement (2)

III. Proposal for Method to Improve the Gate Operation Technique

1. Issues of present the Gate Operation Technique

We have seen that despite the fact that operations in an irrigation canal system generally affect the entire flow condition as a system, there are many cases where the gate is operated as independently by only taking into consideration the water level difference between the upstream and downstream at a certain time, necessary passing discharge, etc. In such cases, the entire system is often operated simultaneously at a specific time, according to a sixth sense based on the long experience of the irrigation engineers or gate operators. For example, in the Project M of country A, gates are simultaneously operated three times a day (7:00 AM, 11:00 AM, 3:00 PM) according to the flow condition at that time and discharge instructed by the control center.

Since the gate openings are adjusted all at once according to the flow condition at that time, it is assumed that the realization of the target flow condition will be impossible because the flow condition itself will change over time. In addition, although it was explained that the gate operator in the Huay Luang area operated the check gate at need, there are no operation manuals on which to base water management operations; there is nothing which shows clearly the relations among the water level difference, gate opening and passing discharge; the check water level is also not clear. Therefore, it appeared that gate operation relied on the sixth sense of the gate operator, and gates were operated so that water did not overflow from the main canal.

Originally, it seems that the basis for water management of the main canal was to supply efficiently and steadily the irrigation water conforming to the water requirement which changes spatially and with time. However, it is also appears that ideal water management will be impossible as long as the gates are operated as described above.

2. Methods of Operating Check Gates and Offtake Regulator Gates

This paragraph will examine a gate operation method capable of supplying irrigation water efficiently and steadily according to changes in water requirement by using the unsteady flow simulation model constructed in Paragraph II.

First of all, the water requirement will be estimated as follows. There are an offtake regulator to secondary canal in one place and offtake regulator to tertiary canal in 21 places in the 11 km section of the upper reaches of the right main canal. When all of these offtake regulator gates are closed and check gate No. 3 is supplying the water of 2.2 m³/s to the lower reaches only, while, simultaneously a designed discharge becomes necessary in comparatively large-scaled offtake regulators in 5 places, a discharge increase of 0.298 m³/s becomes necessary in the lower reaches of check gate No. 3, that is, the total water of 3.4 m³/s gets necessary. After the continuation of these necessary discharges for 24 hours, it is decreased to the initial necessary discharge again. (See Figure 5.) One typical operation method to consider for these gates is the method of operating the gates all at once, and another method is to operate them in order from the upstream gates by taking into consideration the water arrival time.

Various times can be considered also on the water arrival time in this case. For example, when water is discharged in the end of the upper reaches of the canal and the discharge is increased by delta Q, flow condition change of point A at a fixed distance generally is as shown in Figure 6.

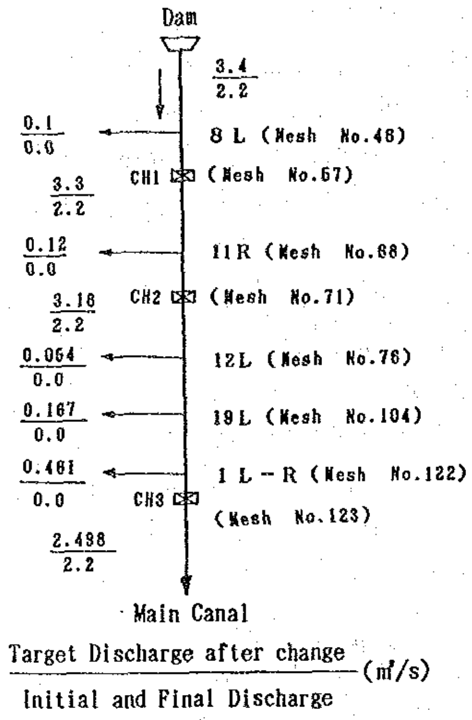

Fig.5 Full illustration of water requirement

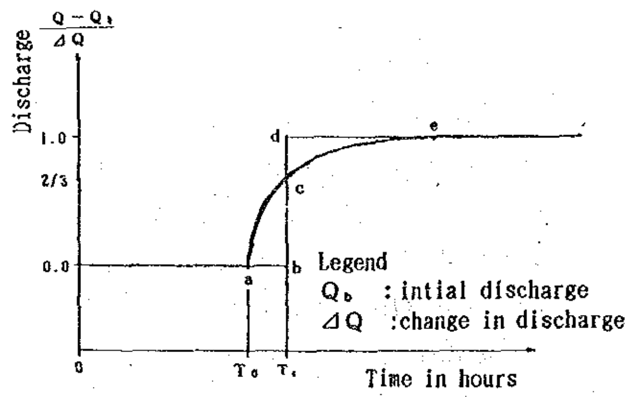

Fig.6 Illustration of concept of water arrival

In this case, there may be occasions when time T_0, the change starting time, is made the water arrival time. Moreover, there may also be occasions when the arrival time of the irrigation water of 0.9 delta Q discharge is made the water arrival time. However, the water arrival time will be made 2/3 delta Q here. According to study by Dr.K.Iwasaki[4], the reason for this is that the area of abc is equal to the area of cde and even when operating a gate for water-intake of necessary discharge by the offtake regulator provided at point A at time T_1, it can be assumed that effects will not be exerted more substantially at the main canal discharge in the lower reaches than they are at the offtake regulator. In addition, time T_1 is obtainable by delta V/ delta Q. Here, delta V is the change of storage volume in the canal.

Precisely speaking, it is considered that the water arrival time T_1 up to each check gate and each offtake regulator should be obtained, and these facilities should be operated according to these arrival times. However, it becomes complicated when considering actual gate operation, and it is almost impossible. Therefore, the irrigation water arrival time up to the check gate will be obtained in the first place, and the offtake regulator located in the upper reaches from this will be operated by this time. The water arrival time up to each check gate is shown in Table 2.

Table 2 Arrival Time between each section

	Mesh No	Distance (km)	Initial Discharge Q_1 (m³/s)	Initial Storage Volume V_1 (m³)	Final Dischage Q_2 (m³/s)	Final Storage Volume V_2 (m³)	ΔQ (Q_2-Q_1) V (m³/s)	ΔV (V_2-V_1) (m³)	$\Delta V/\Delta Q$ (T) (sec)	T_1 (ΣT) (sec)	Remarks
Upstream	2	0									
			2.2	30,682	3.4	35,518	1.2	4,836	4,030 (1.12)		( ) means hour unit
CH1	57	5.198									
			2.2	6,318	3.3	6,437	1.1	119	108 (0.03)	4,138 (1.15)	
CH2	71	6.040									
			2.2	71,593	3.18	75,870	0.98	4,277	4,364 (1.21)	8,502 (2.36)	
CH3	123	11.100									

By taking into consideration the water management which we have observed up to now, the following 7 cases of concrete operation methods which aimed at realizing a more suitable operation method were assumed based on the method of: operating gates all at once regularly; operating them several times all at once regularly and daily; and operating them by considering the water arrival time.

① Case 1 : Simultaneously with changing the discharge from the dam, the opening of the check gates and gates of offtake regulators are adjusted according to the upstream and downstream water levels and necessary discharges of facilities at that time.

② Case 2 : T_1 hours after changing the discharge from the dam, the gate openings are adjusted based on the water level at that (moment).

③ Case 3 : $2 \cdot T_1$ hours after changing the discharge from the dam, the gate openings are adjusted based on the water level at that (moment).

④ Case 4 : Simultaneously with changing the discharge from the dam, the check gate openings are adjusted according to the check water level for the upstream water level and according to water level at that time for the downstream water level. Moreover, the openings for the gates of offtake regulators will be set so that the necessary offtake discharge can be diverted according to the main canal level at that time. In addition, readjustment of gates shall be conducted three hours later under a similar idea. Since the check water level was not expressed clearly in this project, the water level at designed discharge was made the check water level.

⑤ Case 5 : After changing the discharge from the dam, the gates are operated in the same way as case 4 after the lapse of time T_1 and time $(3.0 + T_1)$.

⑥ Case 6 : Obtain in advance the water level of the flow directly under each check gate after discharge increase by the non-uniform flow calculation, and obtain the necessary opening of the gates after discharge increase from this level and the check water level. Simultaneously with the dam discharge change, the opening of gates will be changed to this opening. The opening of the gate of offtake regulator 1L-R will be calculated according to the check water level of check gate No. 3, and other offtake regulators will have their gate opening adjusted according to the main canal level at that time.

⑦ Case 7 : As in case 6, the necessary opening of check gates are calculated in advance. Adjust check gates and offtake regulators gates openings, after the lapse of time T_1 from the discharge from the dam change under the same reasoning as in case 6.

3. Unsteady flow simulation results and their discussion

Table 3 shows the 24-hour water volume difference through the check gates and offtake regulators obtained from the respective unsteady flow simulation and the originally necessary discharge. Moreover, the discharge change of major points among the simulation results are shown in Figures 7 to 10. The discharge from the dam was increased to 3.4 ㎥/s in 20 hours and the operation of discharge reduction was conducted in 44 hours.

The effect of gate operation which considers the water arrival time becomes clear when comparing case 1 with case 7. In operations of case 1, case 4 and case 6, in which the water arrival time is not considered, together with the necessary discharge not being secured in each check gate and offtake regulators located in the downstream section of the main canal in case of discharge increase, the flow condition also becomes unstable for a long period. Furthermore, more than the necessary discharge become diverted on the offtake regulators located upstream. (See Figure 7 to 10 and Table 3.)

For example, in case 1, the 24 hours mean surplus volume of the offtake regulator 8L is 15 percent. The offtake discharge had actually reduced with the dropping of the water level at the beginning, and the surplus offtake discharge of about 20 percent was generated just before the lapse of 44 hours. On the other hand, a large water shortage was generated for a long period in the offtake regulator 1L-R located downstream as a result of this influence, and it was clear that it became equal to of upstream priority.

In contrast to this, the irrigation water which has become unnecessary flows down to the lower reaches of the check gate in the case of discharge decrement, and as long as there are no storage facilities such as regulating

Table 3 Water through gates and Water in Offtakes calculated by Simulation

Name of Structure	Pattern	Water Requirement(m^3)	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6		Case 7	
			Surplus/ Shortage Water(m^3)	Ratio (%)	Surplus/ Shortage Water(m^3)	Ratio (%)	Surplus/ Shortage Water(m^3)	Ratio (%)	Surplus/ Shortage Water(m^3)	Ratio (%)	Surplus/ Shortage Water(m^3)	Ratio (%)	Surplus/ Shortage Water(m^3)	Ratio (%)	Surplus/ Shortage Water(m^3)	Ratio (%)
CH 1	Increment	285,120	-9,893	-3.5	-7,028	-2.5	-6,987	2.5	-6,464	-2.3	957	0.3	-4,560	-1.6	75	0.0
	Decrement	190,080	7,708	4.1	4,828	2.5	4,362	2.3	8,750	4.6	-1,655	-0.9	4,362	2.3	139	0.1
CH 2	Increment	274,752	-11,807	-4.3	-8,575	-3.1	-7,214	-2.6	-6,862	-2.5	1,334	0.5	-4,360	-1.6	846	0.3
	Decrement	190,080	8,596	4.5	5,635	3.0	5,067	2.7	9,336	4.9	-1,672	-0.9	4,371	2.3	96	0.1
CH 3	Increment	215,827	-10,235	-4.7	-7,277	-3.4	-5,776	-2.7	-7,349	-3.4	738	0.3	-3,597	-1.7	1,107	0.5
	Decrement	190,080	9,887	5.2	6,026	3.2	4,874	2.5	9,725	5.1	-45	-0.0	5,227	2.7	-309	-0.2
8 L	Increment	8,640	1,279	14.8	1,030	11.9	605	7.0	650	7.5	-487	-5.6	115	1.3	-368	-4.3
11 R	Increment	10,368	737	7.1	768	7.4	389	3.8	79	0.8	39	0.4	-263	-2.5	-262	-2.5
12 L	Increment	4,666	380	8.1	88	1.9	21	0.5	412	8.8	114	2.4	432	9.3	6	0.1
19 L	Increment	14,429	-176	-1.2	-57	-0.4	-67	-0.5	-162	-1.1	3	0.0	-132	-0.9	-138	-1.0
1 L-R	Increment	39,830	-4,672	-11.7	-2,038	-5.1	-1,479	-3.7	-2,238	-5.6	609	1.5	-3,614	-9.1	607	1.5

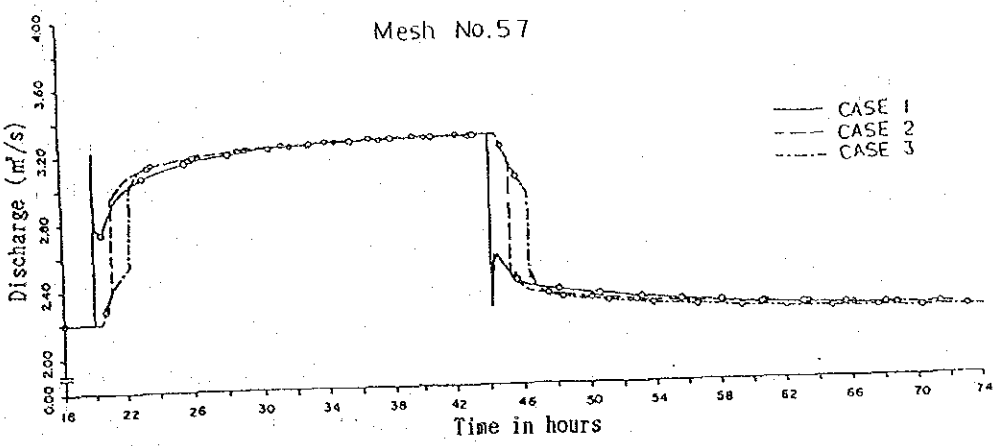

Fig. 7 Simulation Results at Check No. 1 (1)

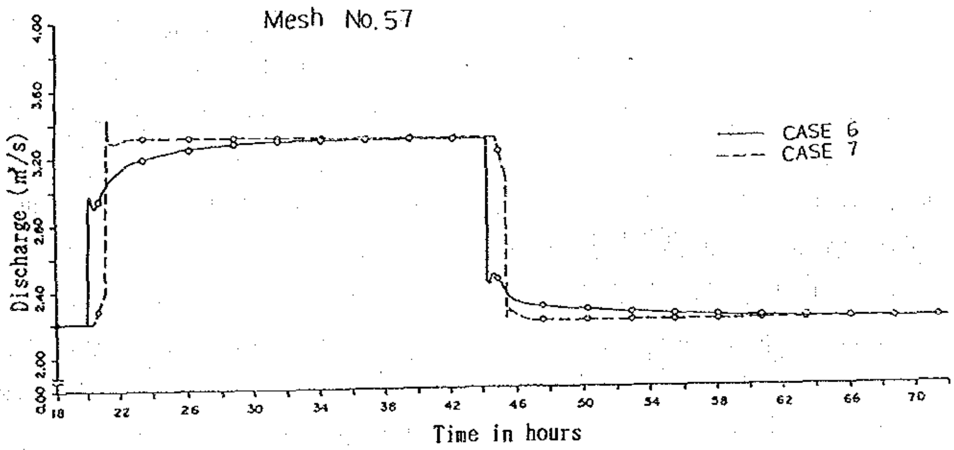

Fig. 8 Simulation Results at Check No. 1 (2)

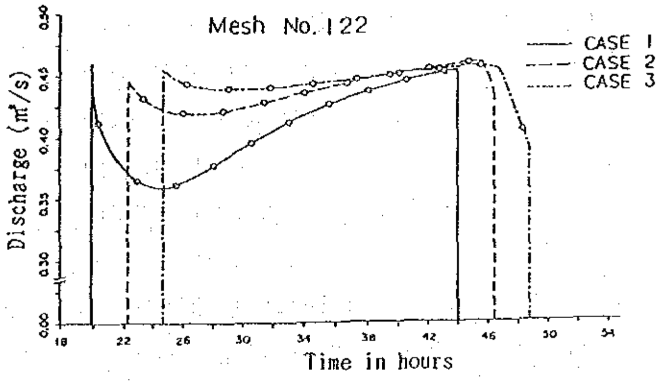

Fig. 9 Simulation Results at 1L - R (1)

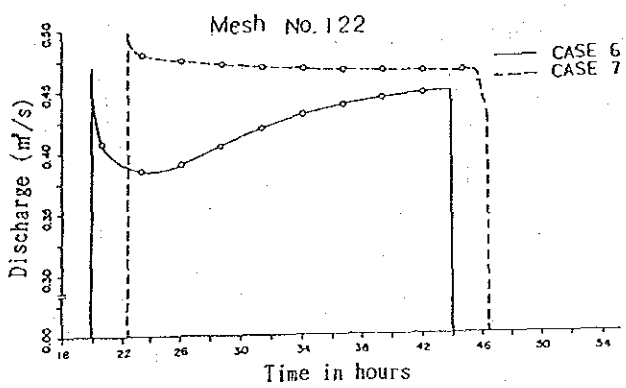

Fig. 10 Simulation Results at 1L - R (2)

reservoir, etc., it will finally be discharged wastefully from the irrigation canal system. For example, in check gate No. 3 of case 1, it rises to about 5 percent more than the necessary discharge at 24 hours average, reaching 10,000 m^3. However, in case 2 and case 3 the water arrival time is taken into consideration and gates are operated according to the water level at that time. Although the surplus and shortage volume are reduced in comparison with case 1, it appears that there is not so much difference in comparison with case 6, which does not include the water arrival time and operates gates by estimating the future water level. However, when observing the volume change of offtake regulator 1L-R over time, the offtake discharge is comparatively stable in cases 2 and 3, in contrast to the offtake discharge shortage that continues for a long period in case 6 and case 1. Although the flow condition is slightly improved in case 3 when compared with case 2, offtake discharge drops remarkably just before the closing of offtake regulator gate. (See Figures 9 and 10).

Among the seven water management methods, case 7 can be considered as the best method, and case 5 ranks next. Especially in case 7, a necessary and stable flow condition is realizable over the entire area almost simultaneously with gate operations. In addition, unnecessary irrigation water does not flow down lower than the check gate No. 3 in the case of discharge decrement, and storage within the system is possible.

The gate operation method of case 7 includes the work of estimating the check gate downstream water level in advance and becomes slightly more complicated than other methods. However, estimation is possible by calculating the various non-uniform flow, and gate operation by this method is certainly possible. In addition, although the operation frequency of each gate will increase by one time in case 5, it will not be impossible as a general gate operation system. In any event, it has become clear from these simulations that wasteful discharge can be prevented at discharge decrement while stabilizing the flow condition of the entire system and making stable offtake discharge possible by operating gates according to the water arrival time.

IV. Conclusion

Taking the upstream section of the right bank main canal of the Huay Luang Project in the Kingdom of Thailand as the example objective project. The improvement of the gate operation method were studied by using the unsteady flow simulation.

By gate operation which has added the water arrival time into consideration following these various researches, it appears that many irrigation engineers had expected more stable water management would become possible. However, the effect has not been proved with concrete numerical values, but we believe that this analysis has clarified the matter. An individual unsteady flow for each system must be simulated on the water arrival time or the time obtained empirically must be used, since the water arrival time differs according to each irrigation canal system. However, it can be assumed that the water management of the main canal can be greatly improved by gate operation which includes the water arrival time.

The principal emphasis of water management in this study has been strictly placed on stabilizing the flow condition, with the premise that water requirement for irrigation could be suspended during the water arrival time or that estimated operation was possible. In other words, it made the supply oriented type water management the premise. Enhancement of the buffer function must be promoted in an irrigation canal system which aims at a demand oriented type water management with irrigation water supply conforming to water requirement[5]. In addition, for managing efficiently the water, the exact grasping of water requirement in each field and the preparation of offtake regulator or check gate capable of realizing a accurate discharge realizing and water measurement facilities are necessary.

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6．報道

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