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Studies on the Simulation Model of Automatic Diversion Apparatus with Buoyancy Effect

Basic Model and its Application to Neribeya Diversion Works in Toban Irrigation Project

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浮力を利用した自動定量分水装置のシミュレーションモデルについて

基礎的な定式化と東播用水練部屋分水工への適用について

竹　山　光　一

1. Introduction

Many diversion apparatus from open canal are set in many agricultual area and projects. The investigations on the improvements of the accuracy and the hydraulic functions of the automatic diversion control devices are, so far, not enough. Generally pipes in the bank, orifices and sluice gates in diversion works have been used for water diversion from open canal.[1)] These traditional diversion apparatus have originally the peculiar defects that the diversion discharge is affected due to the variaion of water level of the canal. To prevent this variation, engineers have used the check gates in the canal, but these gates sometimes artificially disturb water flow in the canal.[2),3)] The automatic diversion apparatus is demanded to have a simple structure and high accuracy from the standpoints of saving the water and labour in the management of water uses. With some experimental results, this paper deals with the hydraulics of the apparatus with stop plate, its simulation model of the diversion work and its application to the Neribeya diversion works in Toban Irrigation Project.

2. The function of automatic diversion apparatus with buoyancy effect

Fig. 1 is the plan of the apparatus from open canal. It is a hydraulic system connecting with open canal by screen inlet, diversion tank and measurement weir. And below them, a vertical diversion pipe and a stop plate are rigidly connected by some slender staffs with regulation float, which consists of styrene foam and coverd with light metal. As the pipe opening is controlled in the height by the automatic rise and fall device of the regulation float according to the larger or smaller discharge than the designed one, we can set the compatible length of the staffs according to the designed discharge of the weir.[4),5)]

3. Experimental investigation and basic hydraulic computation

3-1. Dimensions of the canal and the weir in experiment

The dimensions of open canal were 0.5 (m) in width, 5.1 (m) in length, 0.5 (m) in

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height and its bottom was about 0.2 (m) higher than the crest of the diversion weir.

The dimensions of diversion weir and tank were D1 0.1 (m), and D2 0.125 (m) in the diameter of the diversion pipe, 3.0 (m) in length, 0.3 (m) in width of the weir b, 1.0 (m) in width of the tank B, 0.55 (m) in height from the crest Hd, 0.7 (m) in side length of the regulation float of square BL and 0.1 (m) in thickness.

In addition, the case without stop plate was added on account of comparison.

3-2 Outline of the basic equations

The height between the plate and the pipe is a0 on the first condition and it varies as a in (1) by the moment of the water and its buoyancy.

$$a = a0 - h + \rho Q*v1/((BL)**2*g) \qquad (1)$$

where ρ is density of water ang g is acceleration of gravity.

When the cross section is contracted suddenly from A1 to A3,

(A1 = pai*D1*D1/4, A2 = pai*D2*D2/4, A3 = pai*D1*a, where pai = π)

the head loss between the water head in open canal H and one in diversion weir h is eq. (2).

$$H - h = (fi + fb*2 + fn2*\ell 2/D2)*v2**2/(2*g) + (fb + fn1*\ell 1/D1)*v1**2/(2*g)$$
$$+ (fo + fsc)*(A1/A3)**2*(v1**2/(2*g)) \qquad (2)$$

fi : coefficient of entrance loss

fb : coefficient of bend loss

fnj (j = 1, 2) : coefficient of friction loss

fo : coefficient of outlet loss

fsc : coefficient of sudden contraction loss

Then the diversion water discharge from the weir Q can be get as (3) by the Francis formula, and considering the appraoch velocity head, (4) is obtained.

$$Q = 1.84*(b - 0.1*N*h)*h*SQRT(h) \qquad (3)$$

$$Q = 1.84*(b - 0.1*N*h)*((h + va**2/(2*g))**1.5 - (va**2/(2*g))**1.5) \qquad (4)$$

$$va = Q/((Hd + h)*B)$$

N is a number of contracted side in the Francis formula (where N = 2). And the cross section between the pipe and the plate becomes narrow, the head loss at the outlet must be estimated by some another equations. That is, two equations (eq.) were examined, one was eq. (6)[6], and the other was eq. (7), which was eq. of orifice in pipe.

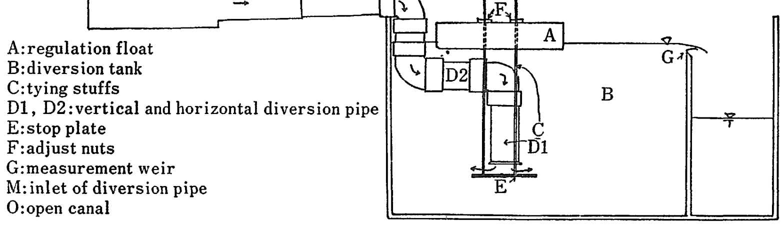

Fig. 1 Cross section of automatic eiversion apparatus producted experimentally

The comparison of (6) and (7) showed that when A3/A1 varied from 0.6 to 0.3 ZETA varied from 0.12 to 0.33 in (6), but FSC varied from 0.96 to 2.50. With some problems, eq. (7) was tried to adopt for its larger range of variation and it's estimation of the ability to "stop" the outlet flow.

$$C = 6.0/40.0*(A3/A1-0.2)+0.62 \tag{5}$$

$$ZETA = (1.0/C-1.0)**2 \tag{6}$$

$$FSC = (1/(0.597-0.011*(A3/A1)+0.432*(A3/A1)**2)**2 \tag{7}$$

3-3 Examination of the equation and experimental results

Table 1 is experimental results and Fig. 2 is the flow chart of this model. And their comparisons are shown in Fig. 3. Sensibility Sp is defined as Dh/DH.[1] In traditional diversion system with no stop plate, Sp was about 22 % as NO. 5 in table 1 (formerly abot 30 %[3]). On the contrary in the case with stop plate, Dh/DH were less than 0.035 as NO. 1-NO. 4 in table 1. Thus the apparatus showed the high function of stop plate.

The variation range Dh/DH of NO. 1 or NO. 4 in table 1 about 0.01 to 0.02 correspo-nded with the range 1.2 to 1.4 of DQ/DH (ℓ/sec/m).

Table 2 or Fig. 3 indicated that the relation between Q or h and H could be approxi-mated to parabolic curve by regression analysis as $Q = C0+C1*H+C2*H*H$.

Then the hydraulics of the apparatus with buoyancy effect could be expressed by this equation, and many simulations could be done aiming at reasonable design of demand discharge of agricultual crops.

4. Application of the model to Neribeya diversion works

4-1. Outline of Neribeya diversion works

Fig. 4 is the plan of this works, and which has 4 branch canals named TENMA INAMI, MORIYASU, KAKO and one overflow spillway named TENAKAIKE (Hy is the height of weir). Table 3 shows the maximam designed discharge of them. Nextly, the diversion pipe and stop plate are designed and set as Fig. 4.

Here, as Fig. 5, discharge in open canal is denoted by Qin, height by Z, water surface areas by F1 (at t = 0.0, F10), width by B0 and canal bottom slope i. They are connected by the pipe (length ℓ, cross section area f) with circular tank which has diameter D0. Branch pipes j (j = 1, 4) are set to stop plate as Fig. 4, and n (coefficient of the roughness of Mann-ing) was estimated as 0.013. Some of these values

Table 1 Experimental results

No.	1	2	3	4	5
MAX. H(cm)	32.0	45.0	43.0	42.0	35.0
MIN. H(cm)	11.0	20.0	17.5	18.0	11.5
MAX. h(cm)	3.9	5.9	6.0	4.7	5.1
MIN. h(cm)	3.7	4.5	5.2	4.2	0.0
ao(cm)	3—5	5—7	4—6	2—4	*
Sp	0.011	0.006	0.031	0.021	0.220

* no stop plate

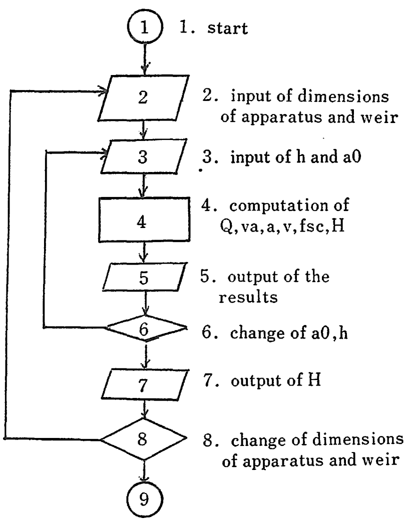

Fig. 2 Flow chart of the computaion of the hydraulics of the apparatus

are put in Fig. 5 (m•sec).

4-2 Basic hydraulic simulation model of RKG method

With the regression model it is possible to approximate any diversion discharge at NO. j by H to parabolic curve as in table 4.

Then the next model can be made.

$$F1 = F10 + Z*B0/i$$

$$F1*dZ/dt = Qin - f*V \qquad (8)$$

Table 2 The variation H (water height in open canal) and Q (diversion discharge) in basic formula by fsc=FSC, No. 2

(ℓ/sec)Q1=4.3		Q2=4.7	DQ=0.4	
BL(m)	H1(m)	H2(m)	DH(m)	DQ/DH(ℓ/sec/m)
0.3	0.25	0.41	0.16	2.50
0.4	0.30	0.54	0.24	1.70
0.5	0.33	0.62	0.29	1.40
0.6	0.35	0.67	0.32	1.30
0.7	0.36	0.70	0.34	1.20

b=0.30(m), a0=0.05(m)

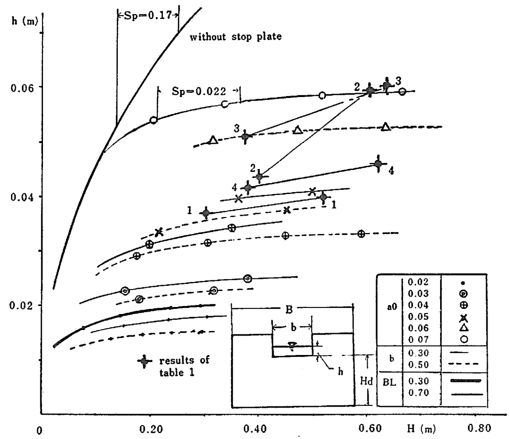

Fig. 3 results of caluculation and comparion with experiment

$$F2*dH/dt = f*V - Qout \qquad (9)$$

$$dV/dt*(\rho*f*\ell) = \rho*g*f*(Z-H) \mp C*V*V, \ (-: V>0, \ +V<0) \qquad (10)$$

$$C = \rho*f*1/2*(fio + fbo*2 + fno*\ell/D_o + foo)$$

subscript o in above eq. is coefficient in the flow from open canal to circular tank.

$$Qin = Qave + A0*sin(2.0*pai/T*t - pai/2), \ pai = 3.1415$$

$$dQin/dt = A0*2.0*pai/T*cos(2.0*pai/T*t - pai/2) \qquad (11)$$

$$Qtotal = C0 + C1*H + C2*H*H$$

$$d(Qtotal)/dt = (C1 + 2.0*C2*H)*dH/dt \qquad (12)$$

$$Qout = Qtotal + Qy$$

$$d(Qy)/dt = 1.84*SQRT(ABC)*(1.5*(By - 0.2*(H - Hy) - 0.2*ABC))*dH/dt \qquad (13)$$

$$ABC = H - Hy + Vy**2/(2*g), \ Vy = Qy/(By*H)$$

5. Results and Discussion

This simulation model consisted of 6 simultaneous differential equations with eq. (7). And this model was solved by RKG (Runge Kutta Gill) method in SSL II.[8)]

Initial conditions were t = 0.0 (sec), V = 0.0, v_j = 0.0 (j = 1−4), Z=H = h_j = 0.0 (j = 1−4), F10 = 50.0, F20 = 10.0 (m**2).

Boundary conditions were Qave = 1.25, 1.0, A0 = 0.5, 0 (m**3/sec), T = 120.0, dt = 0.4, 1.0, 2.0 (sec).

Fig. 6 and 7 were simulation results under these conditions. And they showed that this simulation had any vibrations in about t = 0∼300, and especially big vibration was founded about t = 300 (sec).

Then firstly that the water storage by dh/dt on surface in the tank and time lag were not reflected on this model, secondly that in the starting region as $H<0.1$, regression curve was so sharp that it made much vibrations, thirdly the response characteristics of the diversion works, and fourthly the variation of Qin was ± 40 % of average in 120 sec. They might have caused such effect, but they must be ensured by more complicated surging model of simultaneous differential equations as $d(a_j/dt$, $d(h_j/dt$, $d(v_j)/dt$, $(j = 1\sim4)$ are added and moreover with very short interval of time step.

Table 3 Designed max. discharge of each diversion and width of the weir by simulation

branch name	max. designed discharge	number	width of the weir by one	total width of the weir
KAKO	0.542	3	1.47	4.41
MORIYASU	0.356	3	0.96	2.88
TENMA	0.062	1	0.51	0.51
INAMI	0.137	1	1.10	1.10
TENAKAIKE(spillway)		1	2.42	2.42
TOTAL	1.097			

(m・sec.)

The selection FSC as fsc must be compared with the case of ZETA, but the strict coefficient fsc is so difficult to be estimated that many cases of fsc must be computed and their variations must be compared each other. After stabilization, this model showed the outline of diversion system.

These informations are very useful to design and examine the diversion works systems.

6. Summary

To estimate the head loss of the automatic diversion apparatus with buoyancy effect,

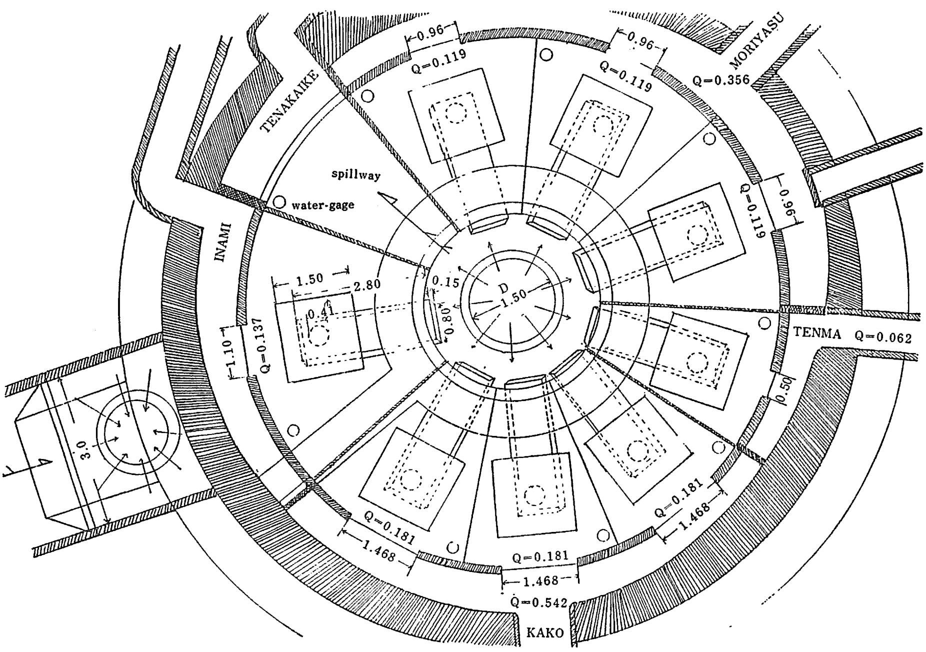

Fig. 4 plan of application of the apparatus to Neribeya diversion works in Toban irrigation project equation of orifices in pipe was used. And this equation showed approximate relations of the heights in the weir and in the open canal in experiments. Then in the application of it to Neribeya diversion work, 6 simultaneous differential equations could be achieved, and were solved by Runge Kutta Gill methods.

Before stabilization, this model showed any vibrations by some reasons that should be expalained by the simulation of more complicated surging model for comparison.

But after stabilization this model showed the moderate regulation by the stop plate of the diversion system and gave much informations to design and examine the system without complicated surging model that needs very short interval of time step and much labour and cost.

Table 4 Coefficient of parabolic curve by regression analysis

branch name	a0	C0	C1	C2
TENMA	0.20	0.00239	0.31806	−0.41145
MORIYASU	0.23	0.08982	1.12719	−0.13925
INAMI	0.23	0.04078	0.35086	−0.30543
KAKO	0.25	0.07746	1.27486	−0.60201
TOTAL		0.21045	3.07090	−2.45814

Table 5 Example of variation of Dh/DH and DQ/Q by simulation

MIN.H	MAX.H	DH	MIN.Q	MAX.Q	DQ/Q	Dh(MORIYAMA)/DH
0.419	0.496	0.077	1.066	1.129	0.032	0.06
0.440	0.493	0.053	1.086	1.127	0.037	
0.474	0.506	0.032	1.114	1.135	0.020	
0.506	0.541	0.035	1.135	1.152	0.015	

(m・sec.)

7. Acknowlegement

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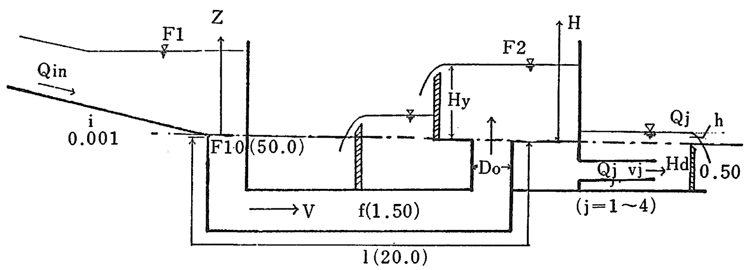

Fig. 5 plan of simulation model and variables

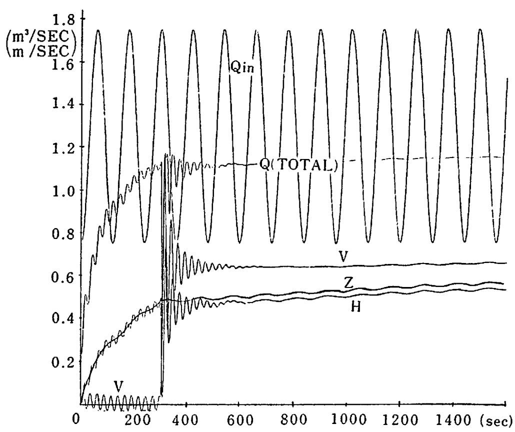

Fig. 6 Results of simulation by basic model part 1

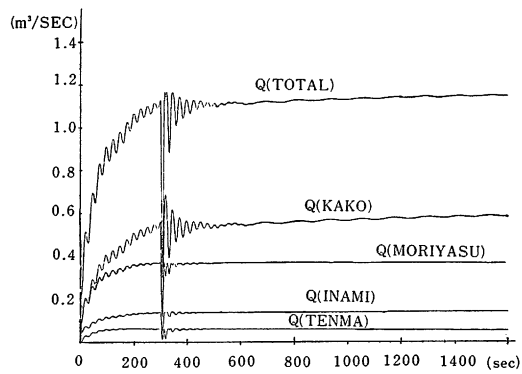

Fig. 7 Results of simulation by basic model part 2

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The notations of operator of formula are subjected its of FORTRAN.

Caluculations were done by M140F of Shimane Univ. M200 of Kyoto Univ. and micro-computer FM8, which could transfer data each other.

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摘　　要

浮力を用いた自動定量分水装置の制御部の水理を評価し，表現するために管内オリフィスの式を用いた．これは実験によって得られた分水セキの越流水深と用水路水位の対応関係に近いものとなった．これを東播用水練部屋分水工に適用するにあたり，6元一次の連立常微分方程式でモデル化するときに組み込んで基礎的数学モデルをつくり，ルンゲ，クッタ，ギル法で解くことができた．このモデルは各支線の分水量を実験的結果を下にして関数近似しているため，関数の形によって急勾配の領域の下では，越流水深の増減による分水槽内の貯留とこれによって生じる時間差の十分な反映がないため，不安定な振動を生じる．分水システム自体の応答系についての検討も必要であるが，サージング系の相互干渉を考慮して，多岐支線を有する分水システムをモデル化し計算するときの労力を考えると，緩やかな安定条件の下では，流れの規制を知るうえに有用であると考える．またこのモデルによるシミュレーションは，分水システムの水理とりわけ計画最大流量の設計と検討に有用であると考える．