剛性數理 모델에 의한 파이프라인系의 서어징 解析

Analysis of Surging in Pipeline Systems Using Rigid Water Column Model

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要 約

開放式 管水路는 낮은 压力으로 多量의 用水를 輸送할 수 있는 장점을 갖고 있으나 流量變動에 의한 서어징現像이 현저한 것이 단점이다. 管路内의 흐름을 安定시키기 위해서는 이 서어징의 特性이 究明되어 대규모의 서어징에 대한 対策이 講究되어야 할 것이다.

開放式 管水路系의 서어징을 剛性水柱理論으로 計算하기 위하여 運動方程式, 運續方程式과 스탠드 水槽의 中間壁에 設置된 堰의 越流公式等을 組合한 基礎方程式이 유도되었다. 本 研究의 数値解析모델은 가장 一般的인 4次의 Runge-Kutter 方法을 使用하였으며, 이 모델의 正當性과 프로그램의 有用性을 検証하기 위하여 水理模型実験値와 数値解析値가 比較 되었다. 그 結果 管路에 空氣의 混入이 없는 경우에는 實驗値와 解析値가 実用上 支章이 없는 程度로 잘 一致되었지만, 空氣의 混入이 發生되는 경우에는 實驗値가 解析値에 비해 약간 크게 나타나서 이 경우에도 서어징의 解析이 可能한 새로운 모델의 開發이 필요한 것으로 생각된다.

또한 本 剛性水柱 모델을 利用하여 現在 서어징 問題로 困難을 받고 있는 日本 滋賀県 琵琶湖 부근의 用水幹線을 対象으로 그 서어징의 特性과 改善方法을 検討한 結果 既設 開放式管水路系의 스탠드 중 每3個所 스탠드마다 1個所 스탠드의 下流側 水槽 水面積을 擴張하는 것이 妥當性이 있는 것으로 解析되었다.

I. Introduction

Water demands have been highly expanding as time passes by the industrialization and multilateralization of human life, whereas the water resources is limited. So it is a matter of great importance how to distribute and use efficiently this limited water resources. Particularly the effective utilization of irrigation water which forms the absolute quantity of total water demand is becoming an urgent problem.

Pipeline system has several advantages com-

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pare to open channel system. For instance, the effective utilization of water resources, usually easy to obtain the wayleaves and conservation of water quality, etc.

In case of Japan, during the 1960's, a rapid progress of industrialization led to the concentration of young labour force in cities and the population in village decreased greatly. Against this back ground, the use of pipeline for irrigation which can save the labour force introduced quickly. In the second half of 1960's the introduction of pipelines, which had been limited to the terminal irrigation blocks, started for the trunk canals, too. One of these reasons was to maintain the water quality in suburbs. Nowadays, nearly half of the new construction projects or repairing of irrigation canals are adopting the pipeline systems. Since it is very difficult and expensive to acquire the new land for irrigation, pipelines are more economical than open canals in most cases. In case of Korea, nearly almost of agricultural land is irrigated by open canals and pipeline irrigation is seldom used. However, as the case of Japan, it would be necessitated to adopt pipeline system for irrigation as time passes. Irrigation to the agricultural land is differ from the urban waterworks in the aspect of its consumption, and usually some difficulties ate followed in its maintenance and management. In the system of urban wataer works, once flow starts in the line, the line can almost be kept in full except some unexpected incidents or troubles, and also fluctuation of discharge rates is relatively small. On the contrary, in the agricultural pipeline system, air can be accumulated in water lines which may give rise to a variety of problelms, since the stoppage of flow or changes of flow rates are repeatedly operated.

Pipelines are usually classified into the next three types in its structures, that is, open type, semi-closed type and closed type. In open type pipeline systen, overflow stands are installed according to its head and line distance, so control the pressure in the line and distribute the water. Hence the line can be contacted to the atmosphere in relatively short distances, so hydraulically can be thought as the connection of inverted siphons, and its function as a water conveyance facility have characteristics which are similar to the open channel. In open type pipeline system, surging caused by the change of flow rate is distinguished.

Transients in pipelines are normally classified into the next two categories. Slow motion mass oscillation of water is referred to as surge, whereas rapid change in flow rate which is accompanied by elastic strain of water and pipe is referred to as water hammer. When water level of stand rises highly by suring, water overflows from the stand, and on the contrary, when water drops largely, air is entrained in the down stream pipeline which makes major problens in its maintenance and management. Hence it is claimed to pay attention in designing the stands.

To stabilize the state of flow in open type pipeline, characteristics of suring should be elucidated and it is necessary to devise a countermeasure against a large scaled surge caused by the increase or decrease of flow rates. In the analysis of transient flow of water line, the elastic water hammer theory which considers the changes of volume by pressure of water is adopted for the calculation of water hammer, and the rigid water column surge theory which neglects the compressibility of water and the elastic effect of the pipeline itself is used for the calculation of surging. And the range of application of rigid water column theory is limited to the relatively transient phenomena.

It is normally easier to analyse a pipeline system by figid water column theory, where-ever it is applicable, than by elastic theory. A pressure difference applied across the ends of the column produces an instantaneous acceleration. The basic equation relating the head difference between the ends of the water column to the rate of change in its velocity can be derived from the Newton's law of motion. Rigid column equation is useful for the calculation of water level variations of a surge following the starting up in a pumping line.

H.Shiraishi & K.Iwasaki(1973)11) developed a mathematical model for the simulation of unsteady flow in closed conduits, and this computer oriented simulation technique was capable of handling problems of hydraulic phenomena in an extremely short time and of choosing an optimum scheme from various alternatives.

K.Onizuka(1977,1981,1982)8,9,10) presented a new systematic approch to the anslysis and prediction of complicated surges unavoidable in the large-scale branching pipelines and systemized the analysis of transient phenomena by rigid column model.

Various problems have been identified in many open type pipeline systems used extensively in Japan. The problem of surging is inherent in a pipeline system composed of many stands and it is important to secure the safety of pipeline. From the point of this view, H. Yoshino, et al(1984)14) developed a simulation model appicable to the analysis of surging in a pipeline system in order to facilitate an easier designing.

The authors aimed to introduce the mathematical rigid water column model developed for this study and a main irrigation pipeline system in Lake Biwa area in Shiga-ken, Japan, which had problems of surging in water managment was practically studied and its surging characteristics and improvement ways were investigated by rigid water column model.

Ⅱ. Rigid water column simulation model

1. Outline of simulation model

1.1 Systems of fundamental equations

For the calculation of surging in pipeline system, equation of motion is considered to single pipeline section which connects the stands, thus the systems of fundamental equations can be framed by the conbination of equation of motion, continuity equation which expresses the continuity of flow discharge of each stand, and equation of weir installed at the overflow baffle wall of stand.

The fundamental equation can be derived from the pipeline system described in Fig. 1. Equation of motion of two pipe sections and continuity equation on the four stands are expressed as follows.

$$\left.\begin{aligned}
\frac{dZ_1}{dt} &= (A_1V_1 - Q_{p1})/F_1 \\
\frac{dV_1}{dt} &= (Z_2 - Z_1 - C_1|V_1|V_1)/(L_1/g) \\
\frac{dZ_2}{dt} &= (Q_{s1} - A_1V_1)/F_2 \\
\frac{dZ_3}{dt} &= (A_2V_2 - Q_{s1})/F3 \\
\frac{dV_2}{dt} &= (Z_4 - Z_3 - C_2|V_2|V_2)/(L_2/g) \\
\frac{dZ_4}{dt} &= (Q_{s2} - A_2V_2)/F_4
\end{aligned}\right\} \cdots(1)$$

Where ; Vi : flow velocity of number i pipeline, Li : length of number i pipeline, Ai : cross sectional area of number i pipeline, g : acceleration of gravity, Zi : distance from the datum line to the water surface of number i stand (upwards from the datum line is negative, and downward

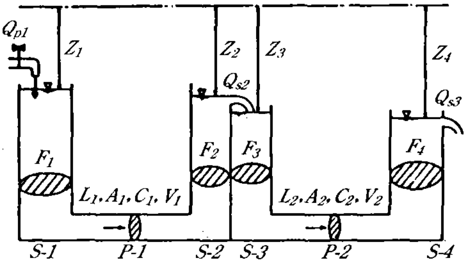

Fig. 1. Outline of the examined pipeline system.

is positive), t : time, Fi : water surface area of number i stand, Ci : coefficient of number i pipeline.

Furthermore, Qp1 is a discharge from the valve into No. 1 water tank,and Qs1 & Qs2 are discharges through the overflow weir, respectively.

From the Manning's formula, the coefficient Ci can be calculated by the following equation.

$$C_i=(f_i+f_o+\frac{124.5n_i^2}{D_i^{1/3}}\cdot\frac{L_i}{D_i})/2g \quad \cdots\cdots\cdots(2)$$

Where ; fi : loss coefficient at an entrance, fo : loss coefficient at an exit, Di : diameter of pipeline, ni : coefficient of roughness.

1.2 Mathematical analysis of fundamental equation and its programming

For the mathematical analysis of equation system expressed in equation(1), Runge-Kutter method of the fourth order which is the most generally used is adopted. Runge-Kutter method of the fourth order on the plural simultaneous ordinary differential equation is integrated by the following equstion.

If the given differential equation is
$Y'n=fn(X,Y_1,Y_2,\cdots,Y_m)$, $n=1,2,\cdots,m$
and its initial condition is $Y_n(X_0)=Y_{n,0}$ (3)
Then the integral equation to equation (3) is expressed as follows :

$$K_{n,1}=h\cdot f_n(X_i, Y_{1,i}\cdots,Y_{m,i})$$
$$K_{n,2}=h\cdot f_n(X_i+0.5h,\ Y_{1,i}+0.5K_{1,1}\cdots,Y_{m,i}+0.5K_{m,1})$$
$$K_{n,3}=h\cdot f_n(X_i+0.5h,\ Y_{1,i}+0.5K_{1,2}\cdots,Y_{m,i}+0.5K_{m,2}) \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots(4)$$
$$K_{n,4}=h\cdot f_n(X_i\ h,\ Y_{1,i}+K_{1,3}\cdots,Y_{m,i}+K_{m,3})$$
$$K_n=(K_{n,1}+2K_{n,2}+K_{n,3}+K_{n,4})/6 \quad X_{i+1}=X_i+h$$
$$Y_{n,i+1}=Y_{n,i}+K_n,\ {}_{n=1,2,\cdots,m}$$

Where ; kn : increment, kn,1 : the first increment, kn,4 : the fourth increment, h : time interval

When the ordinary differential equation (1) derived by the symbols expressed in Fig. 1 is looked into again, it can be found that the form of continuity equation is divided by the next two group which are dZ_1/dt & dZ_3/dt on the S-1,S-3 stand and dZ_2/dt & dZ_4/dt on the S-2,S-4 stand. So the equation of motion forms the same type. And if we have an eye on the subscripts of each variables in equation of motion of pipeline P-2, the uniformed ones in the subscript 2 of V_2 are term of loss, and the length of pipeline, and the pressure head in pipeline with subscript 2 is $(Z_4\text{-}Z_3)$. Therefore it is difficult to obtain some mechanical relationships between the subscripts.

The basic formation of each stand is using overflow weir, and consists of upstream & downstream water tanks. When a single water tank like surge tank is assumed to have a dummy water tank, Fig. 1 could be changed to Fig. 2.

In Fig. 2, upstream water tank of S-1 stand and downstream water tank of S-3 stand are dummy water tanks. So the previous equation (1) can be rewritten to the following equation (5) using the symbols expressed in Fig. 2.

$$\frac{dZ_{d1}}{dt}=(A_1V_1-Q_{P1})/F_{d1}$$

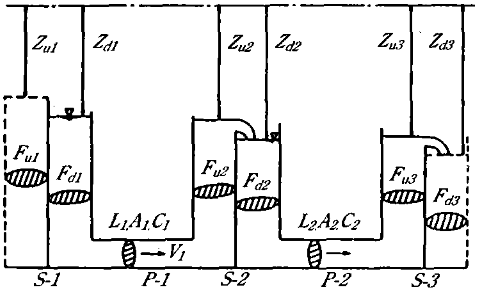

Fig. 2. Outline of the simulation model.

$$\left.\begin{array}{l}\dfrac{dV_1}{dt}=(Z_{u2}-Z_{d1}-C_1|V_1|V_1)/(L_1/g)\\[2ex] \dfrac{dZ_{u2}}{dt}=(Q_{s2}-A_1V_1)/F_{u2}\\[2ex] \dfrac{dZ_{d2}}{dt}=(A_2V_2-Q_{s2})/F_{d2}\\[2ex] \dfrac{dV_2}{dt}=(Z_{u3}-Z_{d2}-C_2|V_2|V_2)/(L_2/g)\\[2ex] \dfrac{dZ_{u3}}{dt}=(Q_{s3}-A_2V_2)/F_{u3}\end{array}\right\}\cdots(5)$$

So the way to add the subscripts into equation (5) is quite mechanical compare to the previous equation (1). For instance, the subscript 2 in equation of motion on the pipeline P-2 can be expressed with the subscript (i) as the following equation (6), so the quation of motion can be expresses by changing the value of subscript (i) form 1 to n.

$$\frac{dV_{(i)}}{dt}=(Z_{u(i+1)}-Z_{d(i)}-C_{(i)}|V_{(i)}|V_{(i)})/\ (L_{(i)}/g)\ \cdots(6)$$

Continuity equation can also be presented as the following equations by dividing each atand into upstream & downstream water tanks. For the upstream water tank of S-1 stand, when the subscript (1) is added, the continuity equation can be expressed as follow.

$$\frac{dZ_{u(i)}}{dt}=(Q_{s(i)}-A_{(i-1)}\cdot V_{(i-1)})/F_{u(i)}\qquad\cdots\cdots\cdots(7)$$

And for the same type of downstream water tank, the continuity equation can be expressed as follow.

$$\frac{dZ_{d(i)}}{dt}=(A_{(i)}\cdot V_{(i)}-Q_{s(i)})/F_{d(i)}\qquad\cdots\cdots\cdots(8)$$

When we observe the way of using symbols presented in Fig. 2, it can be notified that the systems of equation can be automatically framed by indicating the type of stand through data input.

The mathematical simulation model of this study is employing the Runge-Kutter method of the fourth order as presented in equation (4). On each differential equation in equation (4), the first increment at time xi, the second & third increment at time $xi+h/2$ and the fourth increment at time $xi+h$ is calculated as shown in kn,1−kn,4 and their weighted average kn is calculated. Number of these equations are combination of continuity equation of Ns(number of water tanks) and equation of motion of Np(mumber of pipelines), and as mentioned previously, these equations can be classified into three types of fundamemtal equations. So the simulation program is framed by the association of array variables and DO loops.

The program consists of one main program and nine subroutine programs and its flow charts are presented in Fig. 3(a), (b). The function of each subroutine program is briefly explained in accordance with the flow chart as follows.

Subroutine INITIAL performs an initial clearance of variables used, and as a next step, subroutine INPUT inputs the required data. The initial coditions are created in subroutine SET as follows. First, the primary water level of last stand is calculated by the following equation (9).

$$Zu(n)=(Q/B\cdot Cw)^{2/3}+Cr\qquad\cdots\cdots\cdots\cdots\cdots\cdots(9)$$

Where ; Zu(n) : the primary water level of last stand, Q : initial discharge(m^3/sec), B : wi-

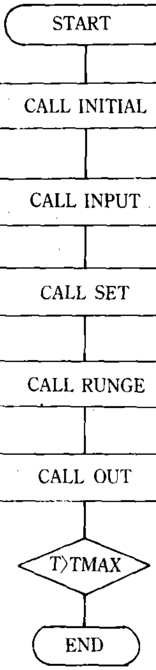

Fig. 3(a). Flow chart of main program.

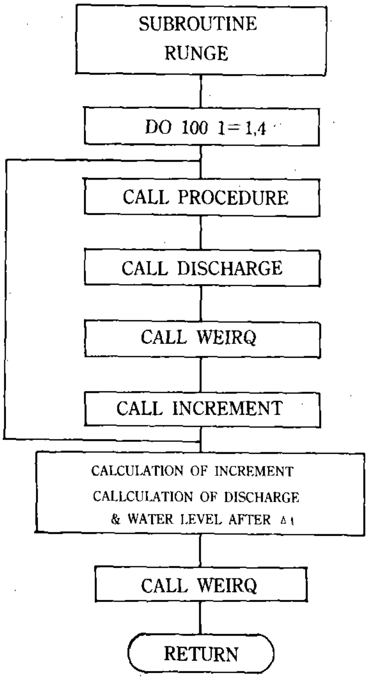

Fig. 3(b). Flow chart of subroutine program RUNGE.

dth of weir (m), Cw : coefficient of weir, Cr : crest level of weir

In equation (9), the coefficient of weir (Cw) is calculated by the following equation (10)[7].

$$Cw = 1.785 + (0.00295/h + 0.237h/D) \cdot (1+\varepsilon) \quad \cdots\cdots (10)$$

Where ; h : overflow depth (m), D : height of weir from the bottom of stand, ε : correction term (if $D \leqq 1m$, $E=0$ and if $D > 1m$, $\varepsilon = 0.55(D-1)$).

Second, the secondary water level of the stand (Zd(n-1)), that is just ahead of the last stand, is calculated by using Zu(n) and equation (6). And at this time, $dV(n-1)/dt$ is equal to zero. Third, the primary water level of the stand, that is just ahead of the last stand, is calculated. So the same procedures are repeated upto the first stand in this way.

The subroutine RUNGE practices integral calculus. The first subroutine PROCEDURE arranges the initial values of time, velocity, and water level when the calculation of concerned increment starts prior to the entering time interval h by the following steps. In the first step, i=1, $t=t_o$, V=V(i,0), Zd(i)=Zd(i,0) and Zu(i)=Zu(i,0). And in the second step, i=2, t=to+0.5Δt, V(i.1)=V(i,0)+0.5ΔV(i,0), where ΔV is calculated in subroutine INCREMENT. In this step, Zu(i,1)=Zu(i,0)+0.5ΔZu(i,0), Zd(i,1)=Zd(i,0)+0.5ΔZd(i,0), where ΔZu(i,0)&ΔZd(i,0) are also calculated in subroutine INCREMENT. In the third (i=3) and the fourth step(i=4), calculations are fulfilled through the same procedures according to equation (4).

The following subroutine DISCHARGE calculates the dicharge through the inlet valve at upstream end, and the discharge through overflow weir and distribution work at each time increment are calculated in subroutine WEIRQ.The discharge through overflow weir is calculated by the following equation(11)

$$Qs(i)=Cw\cdot Bi(Zu(i,n)-Cri)^{3/2} \quad \cdots\cdots\cdots(11)$$

When subroutine INCREMENT practices integral calculus on the previously mentioned equation (6), (7) and (8), increments of each step are calculated. As a next step, the increment kn at time intervral h is calculated by weighted averaging method, and gets the starting value of next step. Afterwards, subroutine WEITQ is called again to provide for the output of subrouting (LINE) in main program. This subroutine OUT is practiced by time interval indicated by the input data.

2. Verification of simulation model by hydraulic model test

2.1 Hydraulic model test

To verify the rigid water column simulation model developed in this study and the efficiency of its programming, values of surging obtained from the hydraulic model test and those simulated by mathematical analysis were compared. The experimental hydraulic model used in this study consists of three overflow type stands, and the model was built to investigate the general characteristics of water tanks when there is no air entrainment by the falling nappe.

The outlined figure of experimental hydraulic model is shown in Fig. 4 and photo. 1. The model was installed at the lab. of canal hydraulics, Japanese National Research Institute of Agricultural Engineering. It was constructed with transparent acrylplates which have a thickness of 10 mm, and its slope was 0.025.

The values used for the mathematical analysis are as follows ; coefficient of roughness (n) : 0.012, loss coefficient at the entrance(fi) : 0.5, loss coefficient at an exit(fo) : 1.0.

Flow discharge was regulated by the operation of control valve installed at the upstream end, and the electronic point gauge was unstalled at the water tank of No. 2 stand to measure the change of water levels. The designed discharge

Photo.1. Hydraulic experimental model.

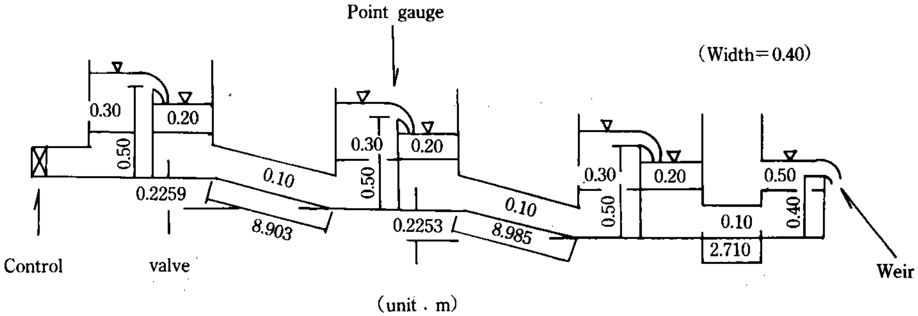

Fig. 4. Outlined figure of experimental hydraulic model.

for the experiment was controlled by the electro-magnetic valve at the upstream end, and by the rectangular weir installed at the downstream end of the hydraulic model. Four different case studies were considered in the hydraulic model test according to the opening or closing time of the electro-magnetic valve as shown in Table 1, and each case studies was performed repeatedly. In case No. 2, the length of downstream water tank was changed from 0.2m to 0.1m to allow the air entrainment in downstream pipeline easily.

Table-1. Case studies in hydraulic model test.

case No.	Initial disch.	Changed disch.	Valve opera.	Valve op. time	Remarks
1	6.18 ℓ/s	0.0 ℓ/s	close	2 sec	Instantaneous closing from max. discharge
2	3.95	0.0	close	2 sec	Instantaneous closing under air entrainment
3	0.0	6.18	open	3 sec	Instantansous opening from max. discharge
4	0.0	6.18	open	10 sec	Gradual opening from max. discharge

2.2 Comparison of the simulated and actually measured surging

The results of simulated and measured values of surging in each case studies are shown in Fig. 5, 6, 7, and 8, respectively.

(Case 1)

The maximum suring was Simulated in this case study, so the valve was closed instantaneously (for two seconds) from the maximum flow rate of 6.18 liter/sec. As shown in Fig. 5, the simulated value almost coincided with the measured value for initial 100 seconds after valve closing, howrver the measured value declined earlier compare to the simulated value by the time lapse. For this reason, changes of the viscosity of water and the coefficient of roughness could be considered.

(Case 2)

Air entrainment in the line was occurred in this case study. The value measured by hydraulic model test did not well coincided with the value simulated by rigid water column model. The measured surging was about 10% stronger than the simulated value as shown in Fig. 6.

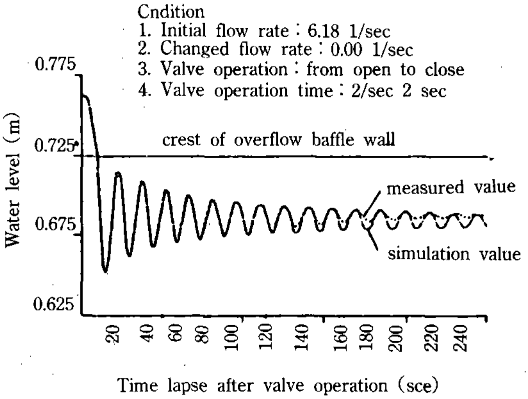

Fig. 5. Surging phenomena in No. 2 Stand (case 1).

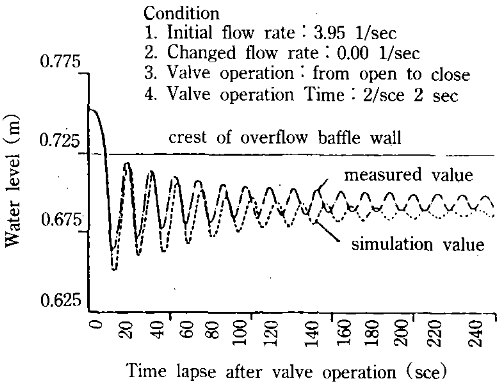

Fig. 6. Surging in No. 2 stand under air entrained condition in the line (case 2).

This result well indicates the limitation of simulation model.

(Case 3&4)

Valve opening time from the closed condition was considered in these two case studies. Valve opened instantaneously (for three seconds) in case 3,and opened gradually (for ten seconds) in case 4. The results showed a little difference as seen in Fig. 7, 8. It is because the flow discharge through the valve was changed proportionally by the opening of valve in simulation model, whereas the practical opening of valve was difficult to operate compare to the valve closing.

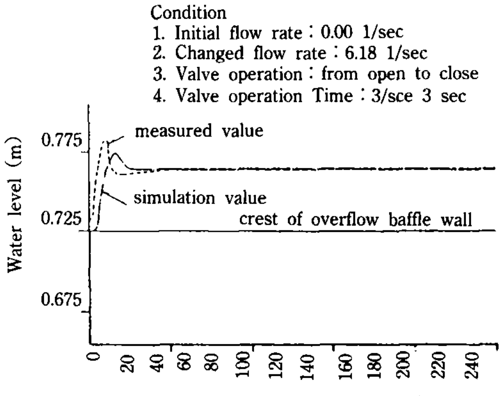

Fig. 7. Surging in No. 2 stand (case 3).

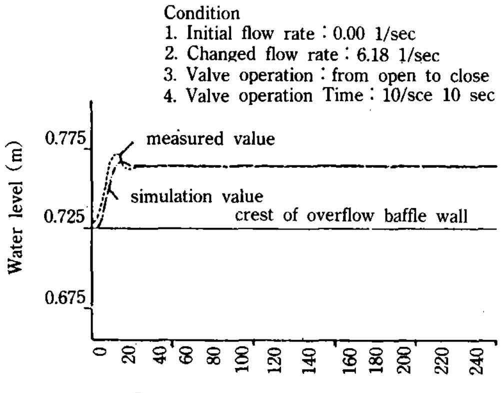

Fig. 8. Surging in No. 2 stand (case 4).

III. Practical analysis of surging on a main pipeline system in Aichi river area using rigid water column model

1. Oitlined figure of a studied pipeline system and its problem

The characteristics of surging were studied by rigid water column simulation model on a main open type irrigation pipeline system in Lake Biwa area, Shiga-ken, Japan (see Fig. 9) which has been in troubles of distinguished surging For instance, as shown in Fig. 10, the measured maximum suring of downstreanm tank reached more than 5m above the crest of overflow baffle wall, and because of this surging, flow rates through the main pipe and distribution works are very unstable. Furthermore water sometimes overflows from the wall of stand, and damages fields and roads.

Surging can be arised by the following two main factors, one is the inertia effect of water by the change of flow discharge rates, and the

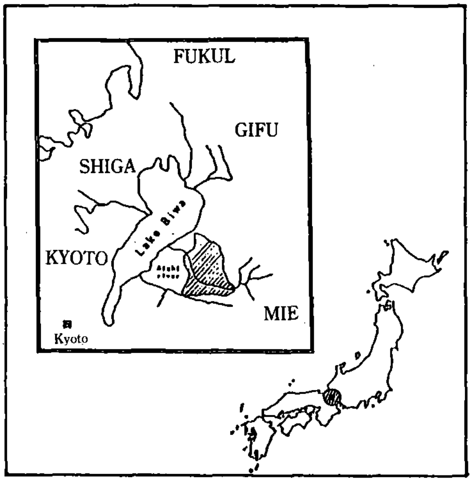

Fig. 9. Location map of studied pipeline system.

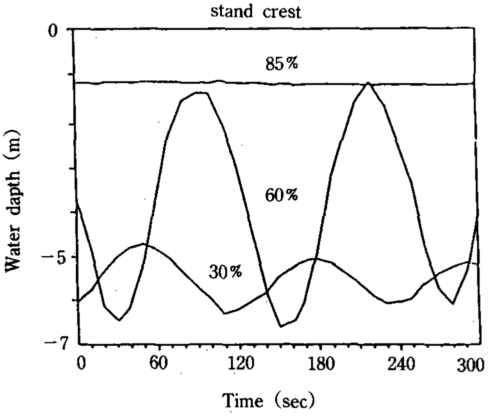

Fig. 10. Water level fluctuation In downstream tank of No. 7 stand(% means the ratio of designed flow rate).

other is the air entrained unsteady flow. The most effective way to prevent surging is enlarging the water surface area of stand. Because the studied pipeline system has been already constructed and it would· be very diffecult to enlarge all stands pracrically, it was thought that the practial and reasonable way is to enlarge certain specific stands concentratedly, so the simulation was performed under the consideration of this point.

As shown in Fig. 11. the upstream part of irrigation canal is open channel and from the downstream of Hirao distribution work is open type pipeline. The total lengrh of pipeline is 5.1km,

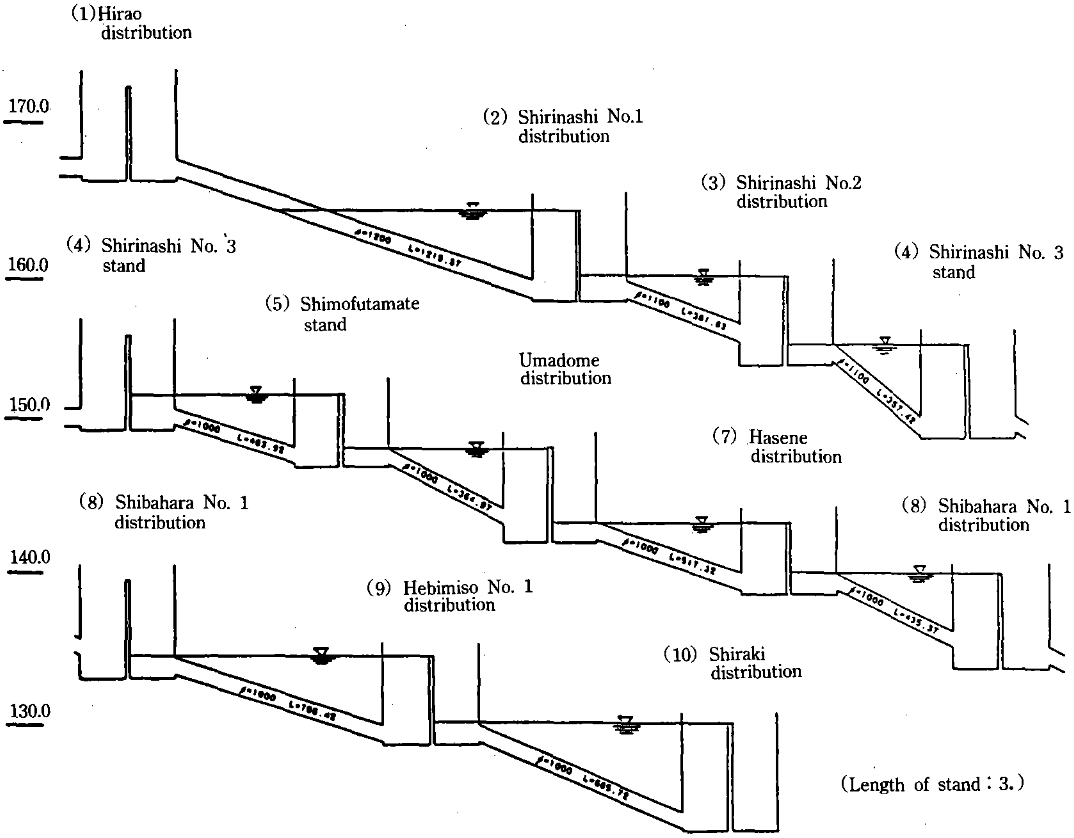

Fig. 11. Diagram of vertical section of stands.

and consists of ten stands including the upstream & downstream end stands. The designed maximum flow discharge is 1.949 m^3/sec at the upstream end, and 1.529 m^3/sec at the downstream end and the diameter of pipeline is 1200 mm−1000 mm. The size of each stand is 3 m (3D−2.5D) long, and 2.5 m−2.0 m (2.5D−2.0 D) wide, so it is quite small in its size compare to presently accepted stand size which is 4D−5 D (D is a diameter of pipeline) in both length and width [6,13]. In the pipeline system in series of such small scale of stands, once surging arised, even the changing time of flow rate is slow, it would be amplified by going dowm to downstream, and paticularly surging will be rapidly expanded after the fourth stand[14]. Therefore it is thought that water management will be in trouble by only small changes of flow discharge rate, even though the air entrainment in pipeline system can be completely protected by certain method except the enlargement of water surface area of stand.

As shown in Fig. 11, the length of pipeline between Hirao & Shirinashi No. 1 distribution work is relatively long and the difference of elevation is high, so the upstream side of pipeline is completely exposed to the atmosphere when flow discharge stops, and air could be entrained in the line when flow starts. Since this pipeline system is very hard to manage hydraulically, it is necessary to install new stand at an appropriate place in between Hirao & Shirinashi No. 1 distribution work to allow the downstream pipeline of Hirao stand to be filled with water when flow discharge stops.

2. Simulation of surging and its conditions

Surging of the studied open type pipeline system was simulated by rigid water column model in six different cases as presented in Table 2, and each case was simulated with three different discharge amounts. These were 100%, 60% (0.917 m^3/sec), and 30% (0.459 m^3/sec) of the designed discharge (1.529 m^3/sec), at the end of downstream, and it was considered that there was no distribution on the way.

In case 1,3 and 5, as shown in Fig. 12, flow discharge rate increased from no flow condition to objective discharge amount for 300 seconds which was according to the closing/opening time of values generally used. In case 2, 4, and 6 as shown in Fig. 13, it was considered that flow discharge rates were varied by the air entrain-

Table-2. Conditions for the simulation.

case	Discharge condition	stand condition	Remarks
case 1	increase	no change	increase : increase from no flow condition to object discharge for 300 sec fluctuation : $Q+0.05Q\cdot\sin\omega\cdot t$ $(\omega=2\pi/120)$
case 2	fluctuat.	no change	
case 3	increase	enlarge No. 4 & No. 7 stand	
case 4	fluctuat.	enlarge No. 4 & No. 7 stand	
case 5	increase	enlarge N0.5 stand	
case 6	fluctuat.	enlarge No.5 stand	

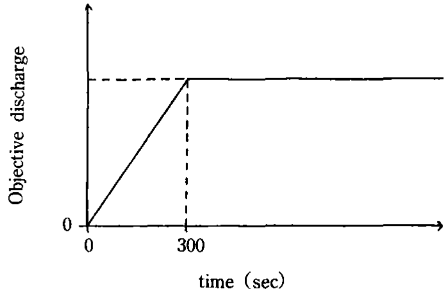

Fig. 12. Flow discharge condition of case 1, 3, 5.

ment in the line or by the operation of distribution works in upstream. As presented in Fig. 13, it was assumed that there was a change of flow rate with a sine curve which had an amplitude of 0.05Q (Q is the objective discharge) and a cycle of 120 seconds from the actually measured data of surging. In simulation case 1 & 2, no change of stand size was considered, and in case 3 & 4, it was supposed that the downstream tank of Shirinashi No.3 & Haseno stands, which is the fourth & sevevth stand, respectively, were enlarged to 48 m^2 (6.4 times of present water surface area). In simulation case 5 & 6, it was assumed that the downstream water tank of Shimofutamata stand, which is the fifth stand from upstream, was enlarged to 48 m^2. The enlarged water surface area of downstream tank was calculated by multiplying the length and width which are four times of pipeline diameter (D=1000 mm), respectively, and again multiplied by 3 which is representing the enlargement of one stand in every three stands.

3. Simulation results and discussion

The simulated results on the changes of water level of downstream water tank are presented in Fig. 14-27. Vertical axis of the figure shows the change of water level when the crest of overflow baffle wall is assumed as a datum line.

In case 1 & 2, the amplitude of surging was enlarged by going down to the downstream stands, and particulary in case 2, the surging at downstream stands under the flow rates of 60% & 30% showed larger than that of 100% of the designed flow rate by minor changes of flow discharge rate. That is to say, surging at ST2 (the second stand which is Shirinashi No. 1 stand, and so forth) under 100% flow rate condition showed 0.5m, and it was magnified to 1.3 m at ST6, 1.5 m at ST9, whereas that of ST2, ST6 and ST9 was enlarged to 0.35 m, 4.0 m and 8.3 m, respectively, under the flow rate of 60%. Also surging under the flow rate of 30% of the above stands reached to 0.3 m, 5.8 m, and 8.4 m, respectively.

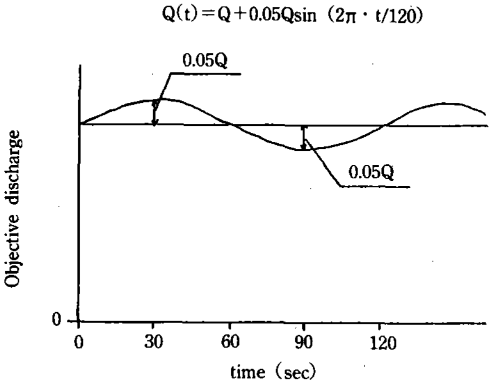

Fig. 13. Flow discharge condition of case 2, 4, 6

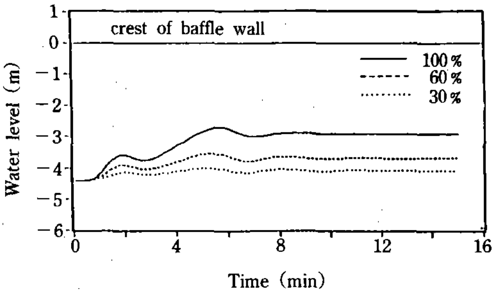

Fig. 14. Simulation result (case 1, ST2).

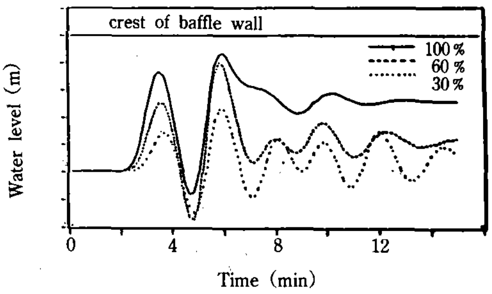

Fig. 15. Simulation result (case 1, ST6).

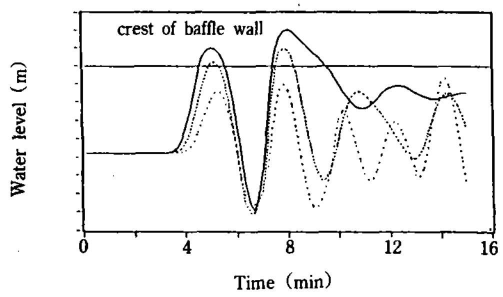

Fig. 16. Simulation result (case 1, ST9).

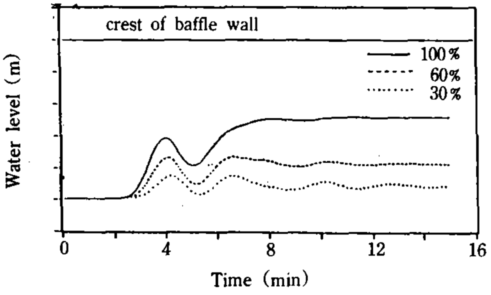

Fig. 20. Simulation result (case 3, ST6).

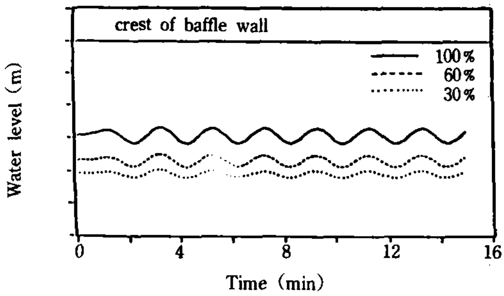

Fig. 17. Simulation result (case 2, ST2).

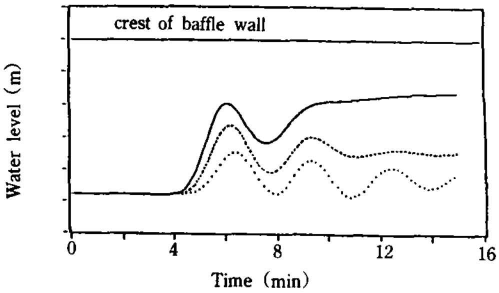

Fig. 21. Simulation result (case 3, ST9).

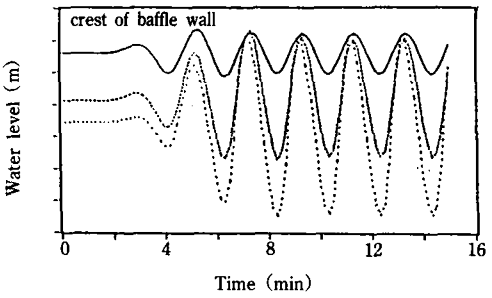

Fig. 18. Simulation result (case 2, ST6).

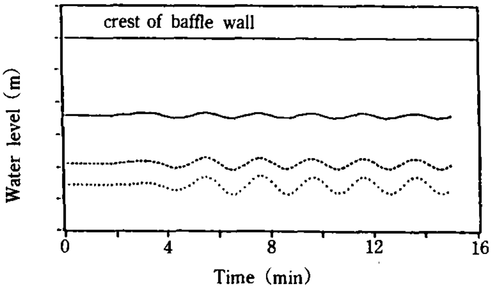

Fig. 22. Simulation result (case 4, ST6).

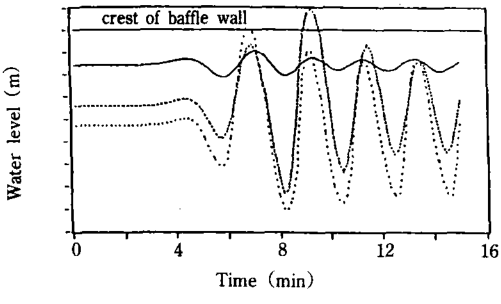

Fig. 19. Simulation result (case 2, ST9).

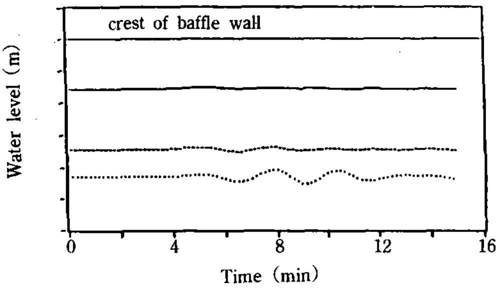

Fig. 23. Simulation result (case 4, ST9).

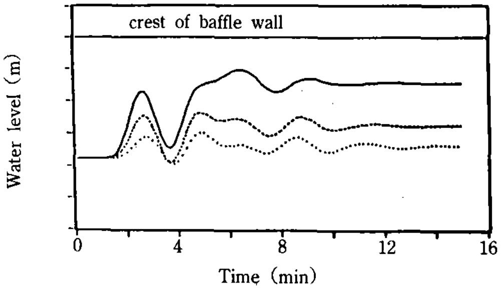

Fig. 24. Simulation result (case 5, ST4).

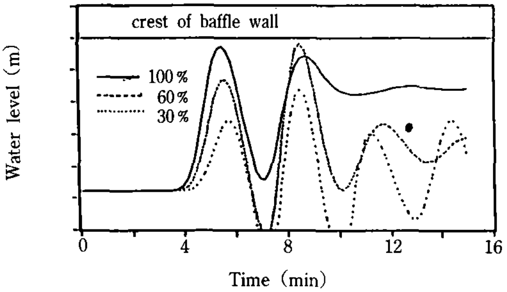

Fig. 25. Simulation result (case 5, ST9).

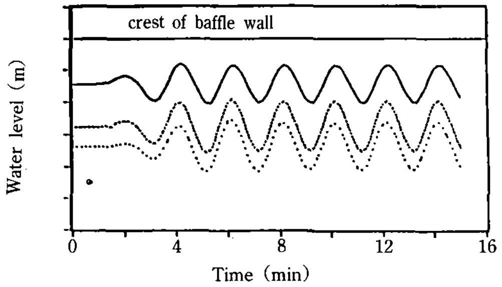

Fig. 26. Simulation result (case 6, ST4).

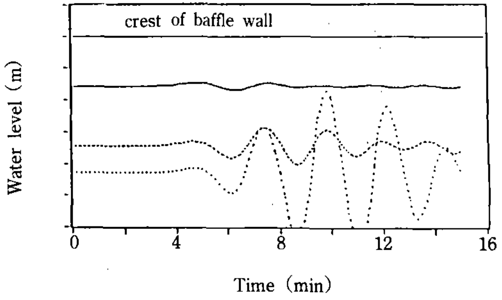

Fig. 27. Simulation result (case 6, ST9).

In this simulation study, the depth of each stand is assumed as unlimited, however the actual depth of each stand is limited. So if a simulated surging which is larger than the actual depth of stand arised, it would be larger than that of the measured. This was approved from the result which was actually measured at the pipeline system as shown in Fig. 10. The actually measured surging at downstream stands under the flow rate of 60% showed the maximum scale and the surging under the flow rate or 30% was small.

Air entrainment in pipeline system is one of the factors cause surging, but because the sumulated surging is too great, it is impossible to prevent surging even though the air entrainment in the pipeline system possibly be protected by certain countermeasures. Therefore the reasonable way to prevent surging in this case is to enlarge the size of stand.

The simulated results of case 3&4, which enlarged the water surface area of downstream tank of the 4th & 7th stand to 48 m^2, showed stable condition compare to the results of case 1&2 as shown in Fig. 20-23. For instance, surging at ST9 of case 2-2 (flow rate of 60%) reached as much as 8 m, whereas that of case 4-2 (flow rate of 60 %) was neary zero, and that of case 4-3 (flow rate of 30 %) was only 0.5 m. Also the smulated results of case 5&6, which enlarged the surface area of downstream water tank of the 5th stand to 48 m^2, were stabilized compare to case 1&2, however surging somthime reached to 6 m as shown in Fig. 24-27.

From the above results obtained, it is recommended to enlarge the size of stand in every three stands for the stable management of the studied pipeline system which has some problems of surging.

IV. Conclusions

For the calculation of surging in open type pipeline system using rigid water column theory, the systems of fundamental equations were framed by the combination of equation of motion, continuity equation and the equation of overflow weir. The simulation model of this study is employing the Rumge-Kutter method of the fourth order and the simulation program consists of one main and nine subroutines.

To verify the rigid water column model used for this study, values of surging measured by the hydraulic model test and those of simulated by the mathematical analysis method were compared. When the value at the upstream end of pipeline was closed instantaneously (for two seconds) from the maximum flow rate of 6.18 liter/sec, the simulated value of surging almost agreed with the measured value for initial hundred seconds after value closing, however the value measured by hydraulic model test declined earlier compare to the simulated value by the time lapse. Fof this reason, changes of the viscosity of water and the coefficient of roughness could be considered. And when the air was entrained in the line, the hydraulically measured result was not well coincided with the value simulated by rigid water column model, as the measure surging was about 10% stronger than that of the simulated. This results well indicate the limitation of simulation model used for this study. By this reason, it is necessary to develop a simulation model which can analyse surging under air entrained condition in the line.

The characteristics of surging were practically investigated by rigid water column surge simulation model on a main open type pipeline system in Lake Biwa area in Shiga-ken, Japan which has been in troubles of distinguished surging. It is generally known from the previous studies that the most effective way to prevent surging in the line is to enlarge the water surface area of stand. Because the studied pipeline system has been already built and it might be not so easy to enlarge the water surface area of all stands in the line, the practical and reasonable way is to enlarge certain specific stands. Once surging arised in actual pipeline system, although the changing time of flow rate is slow, it is usually amplified by going down to the downstream. Because surging is rapidly expanded afrer the fourth stand, the water management will be in trouble by small change of flow rates, although the air entrainment can be completely protected by certain methods except the enlargement of water surface area of stand.

In simulation case 1&2, the amplitude of surging enlarged by going down to downstream stands, especially in case 2, surgung under the flow rates of 60% & 30% showed larger than that of 100% of the designed flow rate by minor change of flow condition. The simulatied results of case 3&4, which enlarged the size of downstream tank of the 4th & 7th stand to 48 m^2, were clearly stabilized. Also the simulated results of case 5&6,which enlarged the 5th stand, were relatively stabilized, however in this case, surging sometimes reached to 6 m.

From the simulated results, it is seemed reasonable and practical to enlarge the size of one stand in every three stands for the stable and safe management of the studied open type pipeline system.

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