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CONTROL OF SURGING IN LOW-PRESSURE PIPELINES

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Engineering and Research Center
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16. ABSTRACT

Review and analysis of available information regarding surging in low-pressure pipelines indicated that such lines, designed to operate with the hydraulic gradeline roughly parallel to the ground surface at design discharge, will undergo a fluid motion which is underdamped when operating at flow rates below the design value. An analytical study of the dynamics of surging flow enumerated the significant design and operation parameters. Numerical values of these parameters were obtained and are tabulated in the report. Through use of these tabulated values, low-pressure pipelines may be designed to minimize the amplitude of flow oscillations.

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by

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UNITED STATES DEPARTMENT OF THE INTERIOR * BUREAU OF RECLAMATION
Rogers C. B. Morton, Secretary
Ellis L. Armstrong, Commissioner

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NOMENCLATURE

Symbol	*Definition*	*Dimensions**
$A_{1,2,3---}$	Cross-sectional areas	L^2
D	Diameter	L
f	Darcy-Weisbach resistance coefficient	
F	Frequency	T^{-1}
g	Gravitational acceleration	L/T^2
H	Total head	L
h_f	Head loss	L
k	Roughness height	L
$L_{1,2,3---}$	Pipe lengths	L
Q	Discharge	L^3/T
Q_o	Steady design discharge	L^3/T
Q_F	Final steady discharge	L^3/T
Q_I	Discharge entering a pipe reach	L^3/T
Q_m	Fluctuating discharge with period T_o superimposed on Q_s	L^3/T
Q_s	Steady – state discharge	L^3/T
R	Dimensionless resistance coefficient	1
S	Distance along pipe centerline	L
T_N	Natural period of reach	T
T_o	Period of superimposed fluctuation or period of first upstream reach	T
t	Time	T
V	Mean velocity	L/T
X	Horizontal coordinate	L
y_1	Depth of water at inlet	L
$y_1 max$	Maximum possible value of y_1	L
y_2	Depth of water at outlet	L
Δy	$y_1 \max - y_2$	L
ΣK	Combined resistance coefficient ($h_L = \Sigma K\, V^2/2g$)	
γ	Specific weight	F/L^3
ρ	Density	M/L^3

*M = mass; F = force; L = length; T = time

PURPOSE

This study was conducted in order to establish valid parameters for use in the design of low-pressure pipe systems by predicting the amplitude and frequency of possible unsteady flow occurring at flow rates less than the design value.

CONCLUSIONS

Pipelines designed to produce an hydraulic gradient roughly parallel to the ground profile are inherently underdamped when operating at less than design discharge. Thus, when operating at less than design flow rates, any unsteadiness in the flow may be amplified depending upon the characteristics of the pipeline system.

In order to assure stable flow, a pipeline system must be proportioned in such a manner that small disturbances in one reach will not be amplified upon entering a succeeding reach. To do this the natural periods of adjoining reaches must be sufficiently different from each other. A reach will not magnify incoming surges if it has a natural period greater than 1.43 times the period of the incoming disturbance. If a reach has a natural period which is less than 1.43 times the period of an incoming surge, the surge will be amplified unless sufficient resistance to flow exists within the reach.

The stability of a given reach for rapid changes in inflow rates has been tabulated graphically in terms of amount of cutback, rate of cutback, and resistance to flow. These results can be used to determine permissible rates of change of inflow which will prevent unstable surging.

APPLICATIONS

This report describes a study centered around experiences with the existing Coachella and Canadian River low-pressure pipe systems. The results of this study could be used directly in the design of low-pressure systems to control the amplitude of the free-surface fluctuations. Many studies have been made in the past to compare the results of one-dimensional analysis with actual flow conditions, and there appears to be little reason to doubt that actual conditions agree, to a reasonable degree, with analytically calculated results.

*Superscripts indicate references at end of report.

BACKGROUND

For the purposes of this report a low-pressure pipe distribution system is defined as one designed to produce an hydraulic gradient roughly parallel with the ground, with discharge controlled at the upstream end of the system. In order to prevent these pipe systems from draining completely when the discharge is shut off, check structures are constructed at intervals along the pipelines.

Typical check structures are illustrated in Figure 1. The pipe system carrying water from the Coachella Branch of the All-American Canal to the Coachella Valley, and the Main Aqueduct of the Canadian River Project in west Texas, are well-known examples of low-pressure pipe distribution systems. Although the former system carries water for irrigation, and the latter for municipal purposes, the dynamic characteristics of the two systems are basically the same.

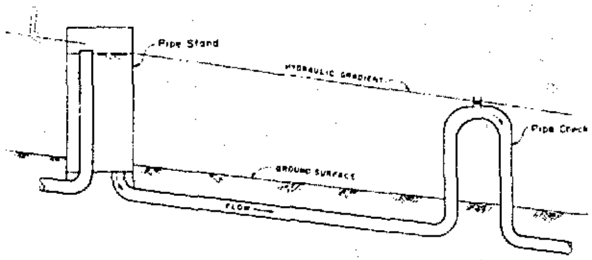

Figure 1. Typical pipe-check and pipe-stand structures.

When the Coachella system was placed in operation in 1948, unsteady flow and resulting operational difficulties arose almost immediately. Although inflow to the system was steady, the flow in some sections became quite unsteady, sometimes developing amplitudes in discharge sufficiently large to cause overtopping of check structures. Subsequent laboratory study suggested two solutions: airtight covers for part of the structures to change the nearly resonant dynamic characteristics of the system,[1] * and air vents downstream from the check structures to provide a means for escape of air entrained in the overfall process at the check.[2]

When the Main Aqueduct of the Canadian River Project was placed in operation in 1968, operation was found to be smooth and trouble free for the design flow rate. However, smooth steady flow was difficult to establish for rates below the design value. Overtopping was noted at three structures when the flow rate was decreased to a final discharge substantially below the design value.

As a result of the experiences on the Coachella and Canadian Rivers, considerable study was initiated, some of which has already been noted,[1,2,3,4]. This study represents an attempt to analyze the problem of unsteady flow in low-pressure pipe distribution systems as a problem in dynamics. A review was made of all available published and unpublished results in order to arrive at representative system parameters and to isolate at least a portion of the problem which had not been thoroughly studied. A summary of previous work is incorporated where the particular work seemed to fit into the current analysis best.

The results of the analysis are set forth in a form which should be useful to designers interested in the analysis of unsteady flow in a low-pressure pipe distribution system.

All results have been computed and displayed in a dimensionless fashion in order to completely generalize the analysis and results. The dimensionless parameters have been defined and can readily be computed from the dimensional properties normally used by designers.

Not all possible combinations of parameters have been included in these numerical results. However, a listing of the program is included in the appendix and can readily be used to make further analyses as required.

EQUATIONS OF MOTION

Development

The equations of motion have been correctly formulated by Glover[1] and by Holley[4] but are reformulated here because they provide the necessary means of systematically analyzing the dynamic characteristics of the pipe distribution system. Figure 2 illustrates the variables involved. Either Newton's second law, F = m (dV/dt), or the unsteady form of the Bernoulli Equation:

$$\frac{\partial V}{\partial t} = -g\,\frac{\partial H}{\partial s} \qquad (1)$$

may be used in the derivation[5]. In Equation (1) V is the average velocity, t is time, g is gravitational attraction, H is total head, and s is distance along the

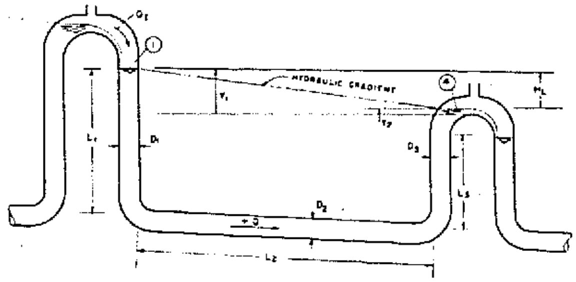

Figure 2. Definition sketch for flow in a pipe reach.

pipe. If it is assumed that flow is one dimensional (velocity is everywhere parallel to the pipe walls) and that elastic effects are unimportant, Equation (1) can be integrated over the volume of liquid contained in the pipe reach of interest:

$$\int_1^4 \frac{\partial V}{\partial t}\,ds = -g\int_1^4 \frac{\partial H}{\partial s}\,ds \qquad (2)$$

where H^1 and H^4 are, respectively, the total heads at Points 1 and 4 on Figure 2. In order to integrate the left side of Equation (2), reaches of constant diameter must be considered. Thus,

$$\int_1^2 \frac{\partial V_1}{\partial t}\,ds + \int_2^3 \frac{\partial V_2}{\partial t}\,ds + \int_3^4 \frac{\partial V_3}{\partial t}\,ds = -g(H_4 - H_1) - gh_L \qquad (3)$$

where h_L is the loss in head through the pipe. But from continuity

$$Q = V_1A_1 = V_2A_2 = V_3A_3 \qquad (4)$$

and $$\frac{\partial V_1}{\partial t}A_1 = \frac{\partial V_2}{\partial t}A_2 = \frac{\partial V_3}{\partial t}A_3$$

so that $$\frac{\partial V_1}{\partial t} = \frac{\partial V_2}{\partial t}\frac{A_2}{A_1}$$

and $$\frac{\partial V_3}{\partial t} = \frac{\partial V_2}{\partial t}\frac{A_2}{A_3} \qquad (5)$$

Incorporating Equation (5) in Equation (3) gives

$$\frac{\partial V_2}{\partial t}\left\{\int_1^2 \frac{A_2}{A_1} ds + \int_2^3 ds + \int_3^4 \frac{A_2}{A_3} ds\right\} =$$

$$gH_1 - gH_4 - gh_L$$

The pressure is zero at both 1 and 4 so that

$$H_1 = \frac{V_1^2}{2g} + Y_1 \text{ and } H_4 = \frac{V_1^2}{2g} + Y_2$$

$$\text{or } A_2 \frac{\partial V_2}{\partial t}\left[\frac{L_1}{A_1} + \frac{L_2}{A_2} + \frac{L_3}{A_3}\right] = g(H_1 - H_4 - H_L) \quad (6)$$

Letting $Q = V_2A_2$, $\Sigma L/A = \frac{L_1}{A_1} + \frac{L_2}{A_2} + \frac{L_3}{A_3}$,

and $h_2 = \Sigma K \frac{V_2^2}{2g}$

we obtain

$$\frac{dQ}{dt} = \frac{g}{\Sigma L/A}\left(\frac{V_1^2}{2g} + Y_1 - \frac{V_4^2}{2g} - Y_2 - \frac{\Sigma K V_2^2}{2g}\right) \quad (7)$$

realizing that Q is a function only of time. Using Equation (4) again we can write Equation (7) as

$$\frac{dQ}{dt} = \frac{V_2^2}{2\Sigma L/A}\left[\left(\frac{A_2}{A_1}\right)^2 - \Sigma k - \left(\frac{A_2}{A_3}\right)^2\right] +$$

$$\frac{g}{\Sigma L/A}(Y_1 - Y_2) \quad (8)$$

Now examine the quantity within brackets in Equation (8):

$$\Sigma K = f_1 \frac{L_1}{D_1}\left(\frac{A_2}{A_1}\right)^2 + f_2\frac{L_2}{D_2} + f_3\frac{L_3}{D_3}\left(\frac{A_2}{A_3}\right)^2$$

using the Darcy-Weisbach form of resistance. For all structures on the Canadian River Project $A_2/A_3 = 1$.

For all pipe checks $A_2/A_1 = 1$, and for all pipe stands A_2/A_1 is less than 0.125. Table I (next page) lists values of ΣK for all Canadian River structures. The ΣK term is never less than 25. Therefore, Equation (8) can be written as

$$\frac{dQ}{dt} = -\frac{\Sigma K|Q|Q}{A_2^2 2\Sigma L/A} + \frac{g}{\Sigma L/A}(Y_1 - Y_2) \quad (9)$$

with little loss in accuracy.

Considering continuity in the upstream vertical leg (Figure 2):

$$Q_I - A_1\frac{dy_1}{dt} = Q \quad (10)$$

where Q_I is the inflow rate from the next upstream reach. Equation (10) can be rewritten as

$$\frac{dy_1}{dt} = \frac{Q_I - Q}{A_1} \quad (11)$$

Equations (9) and (11) must be solved simultaneously to determine what happens to flow in a particular pipe reach as time passes.

Interpretation

In order to provide understanding of the problem of unsteady flow in a pipeline, Equations (9) and (11) will be combined and rearranged in order to put them in a single classical form.

Differentiating Equation (9) gives

$$\frac{d^2Q}{dt^2} + \frac{\Sigma K|Q|}{2A_2^2\Sigma L/A}\frac{dQ}{dt} - \frac{g}{\Sigma L/A}\frac{dy_1}{dt} + g\frac{dy_2}{dt} = 0 \quad (12)$$

Substituting (11) into (12) gives

$$\frac{d^2Q_2}{dt^2} + \frac{\Sigma K|Q|}{2A_2^2\Sigma L/A}\frac{dQ}{dt} + \frac{g|Q|}{A_1\Sigma L/A} = \frac{gQ_I}{A_1\Sigma L/A} -$$

$$\frac{g}{\Sigma L/A}\frac{dy_4}{dt} \quad (13)$$

The absolute value of Q is used in order to insure that resistance always acts opposite to the flow. If it is assumed that dy_2/dt is zero, as it nearly is except for

Table I

DATA ON CANADIAN RIVER PIPES AND STRUCTURES*

Station	$\Sigma L/A$ (ft^{-1})	ΣK	A_1/A_2	R	T_N (Sec)
1882	1,603.4	180.0	850.6	322.0	6,876.0
2430†	1,741.6	156.8	0.9	1.3	245.7
2976	236.8	24.9	8.0	1.6	256.7
3044	1,156.2	121.5	7.1	3.2	533.9
3375	948.2	119.4	29.8	6.6	906.5
3617	674.5	66.5	29.8	6.1	764.6
3777	1,150.8	113.4	7.4	4.0	499.3
4050	2,423.0	238.8	29.8	11.6	1,449.0
4625	2,045.2	201.5	7.4	5.3	665.7
5107	642.3	63.3	7.4	3.0	373.0
5260	510.5	79.7	11.1	4.2	332.6
5356	553.2	47.0	8.3	4.6	300.0
5445†	1,418.8	80.3	0.7	1.7	166.3
5744	2,448.7	241.3	1.0	2.0	267.1
6701†	2,238.2	223.2	1.2	1.9	255.3
7232†	1,363.6	136.5	1.2	2.0	181.2
7487	1,428.3	121.4	8.3	6.9	482.1
7715†	449.9	38.3	1.0	1.3	93.7
7788†	830.0	70.0	1.0	1.6	127.8
7919	682.0	53.3	8.4	4.4	335.0
8026†	303.6	25.8	0.0	1.1	76.9

* "Station" identifies structure. Quantities are for pipe downstream from structure. Station 1882 is Amarillo Reservoir; the end of the pipeline, Station 8074, is Lubbock Reservoir; Stations marked "†" are pipe checks, Station 5744 is also a turnout. A roughness of 0.01-foot was used to compute K.

the case of large amplitude motion, then Equation (13) becomes

$$\frac{d^2Q}{dt^2} + \frac{\Sigma K|Q|}{2A_2^2 \Sigma L/A}\frac{dQ}{dt} + \frac{gQ}{A_1 \Sigma L/A} = \frac{gQ_I}{A_1 \Sigma L/A} \quad (14)$$

The form of Equation (14) is nearly that of the classical second-order differential equation widely known in the area of vibrations.[8] The only difference lies in the second term which is nonlinear in Q. The solution to Equation (14) depends on the relative values of the coefficients of dQ/dt and Q and the nature of Q_I. If the coefficient of the resisting or damping effect, dQ/dt, is small any displacement of the water in the pipe will produce a periodic fluid motion.

Equation (14) is generalized by dividing Q by Q_o (the design discharge) and t by T_N (the undamped natural period of the reach). Thus,

$$\frac{d^2(Q/Q_o)}{d(t/T_N)^2} + \frac{\Sigma K Q_o T_N |Q/Q_o|}{2A_2^2 \Sigma L/A}\frac{d(Q/Q_o)}{d(t/T_N)} + \frac{gT_N^2(Q/Q_o)}{A_1 \Sigma L/A}$$

$$= \frac{gT_N^2}{A_1 \Sigma L/A}(Q_I/Q_o) \quad (15)$$

Now, taking $\frac{Q}{Q_o}$ as being nearly equal to unity gives

$$\frac{d^2(Q/Q_o)}{d(t/T_N)^2} + \frac{\Sigma K Q_o T_N}{2A_2^2 \Sigma L/A}\frac{d(Q/Q_o)}{d(t/T_N)} + \frac{gT_N^2(Q/Q_o)}{A_1 \Sigma L/A} =$$

$$\frac{gT_N^2(Q_I/Q_o)}{A_1 \Sigma L/A} \quad (16)$$

Letting $R = \frac{\Sigma K Q_o T_N}{2A_2^2 \Sigma L/A}$ and $M^2 = \frac{gT_N^2}{A_1 \Sigma L/A}$

Equation (16) becomes

$$\frac{d^2(Q/Q_o)}{d(t/T_N)^2} + R\,\frac{d(Q/Q_o)}{d(t/TN)} + M^2\,(Q/Q_o) = M^2\,(Q_I/Q_o) \tag{17}$$

Solutions to Equation (17) are well known[6]. If R = 0 then the motion of the system will be undamped and any disturbance of the flow will produce a motion with the natural period.

$$T_N = 2\pi\,\sqrt{\frac{A_1 \Sigma L/A}{g}} \tag{18}$$

Thus, Equation (17) can be written as

$$\frac{d^2(Q/Q_o)}{d(t/TN)^2} + R\,\frac{d(Q/Q_o)}{d(t/T_N)} + 4\pi^2(Q/Q_o) = 4\pi^2(Q_I/Q_o) \tag{19}$$

The solutions to Equation (19) will yield considerable insight into the behavior of disturbed flow in the pipeline.

If $R > 4\pi$ the system is said to be overdamped or stable against oscillations. Any sudden change in discharge will produce a flow in the reach which exponentially approaches the final steady-state value. The flow in the reach produced by a periodically fluctuating inflow will also be periodic. However, the amplitude of the incoming fluctuations may or may not be amplified depending upon the relative magnitude of the period of the incoming flow and the natural period of the reach.

In order to adapt this discussion and analysis to the problem of interest, the magnitudes of R, T_N, and T_o which can occur in the real system need to be known. Table I shows the values of T_N and R as calculated for pipe reaches in the Main Aqueduct of the Canadian River Project as originally constructed.

There is only one reach in which R is seen to be greater than 4π. Hence, that is the only reach in which oscillations should not be expected as a result of a rapid change in flow rate. All other reaches are underdamped and may possibly amplify incoming flow osciallations depending upon the incoming frequency.

The Actual Surge Problem

As was shown in the preceding section, considerable qualitative understanding of unsteady pipeline flow can be gleaned from studying solutions of the linear Equation (19). However, quantitative information is obtained from solutions of Equation (15). Using the previous definitions of R and T_N, Equation (15) can be written as

$$\frac{d^2(Q/Q_o)}{d(t/T_N)^2} + R\,\frac{|Q|d(Q/Q_o)}{Q_o d(t/T_N)} + 4\pi^2(Q/Q_o) = 4\pi^2(Q_I/Q_o) \tag{20}$$

The second term in Equation (20) represents the rate at which energy is dissipated by pipe friction. The term Q/Q_o is equal to unity when the flow rate Q is exactly equal to the design discharge, Q_o.

Because of design procedures the pipe runs full throughout when $Q = Q_o$, and the hydraulic gradient passes through the top of each structure as shown in Figure 1. In Equation (9), the first term on the right of the equal sign shows that the dissipation rate is proportional to the square of discharge. Thus, if the discharge is descreased below Q_o the damping decreases more rapidly than does Q. A pipe reach which had a large enough R value to just be stable when flow was taking place at design conditions could then become quite unstable if the flow rate were decreased below Q_o.

ANALYSIS

Stability

As noted in the previous section, Equation (20), which is characteristic of the motion of water in a given pipe reach, can be expected to predict an underdamped motion for the practical case. As well as being dependent upon R, the resulting motion is also dependent upon the inflow Q_I. If Q_I is periodic, the linear Equation (19) predicts an exponentially damped harmonic motion which, as time increases, developes a constant period and amplitude governed by the period and amplitude of Q_I. This motion is illustrated in Figure 3a.

The nonlinear Equation (20) can be expected to give a somewhat similar result. If Q_I is abruptly changed, the linear Equation (19) predicts a resulting motion which will be overdamped or underdamped depending upon R. If Q_I is changed from one constant value to a second constant value, a harmonic motion is produced for the underdamped case. The harmonic motion is damped out as time increases, eventually producing a constant flow Q equal to Q_I. Such a motion is illustrated in Figure 3b. The rate at which

Q_I is changed (dQ_I/dt) is also of importance. If dQ_I/dt is very small, then the resulting unsteady flow will have an insignificant amplitude. As the absolute value of dQ_I/dt is made larger, the resulting unsteady flow will have a larger amplitude.

In classical dynamics, systems are checked for instability with both the periodic and the change-of-flow rate inputs mentioned in the previous two paragraphs as well as with a pulse input.[7] In the case of interest here, both the periodic inflow and the change-in-inflow (Figure 3) need to be considered in the stability analysis. A pulse of input cannot logically occur in the system of interest and need not be considered.

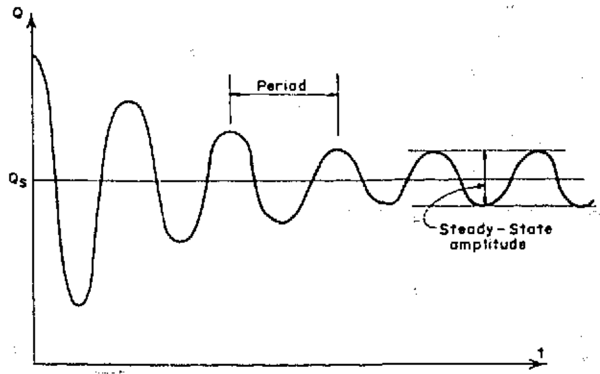

Figure 3a. Damped oscillations due to an oscillating inflow.

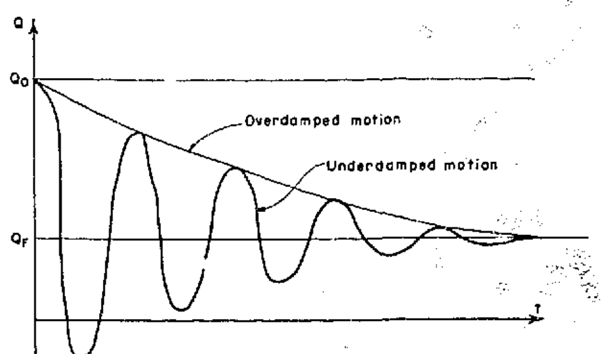

Figure 3b. Flow rate due to a sudden cutback in inflow. Typical types of unsteady flow.

Figure 3. Typical types of unsteady flow.

Periodic Inflow

Instability arising as a result of periodic inflow has been investigated by Holley.[4] In his report, Holley generated numerical solutions to the nonlinear equation of motion for a sinusoidal incoming flow

$$Q_I = Q_m \sin(2\pi t/T_o) + Q_s$$

where Q_m is the amplitude of the fluctuating flow occurring with period T_o superimposed on the steady-state flow rate Q_s. Holley formulated his dimensionless equivalent to Equation (20) as

$$\frac{d^2(Q/Q_m)}{d(T/T_o)^2} + 2R\,\frac{Qm}{Q_o}\frac{T_o}{T_N}\left|\frac{Q}{Q_M}\right|\frac{d(Q/Q_m)}{d(t/T_o)}$$

$$+ 4\pi^2\,\frac{T_o{}^2}{T_N{}^2}\,\frac{Q}{Q_m} = 4\pi^2\,\frac{T_o{}^2}{T_N{}^2}\,\frac{Q_I}{Q_m} \qquad (21)$$

The solution of Equation (21) yields the amplitude and period of motion resulting from periodic inflow. Figure 4 shows Holley's results for two values of Q_s/Q_m. Resonance is seen to occur at $T_o/T_N = 1.0$ just as would occur in a linear system. The ordinate represents the amplification of the incoming flow fluctuations. In order to insure that fluctuations of excessive amplitude do not arise, fluctuations entering one reach should not be amplified in passing through that reach. It would be desirable if they could be diminished.

Two methods of control are immediately obvious from Figure 4. The first and most obvious method would be to make the damping (resistance to flow) large enough to insure that amplification does not occur. The second method (the method of control used on Coachella) is to insure that the natural periods of successive reaches differ sufficiently to make amplification impossible.

In order to use Figure 4 effectively, it is first necessary to insure that values of Q_s/Q_m of 1.0 and 2.0 can reasonably be expected to occur in a pipeline system. Table II was prepared using the values reported by Holley based on his laboratory study.[4] Values of Q_s/Q_m are seen to vary widely but appear to encompass 1.0 and 2.0, as well as both larger and smaller values.

If the amplitude of the resulting fluctuation in Q is greater than the amplitude of the incoming periodict disturbance Q_m, amplification has occurred. On Figure 4 amplification occurs any time $(Q-Q_s)/Q_m$ is greater than 1.0. Thus, for $Q_s/Q_m = 1.0$, amplification occurs whenever Q/Q_m is greater than 2.0 or less than zero.

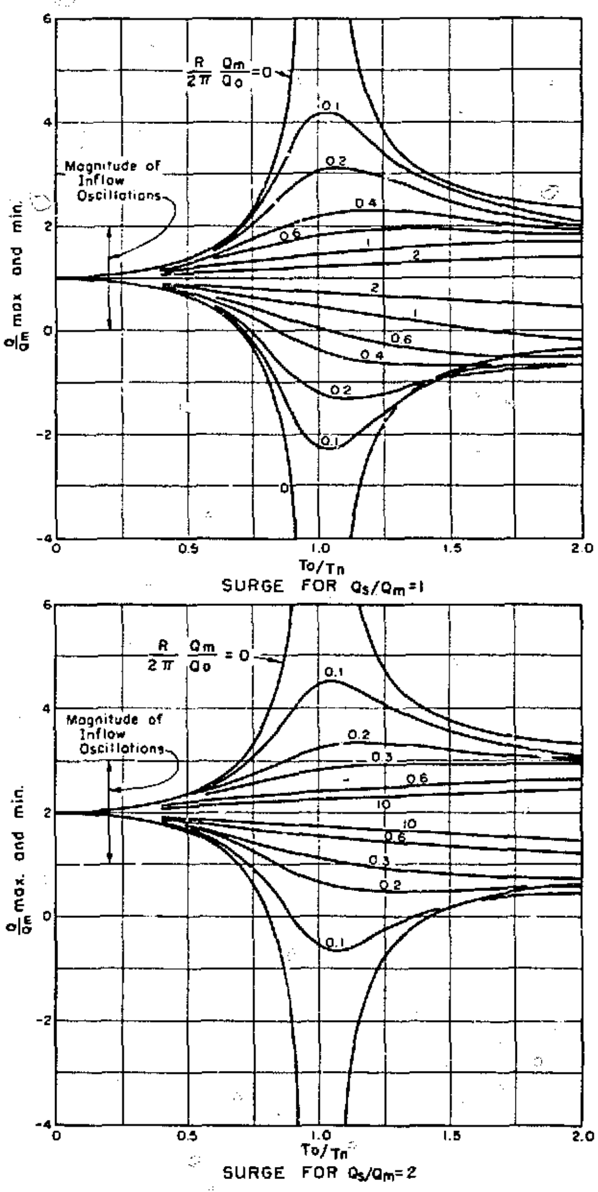

Figure 4. Magnitude of discharge surges

Examination of Figure 4 indicates that, regardless of the value of $(R/2\pi)\ Q_m/Q_o$, amplification cannot occur if T_o/T_N is less than 0.7. For large values of T_o/T_N (2.0 or greater) amplification will not occur if $(R/2\pi)\ Q_m/Q_o$ is at least 1.2 for $Q_s/Q_m = 1.0$ and 0.5 for $Q_s/Q_m = 2.0$.

If a disturbance (a sudden change in flow rate or an oscillation produced by entrained air) occurs in an underdamped reach, the outflow will have a period approximately equal to the natural period of that reach. Passing through the succeeding reach, depending upon the values of $(R/2\pi)Q_m/Q_o$ and T_o/T_N for

Table II

DIMENSIONLESS DISCHARGE RATIOS FROM A SURGING MODEL PIPELINE

Q_o (cfs)	Q_s (cfs)	Q_m (cfs)	Q_s/Q_m	Q_m/Q_o
0.166	0.02	±0.041	0.50	0.247
0.166	0.04	±0.048	0.91	0.289
0.166	0.06	±0.020	5.88	0.120
0.166	0.08	±0.008	16.67	0.048
0.166	0.10	±0.011	14.30	0.066
0.166	0.12	±0.028	4.00	0.169
0.166	0.14	±0.019	4.75	0.114

Values of Q_s and Q_m are taken from Holley[4], Figure 17A - Low head loss.

this succeeding reach, the oscillation may be somewhat reduced, unaffected, or amplified in amplitude. However, unless great amplification occurs, the period of oscillation will not be changed significantly.

Changes in Inflow Rate

When the flow is suddenly cut back, the inertia of the water in motion carries it on through the system. Resistance in the form of fluid friction opposes the inertial movement. The water surface at the upstream end drops and the driving force decreases, finally becoming a restoring force if the upstream level falls to an elevation lower than the downstream level. The solution to Equation (20) describes this motion. Here again if R is large enough, the transition from one flow rate to a smaller one will occur without resulting unsteadiness. However, if R is small compared to 4π, oscillations about the final steady-state flow rate Q_F can occur. If the oscillations are large enough, the upstream water surface could rise into the structure causing overtopping.

In order to study surging caused by sudden flow change, Equation (20) had to be solved numerically. Because Equation (20) is a second-order nonlinear differential equation, it was necessary to write it as two equations — Equations (9) and (11). These two equations, made dimensionless in accord with Equation (20), become

$$\frac{d(Q/Q_o)}{d(t/T_N)} + R\left|\frac{Q}{Q_o}\right|\frac{Q}{Q_o} - 4\pi^2\,\frac{y_1A_1}{Q_oT_N} + 4\pi^2\,\frac{y_2A_1}{Q_oT_N} = 0 \qquad (22)$$

and

$$\frac{d\left(\frac{y_1 A_1}{Q_o T_N}\right)}{d(t/T_N)} = \frac{Q_I}{Q_o} - \frac{Q}{Q_o} \qquad (23)$$

A Runge-Kutta method of solution was used in the numerical simultaneous solutions of Equations (22) and (23).[8] The program used is listed in the Appendix. Actually the solution of Equations (22) and (23) is a six-parameter problem. The parameters are Q/Q_o, t/T_N, A_1/A_2, Q_F/Q_o, R, and $d(Q_I/Q_o)$ $d(t/T_N)$ where

$$(Q_I/Q_o) = 1 + \int_0^{t/T_N} \frac{d(Q_I/Q_o)}{d(t/T_N)}\, d(t/T_N) \qquad (24)$$

Looking at Equation (23) one might expect y_1A_1/Q_oT_N and y_2A_1/Q_oT_N to also be parameters. However, $y_1 - y_2$ is fixed by the head loss occurring between the two structures at the design flow Q_o. The dimensionless elevation y_1A_1/Q_oT_N is determined by Q/Q_o and t/T_N and, thus, is not independent.

Two occurrences are of interest in the solution of Equations (22) and (23). First, if the flow becomes negative (moving upstream), it will unwater the downstream tower rapidly. Second, if the upstream water surface rises to the top of the upstream structure, overtopping will occur.

Figure 5 shows the effect of resistance on the flow rate Q/Q_o for given values of $d(Q_I/Q_o)/d(t/T_N)$, A_1/A_2 and Q_F/Q_o. As R increases, the maximum amplitude becomes smaller with almost no fluctuation arising for R = 10. With A_1/A_2 = 1.0, Figure 5 corresponds to a pipe check. For a pipe stand, A_1/A_2 is much larger than 1.0 (Table I). The pipe stand provides additional storage and thus can be expected to decrease the amplitudes of fluctuation to smaller values than those of Figure 5, other conditions being equal.

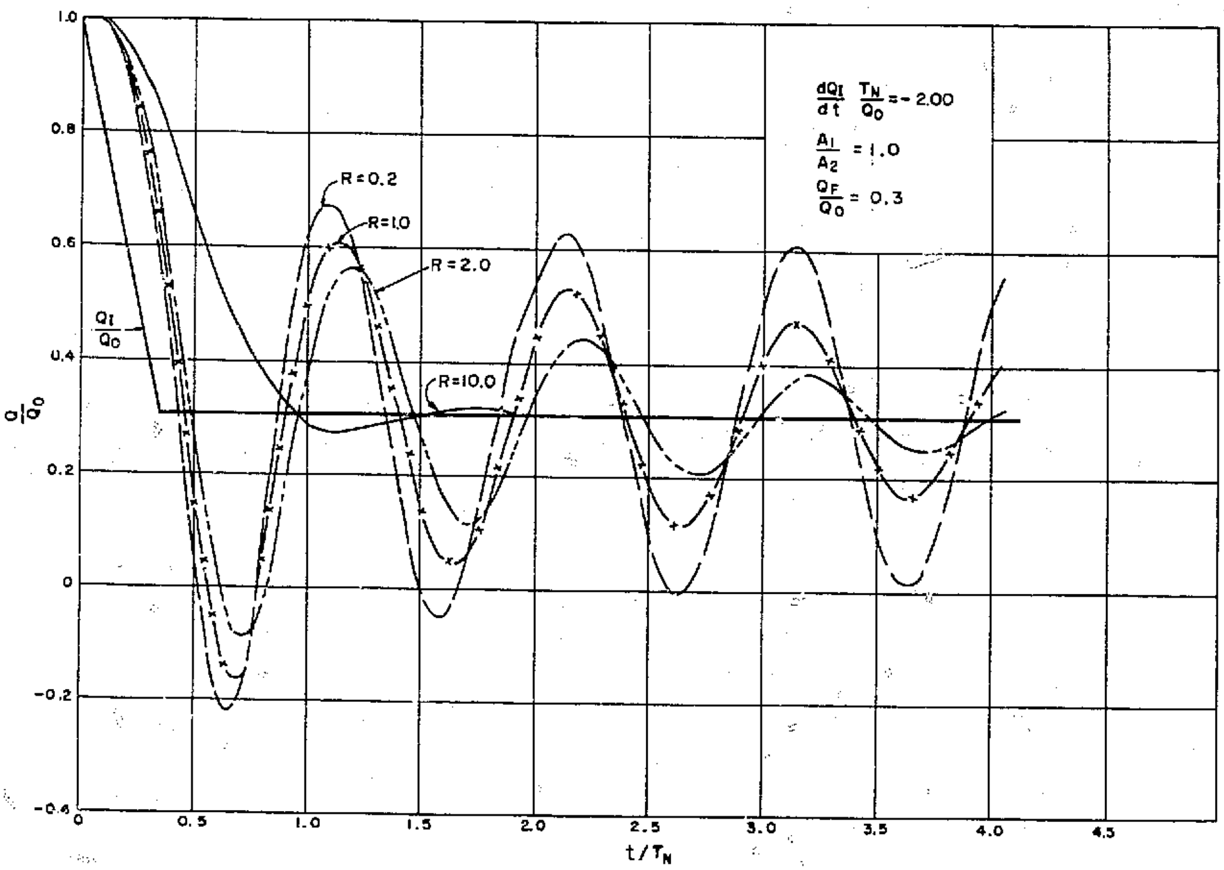

Figure 5. Effect of resistance on the flow rate.

Figures 6 through 15 (at end of report) show Q/Q_o and y_1A_1/Q_oT_N for various combinations of R, Q_F/Q_o, A_1/A_2 and the cutback rate $d(Q_I/Q_o)/d(t/T_N)$. Not all solutions obtained are represented in the figures. Only those depicting special conditions were graphed. In all solutions, except where Q became negative, y_2 was taken to be constant. Actually, it will vary somewhat since the downstream structure acts as a weir and y_2 must be relatively large for large positive flow rates. However, it is $y_1 - y_2$ which is really important in the solution since this elevation difference represents the driving force. Because y_1 undergoes large changes with respect to time, the minor change of y_2 for positive flows was neglected. Figure 12 shows a case where y_2 actually decreased because the flow rate Q became negative. Figures 9 and 15 illustrate more drastic cases.

In Figures 6 through 15, wherever y_1A_1/Q_oT_N rises above its value at $t/T_N = 0$, overflow of the upstream structure is predicted. In some cases such as that represented by Figure 8, overflow occurs despite the fact that Q/Q_o never becomes negative. However, that is a case of rapid cutback of inflow and low resistance such as might be expected in a short reach.

The periods of flow in Figures 6 through 15 are seen to be nearly equal to the natural undamped frequency of the system. Although damping decreases the natural period, in the cases shown, damping is small in comparison to the inertial forces and, hence, has a relatively small influence.

The relative influence of cutback rate is demonstrated in comparing Figures 13 and 15. Slow cutback rate is seen to produce almost no fluctuation (Figure 13) while rapid cutback (Figure 15) produces large fluctuations and even negative flow.

Maximum and minimum values of Q/Q_o are summarized in Figures 16 through 21 for all solutions obtained. Those figures show the extreme values of Q/Q_o for the values of R, A_1/A_2, Q_F/Q_o, and cutback rate. Magnitudes of extreme values of Q/Q_o are seen to be increased as the cutback rate is increased, as R becomes smaller, as Q_F/Q_o becomes smaller, and as A_1/A_2 becomes smaller.

However, as demonstrated by Figure 16 there are cases where magnitudes of extreme fluctuations are reduced as $d(Q_1/Q_o)/d(t/T_N)$ is made larger. For instance, in Figure 16 with R = 0.5, the maximum value of Q/Q_o is 0.32 for $d(Q_I/Q_o)/d(t/T_N) = -0.5$ and 0.24 for $d(Q_I/Q_o)/d(t/T_N) = -0.75$.

When a cutback in flow rate is started, the system is immediately set into unsteady motion with natural period T_N. The cutback reinforces a decreasing flow rate and diminishes an increasing one. If the cutback rate is very slow, its effect is distributed over many periods, and thus its net effect is a small oscillation. If the cutback rate is large, its effect is felt over only a fraction of a total period, and a relatively large oscillation occurs.

However, there is a small range in cutback rate at which the accelerating effect of the cutback, occurring during the first half of the first period in which discharge is falling, is counteracted by the decelerating effect during the second half of the first period. If the cutback rate is slightly less than this, it acts over more than one accelerating period, producing a slightly larger fluctuation.

As pointed out in an earlier section, the overtopping of a structure occurs when y_1 rises to the top of the structure. Thus, critical values of cutback rate can be observed by looking at the motion pattern shown in Figures 6 through 15, where y_1A_1/Q_oT_N curves with flat tops such as Figure 12 indicate that overtopping of the upstream structure would occur.

The extreme movements of the upstream water surface have been summarized in Figures 22 through 27. In those figures the diagonal line going upward to the right is the difference between the tops of the upstream and downstream structures for steady flow [Equation (22) with $d(Q_I/Q_o)/d(t/T_N)$ set equal to zero]. As Equation (22) indicates, this difference in elevation (in dimensionless form) is determined by R alone with $D(Q/Q_o)/d(t/T_N)$ is zero (steady flow) and $Q/Q_o = 1$ (design flow).

The other lines in Figures 22 through 27 are the extreme rises in upstream water surface elevations produced by particular cutback rates for particular values of R, Q_F, and A_1/A_2. Whenever a line for a particular cutback rate crosses to the left of the diagonal line, overtopping has occurred.

The effect of increasing storage in the upstream riser (making A_1/A_2 larger) in general makes it possible to cut back at a faster rate without overtopping than could be done with a pipe-check structure. The effect of the resistance (R) is already seen. For a design situation, a cutback rate could be chosen at which overtopping would not occur in a pipe reach for which the values of R, A_1/A_2, and Q_F/Q_o have been calculated.

CORRELATION WITH FIELD EXPERIENCE

Previous Field Tests

The Coachella and Canadian River systems provide the bulk of the history which can be related to the phenomenon of surging in low-pressure pipeline systems. As mentioned earlier, the Coachella system was constructed with pipe reaches having almost identical natural periods, and a resonant condition was reached. The recommended remedy involved placing covers over adjacent pipe stands to effectively change the natural periods of the reaches.[1] A review of the Project Record for Coachella for 1955 and 1958 shows that this system of control has not always been successful, but that the operators have learned how to avoid trouble in most cases.

Examination of Figure 4 can yield a possible explanation for the failure of airtight lids to work in all cases. If the period of a reach is effectively increased, an oscillation entering it may not be amplified. However, when passing into a following reach, amplification can take place since at that point T_o/T_N will be greater than unity, and R is small. Project records also indicate that problems with surging arise when discharges are changed.

The Canadian River system was tested by personnel from the Hydraulics Branch and the Project in 1968. Observations showed that flow occurred smoothly and trouble free at design flow. However, upon cutting back from design flow, surges developed which overtopped four structures. Two of the overtopped pipe-check structures were subsequently replaced with pipe-stand structures (large A_1/A_2). Subsequent tests by project personnel in 1969 produced overtopping of only one pipe-check structure when the flow rate was changed.

Canadian River Tests, 1968

Table III shows values of $(R/2\pi)Q_m/Q_o$, T_o/T_N, and amplification factors as computed for the Main Aqueduct of the Canadian River system as they existed in 1968. A value of $Q_m/Q_o = 0.29$ was assumed in computing $(R/2\pi)\,Q_m/Q_o$ since that is the maximum value indicated in Table II. T_o/T_N was computed as the ratio of the natural periods (T_N) of each pipe reach and the reach immediately preceding it (T_o). Maximum amplification factors are also shown in Table III $RQ_m/2\pi Q_o = 0$ to indicate the possible range of amplification.

Examination of the maximum amplification factors contained in Table III indicates five reaches in which severe amplification might have been expected (Stations 2976, 3617, 5260, 5356, and 6701). Of these five, only Station 6701 overtopped in actual operation. When the amplification factors computed with $Q_m/Q_o = 0.29$ are examined, only Stations 6701 and 2976 indicate possible trouble due to resonance. No trouble was observed with Station 2976. However, it was near the upstream end of the system and was probably not subject to incoming oscillations as severe as those entering structures farther downstream.

During operation, overtopping of structures also occurred at Stations 5445, 7788, and 7919. At none of these locations could resonance with the structure immediately upstream have been a problem. Station 5445 probably overtopped because of the combined amplification occurring through the structures upstream from it. Overtopping of the structures at Stations 7788 and 7919 appears to be more subtle. Amplification factors for these structures do not appear to be severe.

However, no consideration is given to resonance with structures farther than one reach upstream. Table I indicates that several reaches upstream have periods nearly in resonance with Station 7788 or 7919. A periodic surge generated upstream could pass through several reaches without being amplified before reaching one where large amplification occurs. Thus, a disturbance generated at Station 5260 or 5356 might very well receive large amplification at Station 7919 since, unless overtopping occurred somewhere between these two stations, the period of the surge would remain relatively unchanged. Similarly, a disturbance generated at Station 5445 or 7232 might cause severe problems at Station 7788.

Canadian River Tests, 1969

A review of the discharge records for the 1969 Canadian River tests as recorded at the Kress flowmeter at Station 5080 show that a change of discharge from 40 to 0 cfs took place in about 4-1/2 minutes. For that reach the rate corresponds to dimensionless cutback rate $d(Q_I/Q_o)/d(t/T_N) = -0.50$. Using the values of R shown in Table I, Figure 27 indicates that none of the structures should have been overtopped. Only the structure at Station 8026 was overtopped, and that only slightly. However, the ratio T_o/T_N is 4.4 for the reach following Station 8026 and the value of $RQ_m/2Q_o$ is 0.48 as seen in Table III. Figure 4 shows that amplification might well be expected in that reach. Thus, overtopping of that structure might have been expected.

Table III

CANADIAN RIVER SURGE DATA

Station (ft)	Q_o (cfs)	$\frac{RQ_m}{2\pi Q_o}$	$\frac{T_o}{T_N}$	Amplification† $(Q_{max} - Q_s)/Q_m$ Maximum $\left(\frac{RQ_m}{2\pi Q_o}\right) = 0$	Amplification† Using $\frac{RQ_m}{2\pi Q_o}$ from Column 3
1882	92				
+2430	92	0.06	29.0	1.4	1.0
2976	92	0.08	1.0	≫5.0	≫5.0
3044	92	0.15	0.5	0.4	0.3
3275	92	0.06	0.6	0.5	0.4
3617	92	0.34	1.2	≫5.0	1.6
3777	92	0.27	1.5	1.9	1.3
4050	92	0.54	0.3	0.2	0.2
4625	92	0.24	2.2	1.4	1.0
5107	92	0.14	1.8	1.5	1.2
5260	92	0.20	1.1	≫5.0	2.1
5356	92	0.22	1.1	≫5.0	2.0
+5445*	92	0.08	1.8	1.5	1.3
5744	92	0.09	0.6	0.6	0.4
+6701*	85	0.09	1.0	≫5.0	≫5.0
7232*	85	0.09	1.4	1.9	1.7
7487	85	0.32	0.4	0.3	0.3
7715*	85	0.06	5.2	1.4	1.0
+7788*	85	0.08	0.7	1.2	0.9
+7919	85	0.20	0.4	0.3	0.3
8026*	95	0.48	4.4	1.4	1.0

† Amplification factors from Q_s/Q_m = 1.0 curves, Figure 4a

• Check structures

+ Structures which experienced overflow during the 1968 tests

SIMULTANEOUS CONSIDERATION OF SEVERAL REACHES

At the outset of this study it was planned to consider several reaches simultaneously to see what the mutual effect of one reach was on the remainder. However, when this problem was formulated mathematically, it was found that the system of equations was singular. Practical consideration of the problem showed that this should have been expected. The only way a downstream reach can affect an upstream reach is for flow to move upstream into the upper reach. That condition already violates the desired design conditions. In other words, the surge is already beyond tolerable limits, and it is of little consequence to know how much worse it gets.

Thus, the condition of interest is only the effect of inflow on any given reach. If the reach does not surge excesively for the frequency and rate of decrease expected in the inflow, it will be satisfactory. Consideration of each reach for these conditions should result in a satisfactory overall design.

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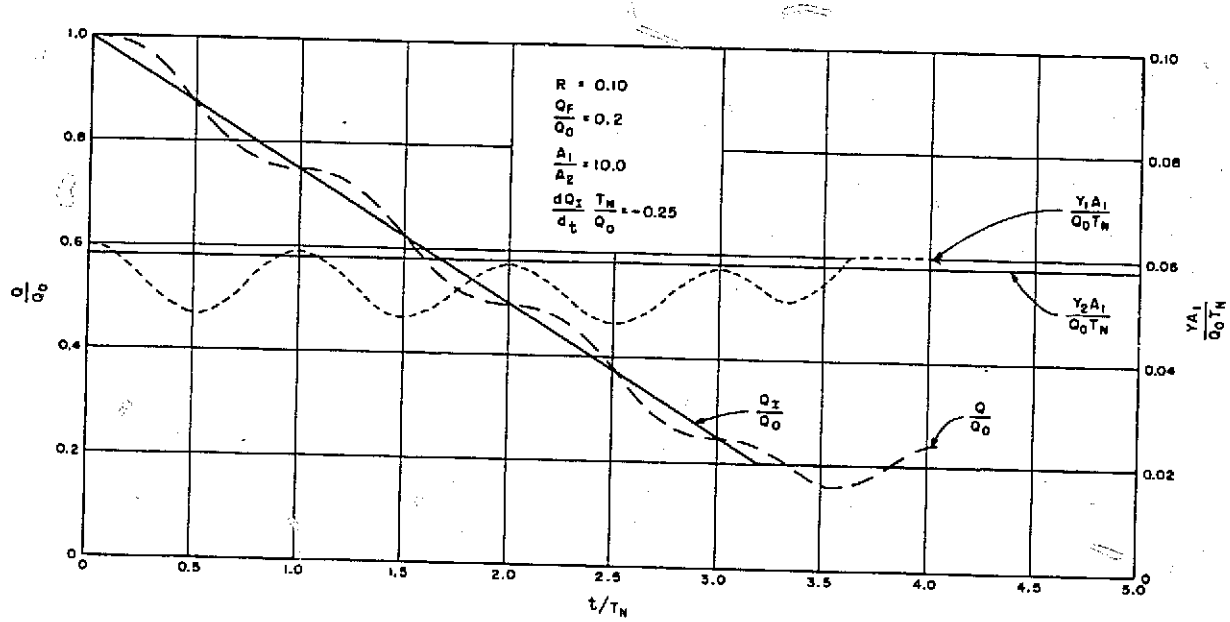


Figure 6.

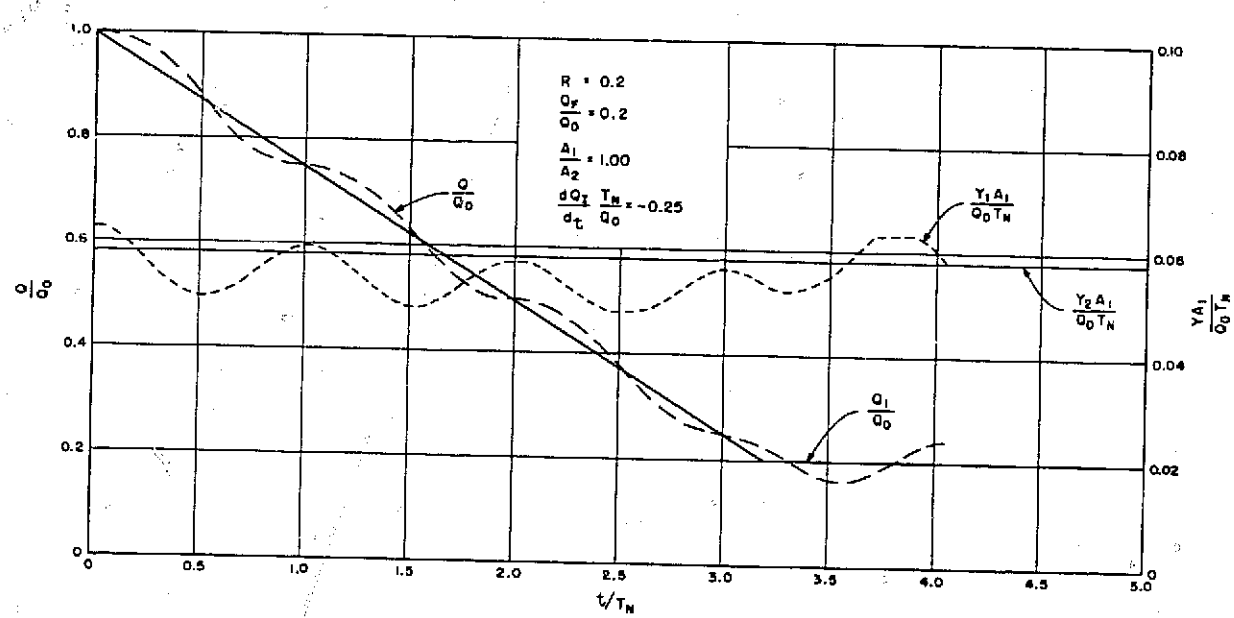


Figure 7.

Dimensionless discharge variations as a function of dimensionless time.

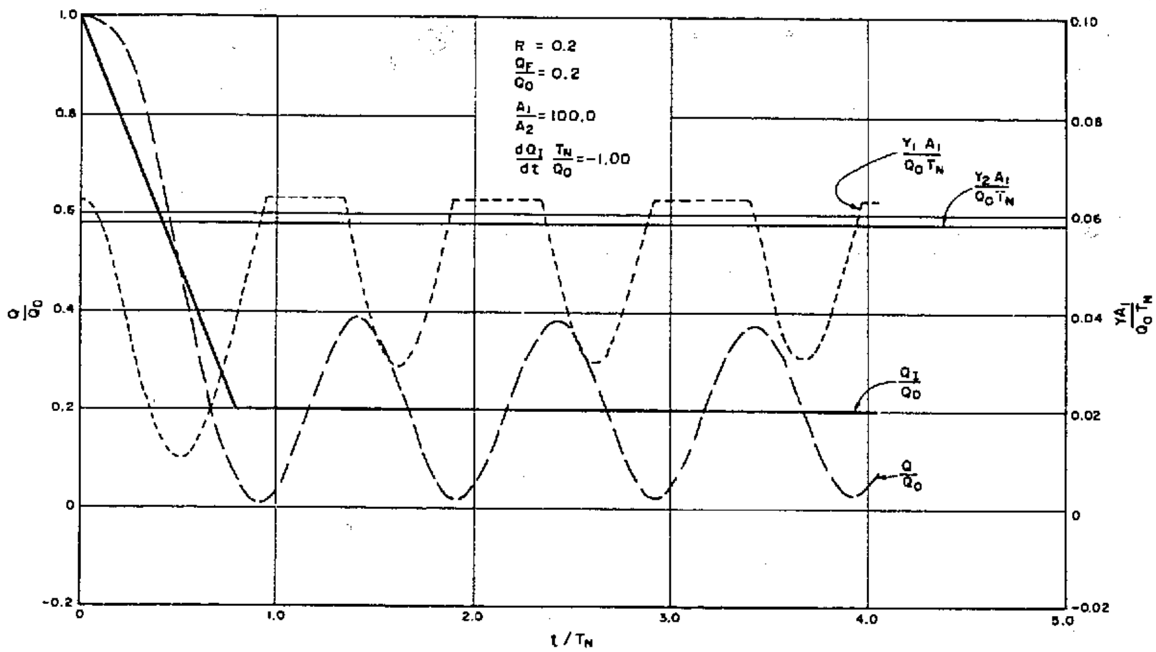


Figure 8.

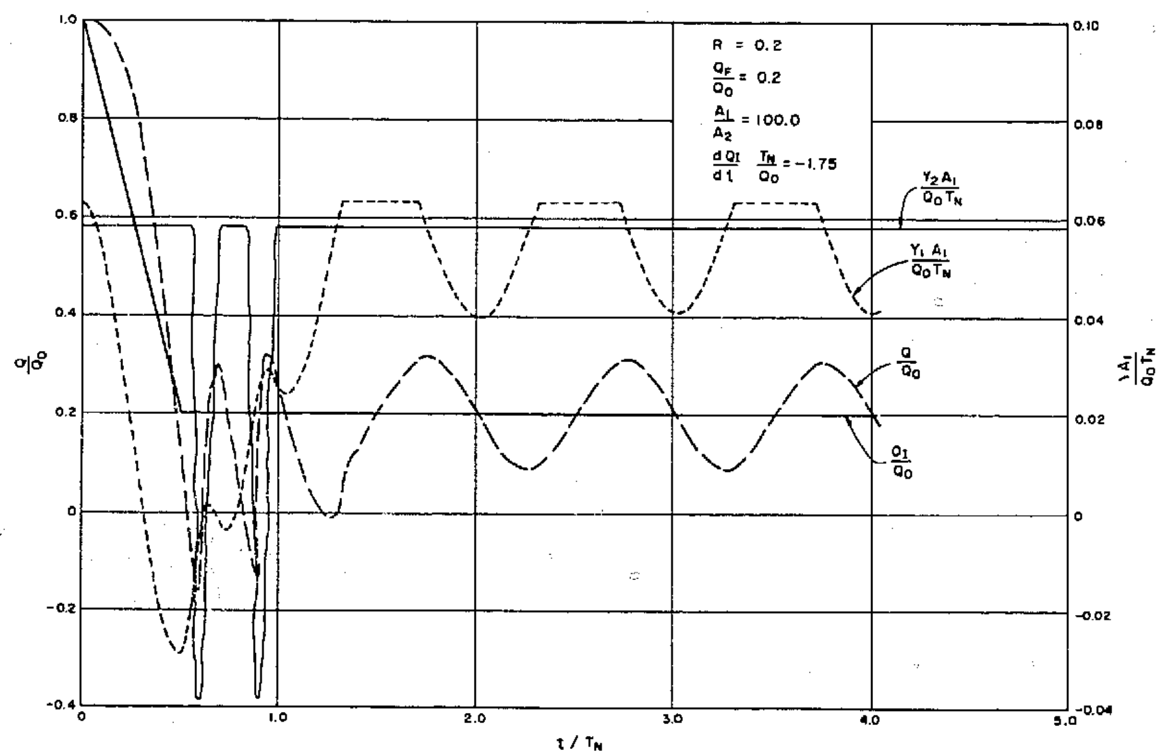


Figure 9.

Dimensionless discharge variations as a function of dimensionless time.

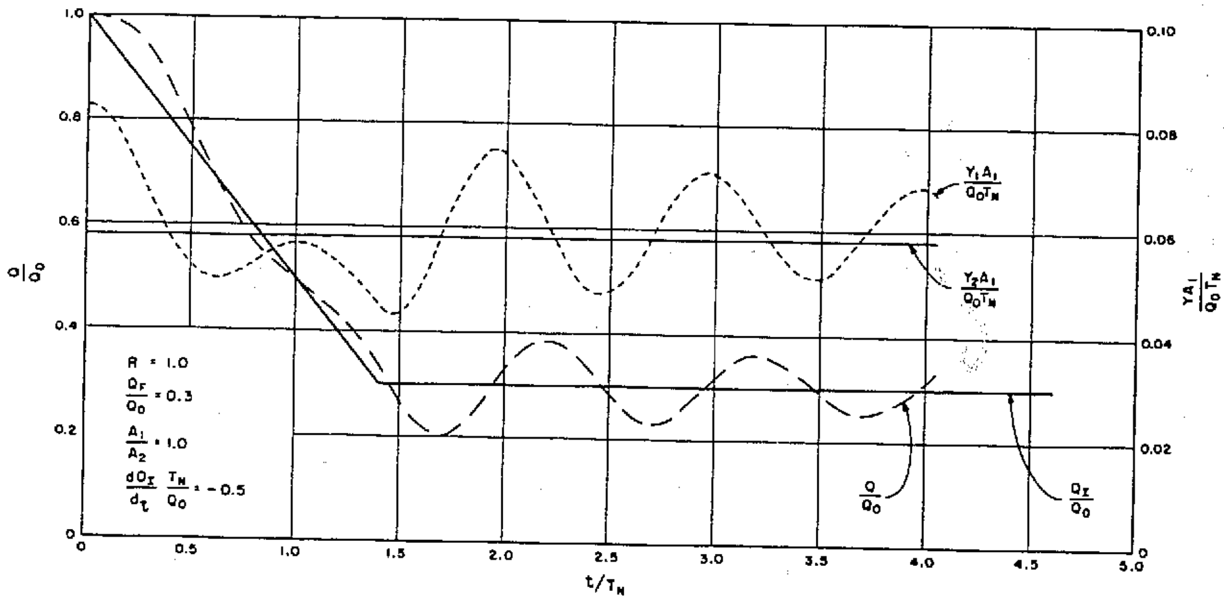


Figure 10.

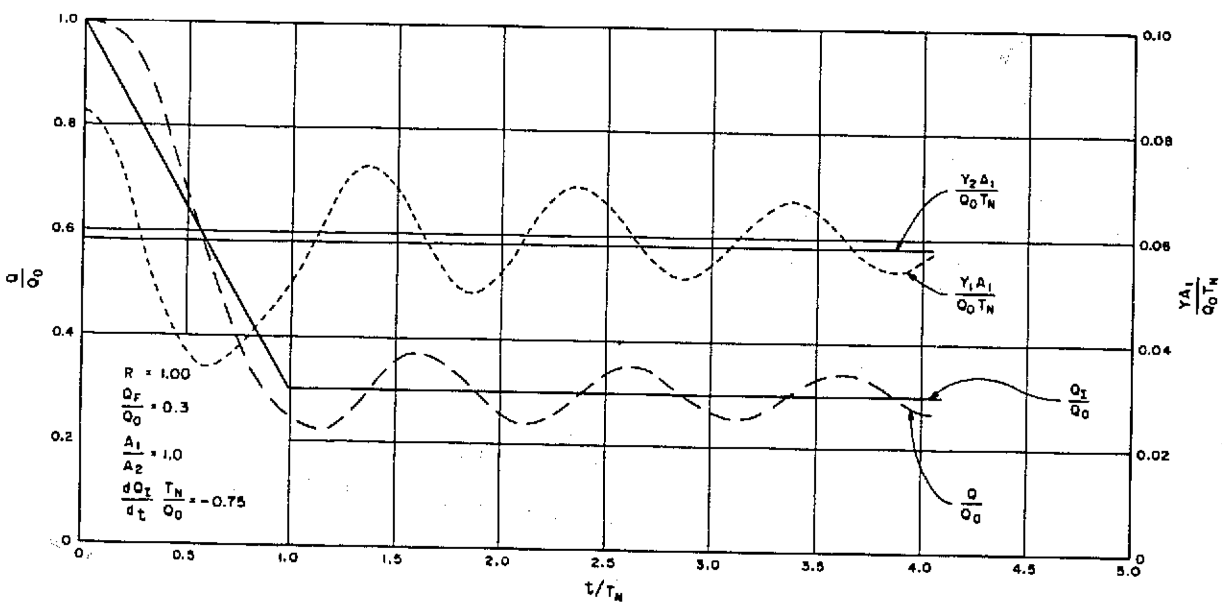


Figure 11.

Dimensionless discharge variations as a function of dimensionless time.

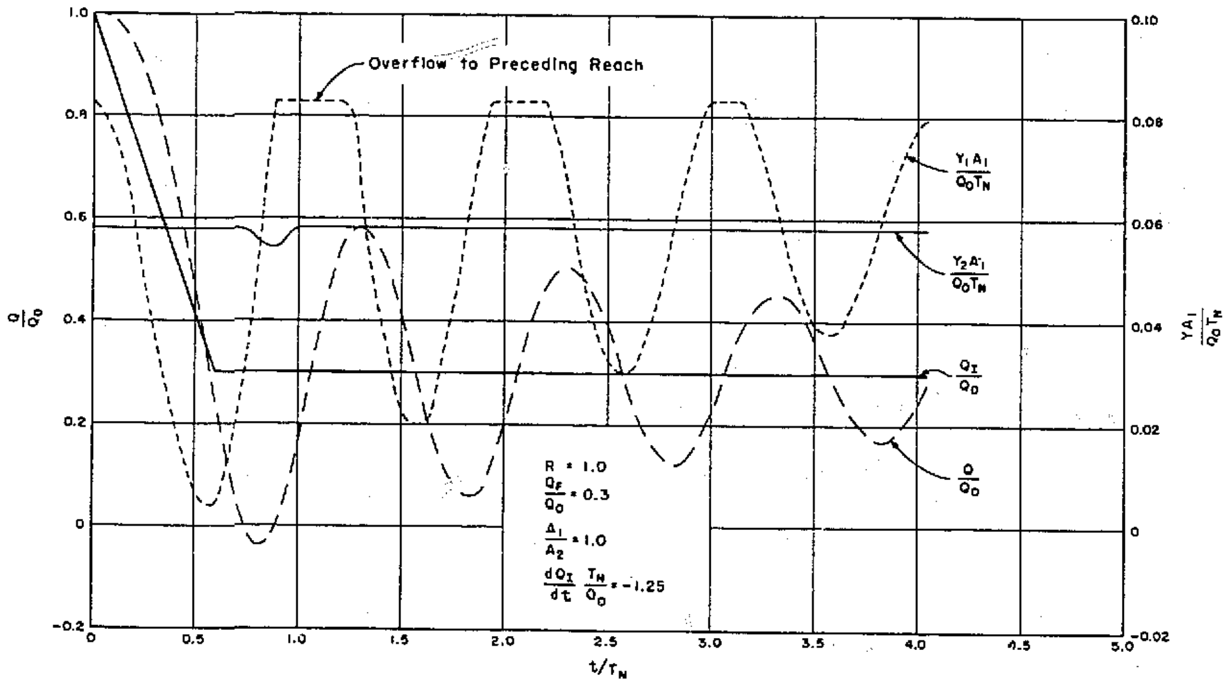


Figure 12.

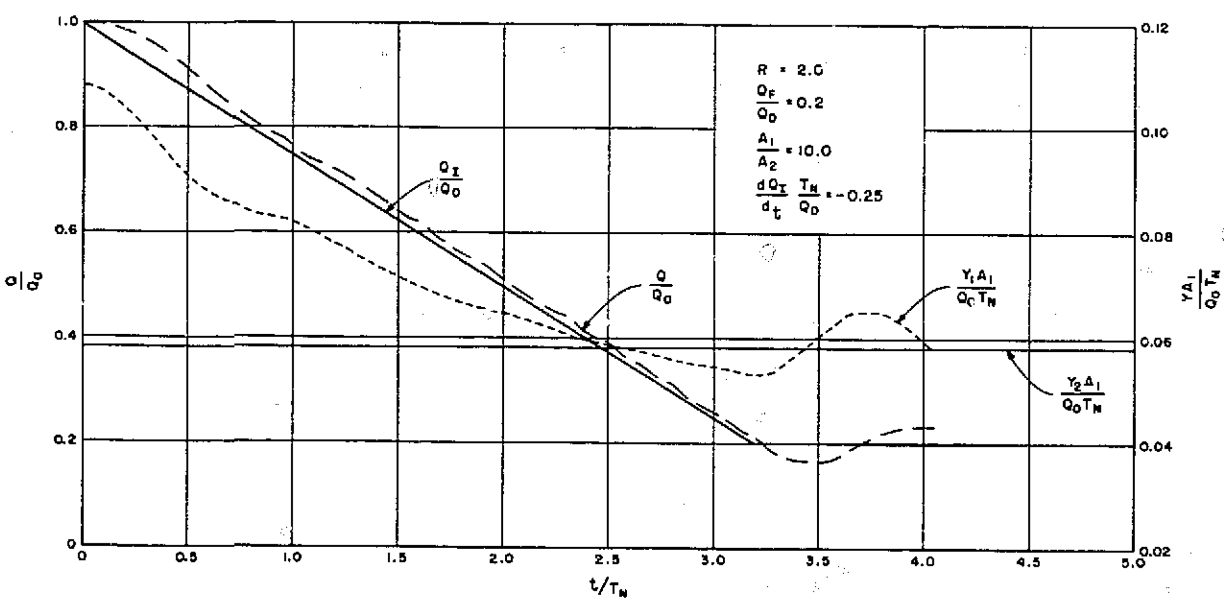


Figure 13.

Dimensionless discharge variations as a function of dimensionless time.

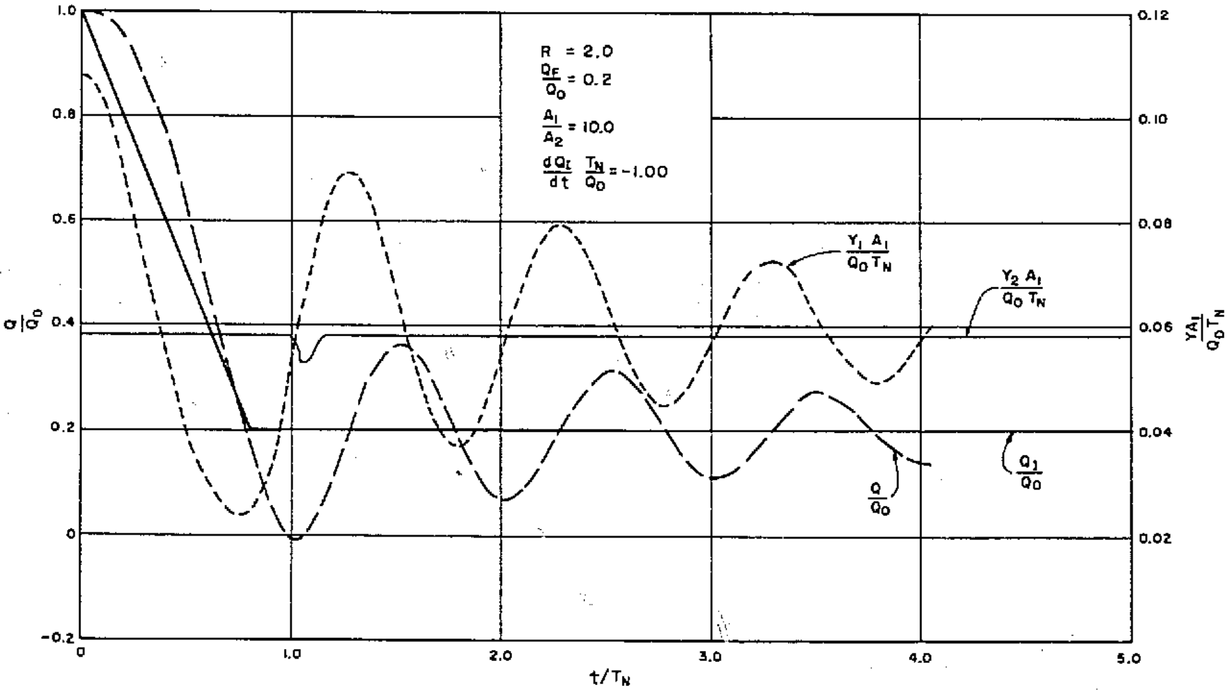


Figure 14.

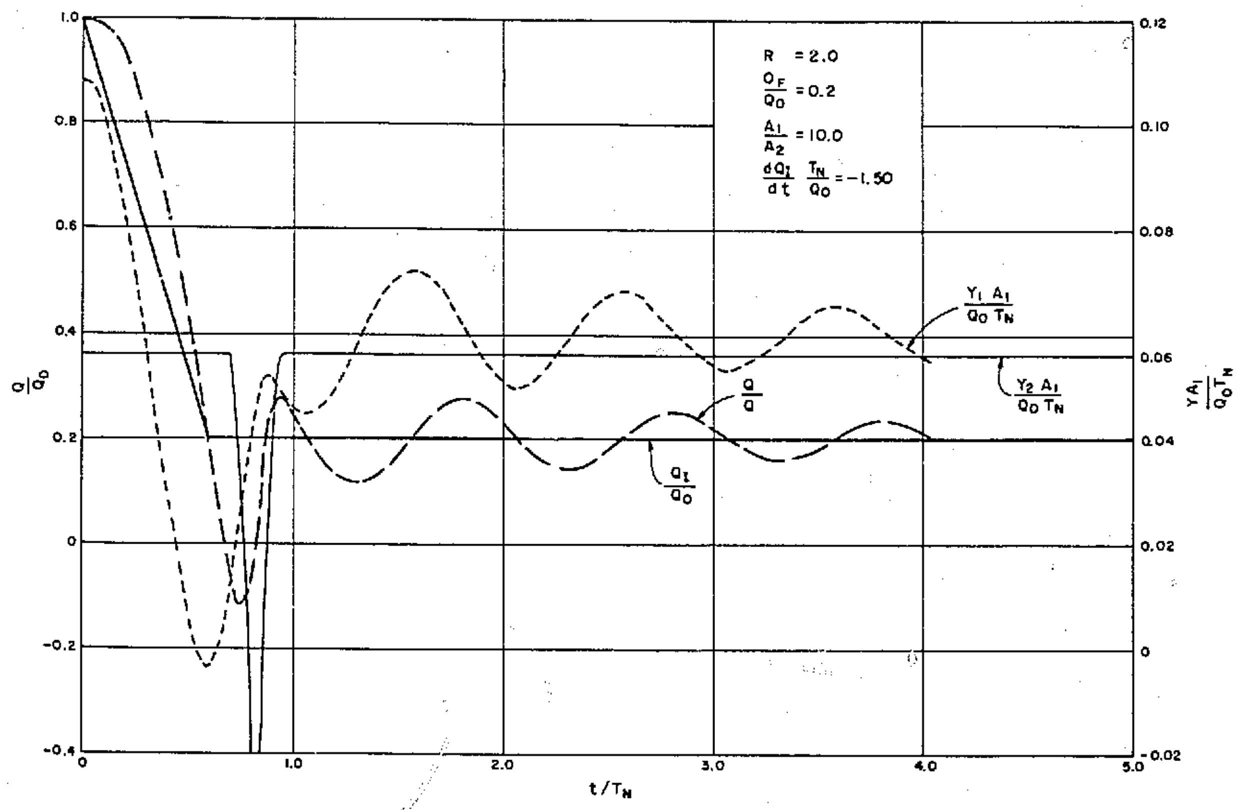


Figure 15.

Dimensionless discharge variations as a function of dimensionless time.

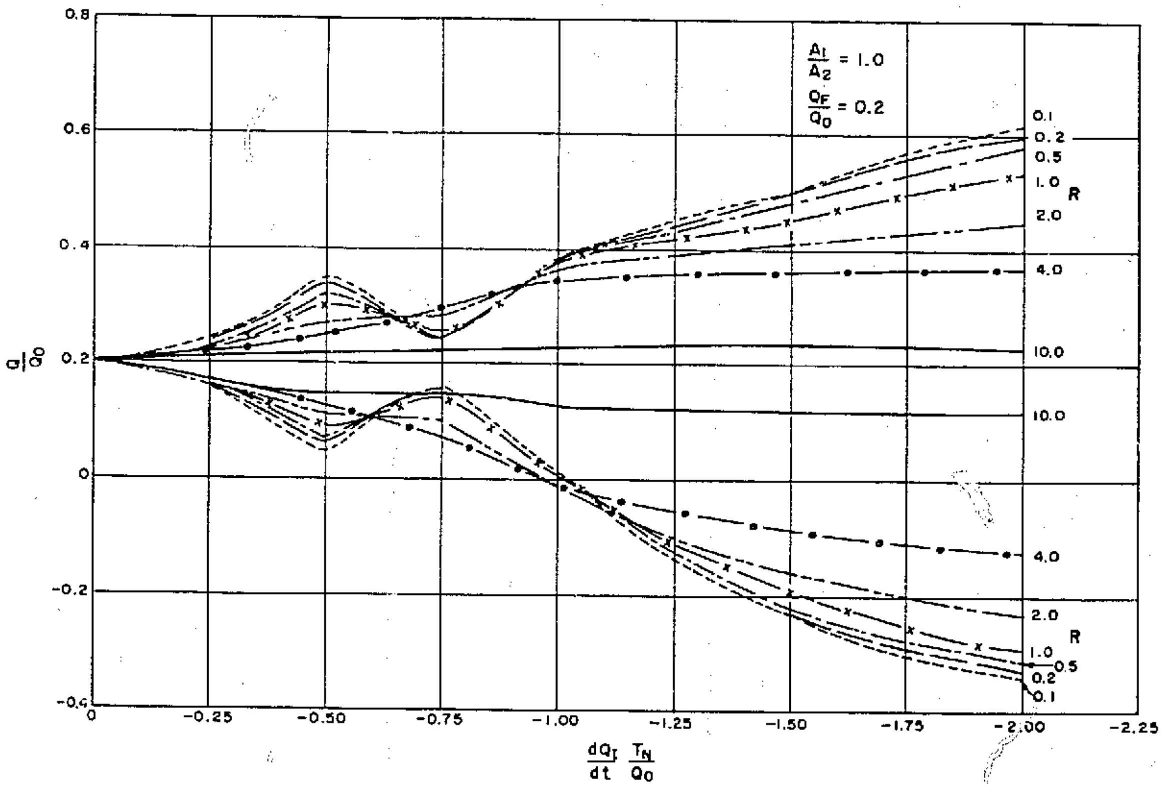


Figure 16.

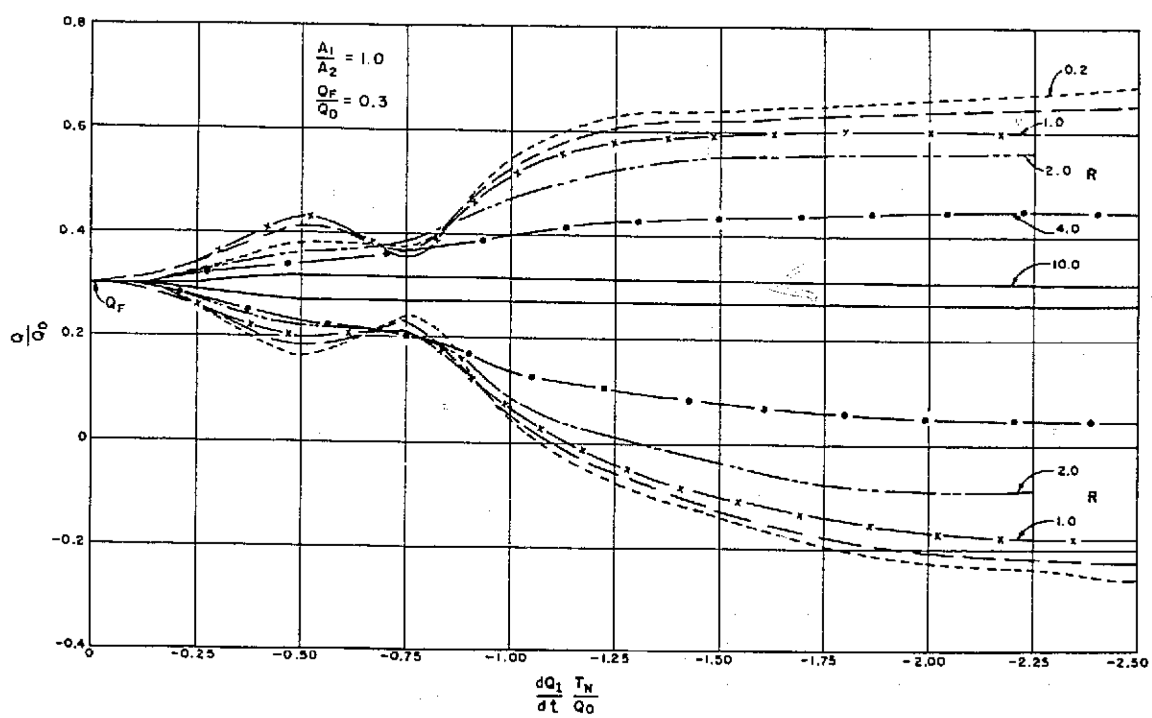


Figure 17.

Range of dimensionless discharge as a function of the dimensionless discharge cutback ratio.

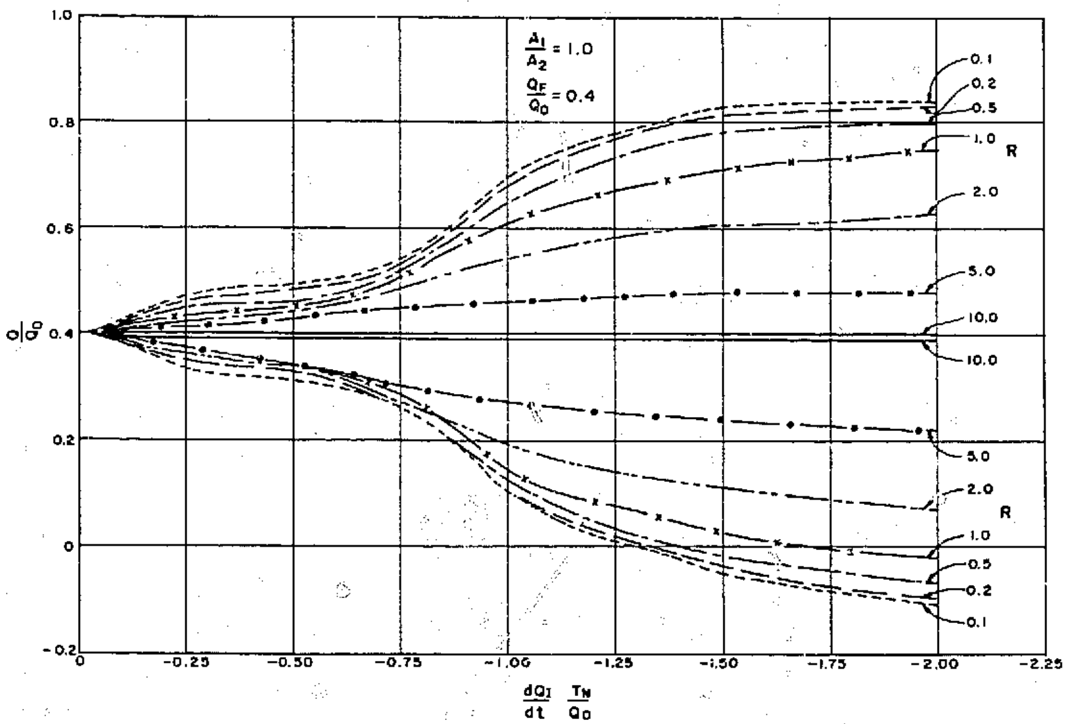


Figure 18.

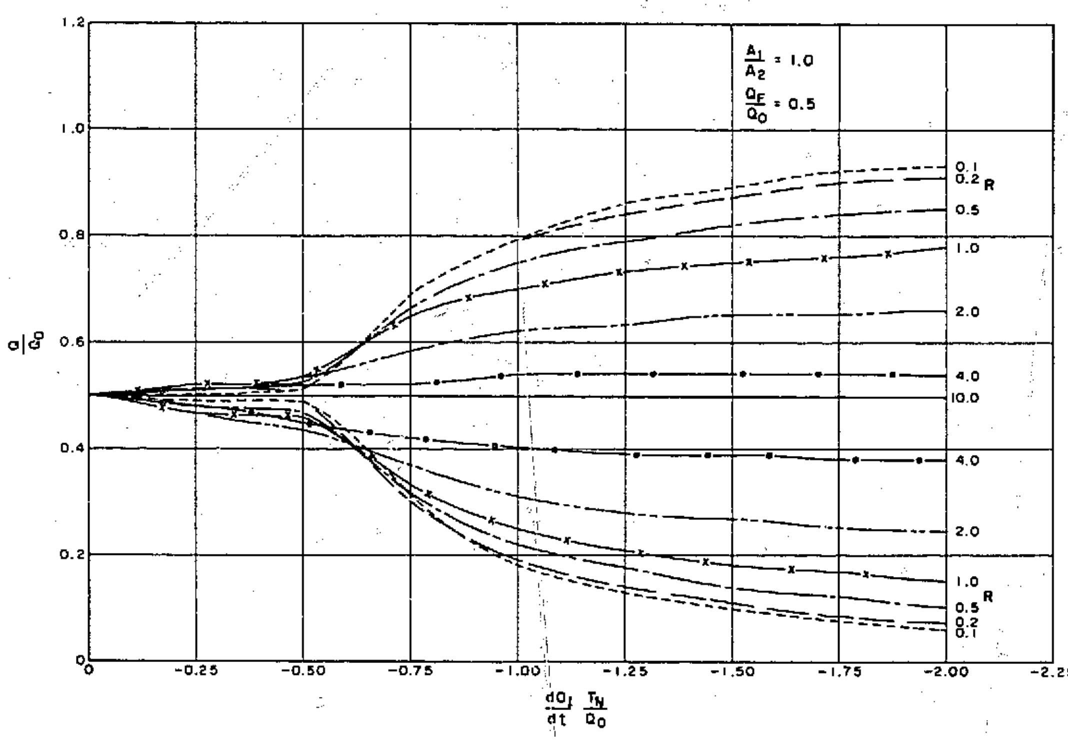
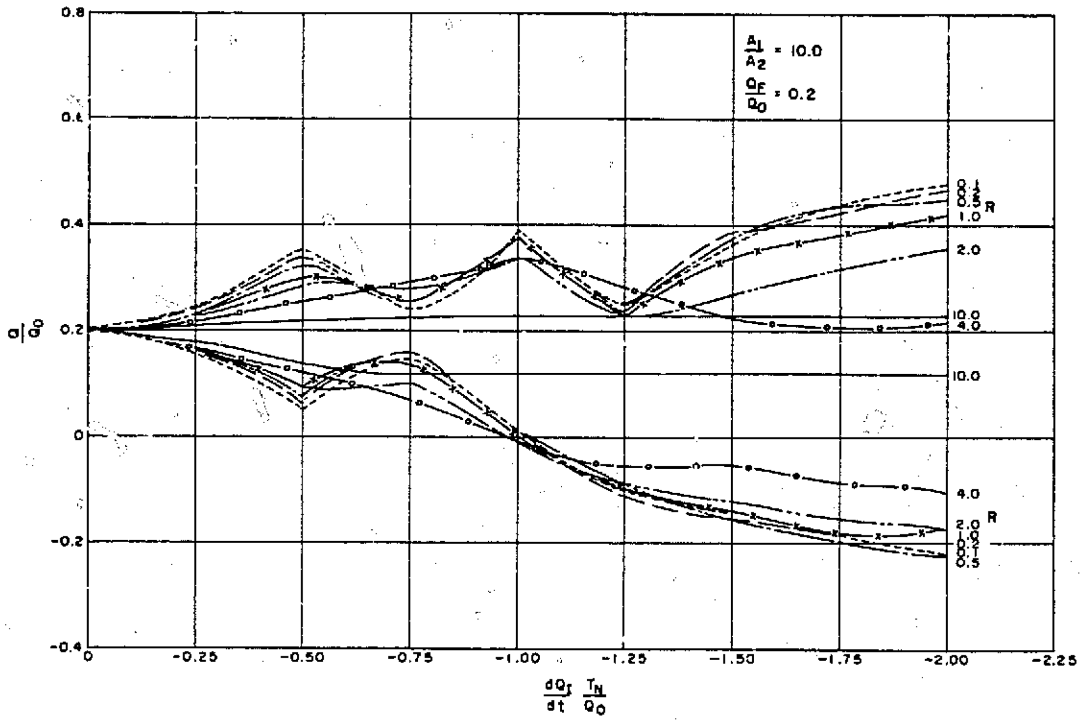


Figure 19.

Range of dimensionless discharge as a function of the dimensionless discharge cutback ratio.



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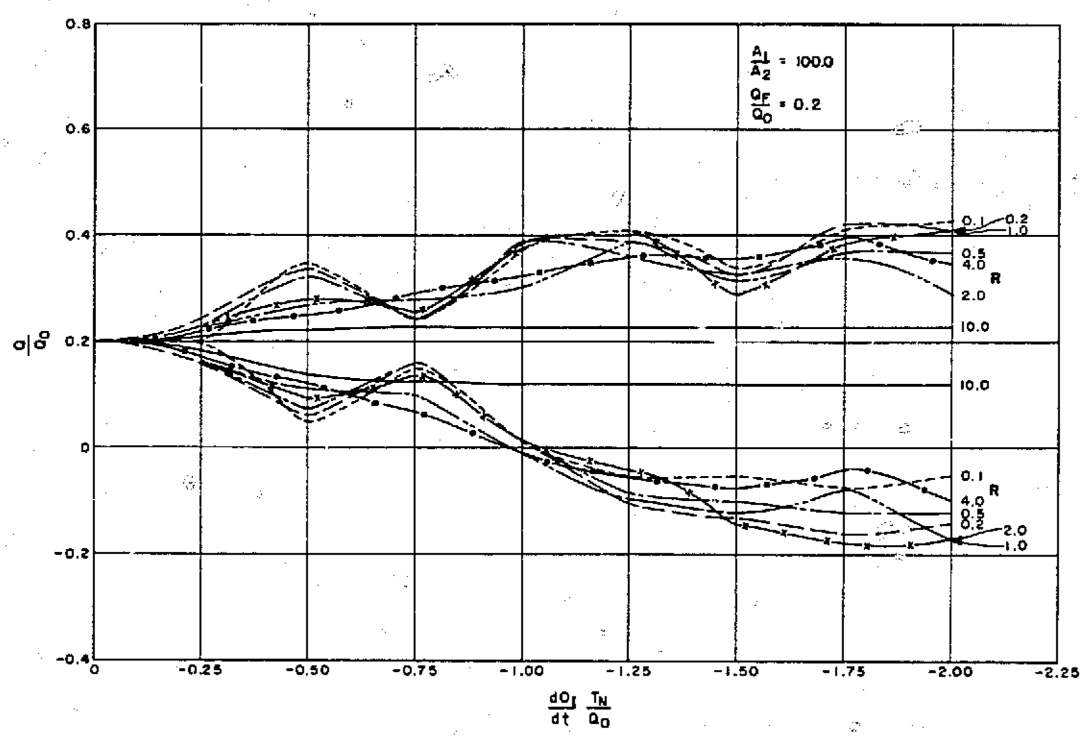
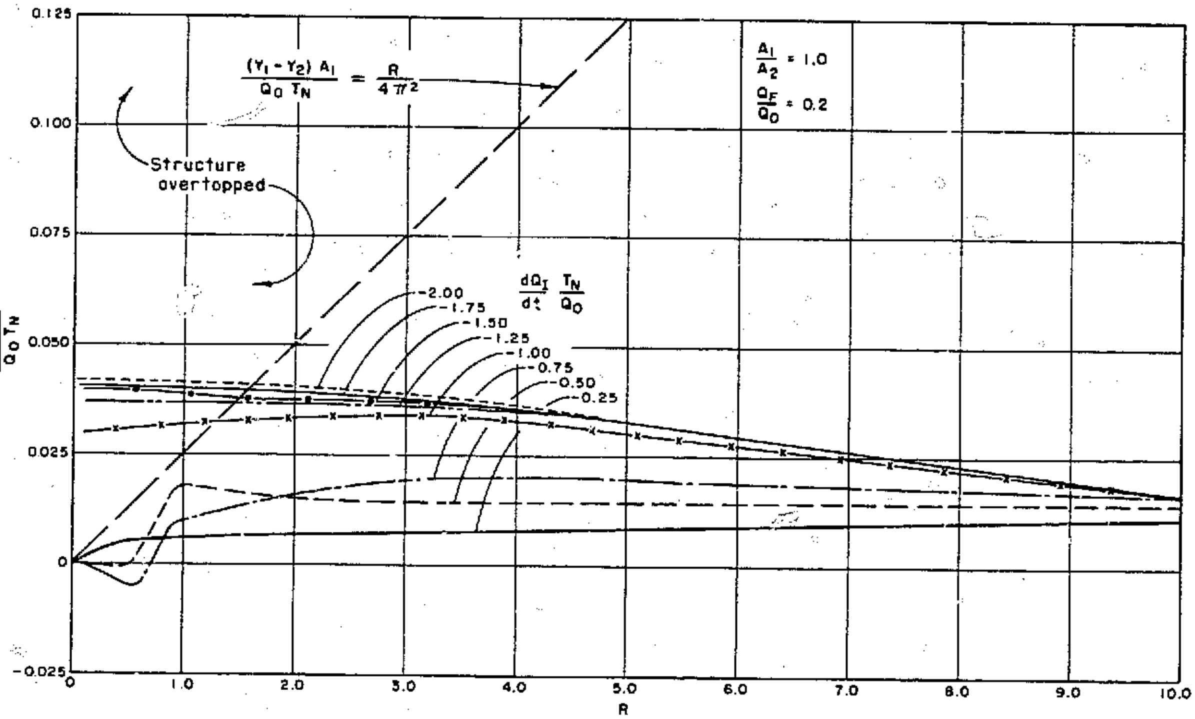


Figure 21.

Range of dimensionless discharge as a function of the dimensionless discharge cutback ratio.



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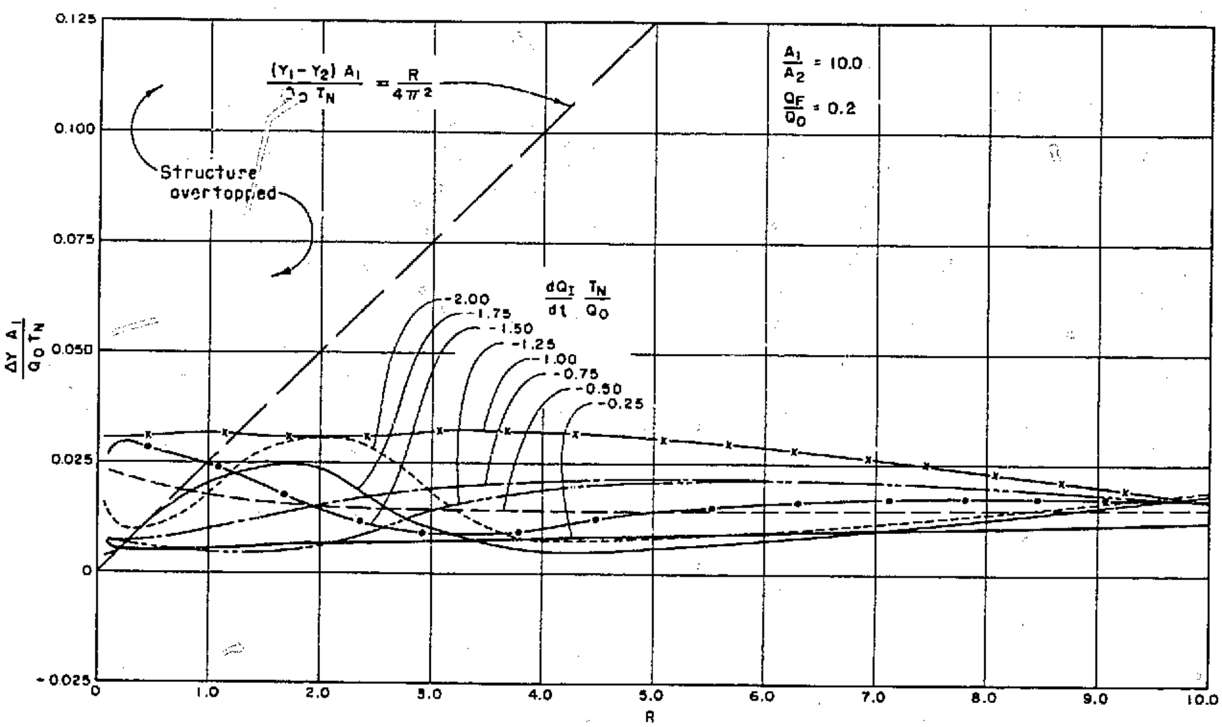


Figure 23.

Surge amplitudes as a function of the resistance coefficient.

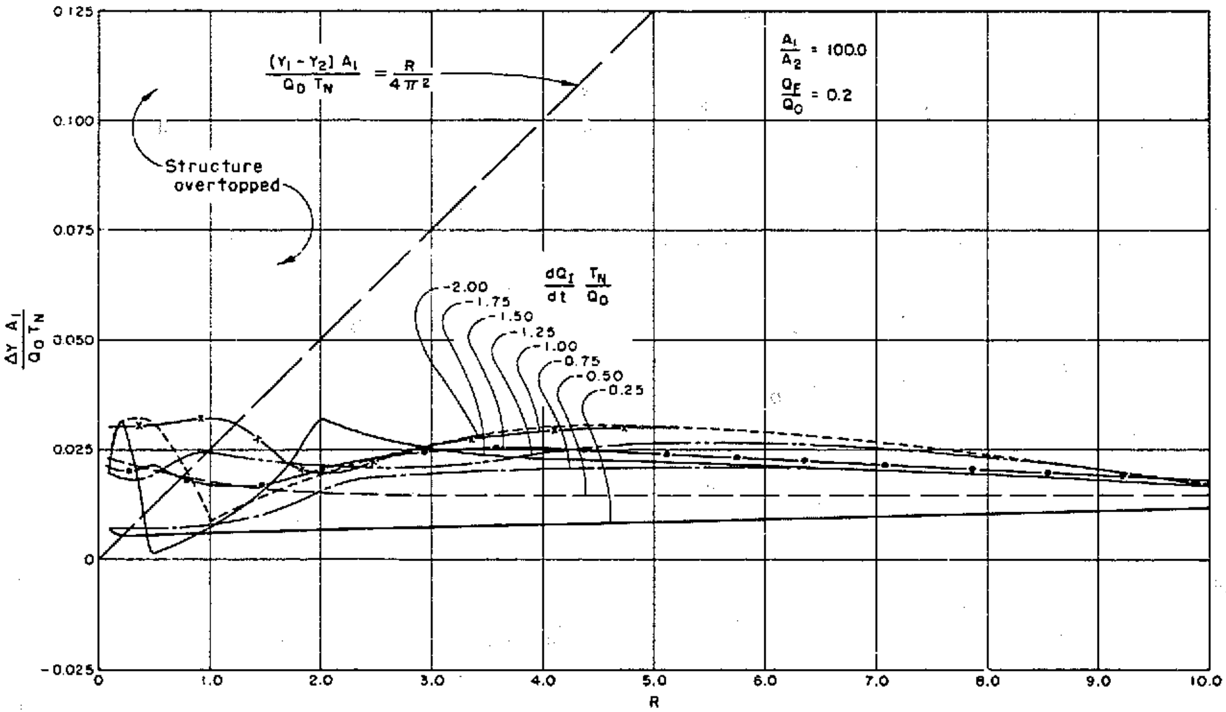


Figure 24.

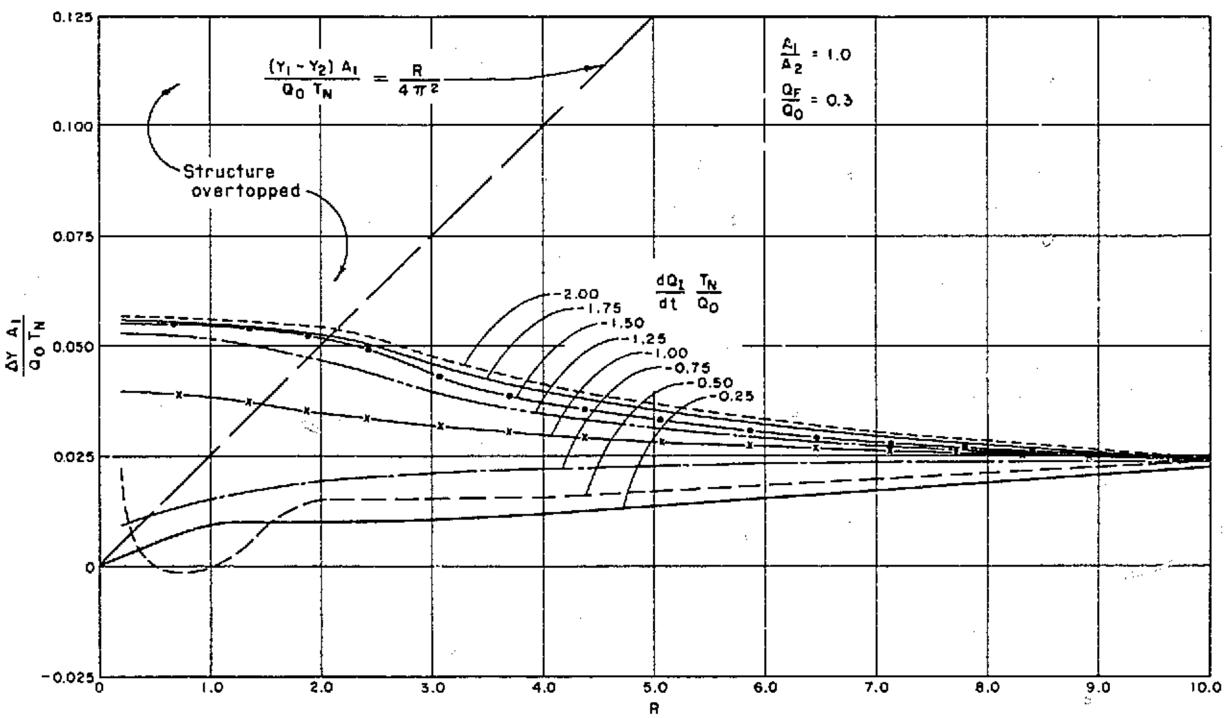
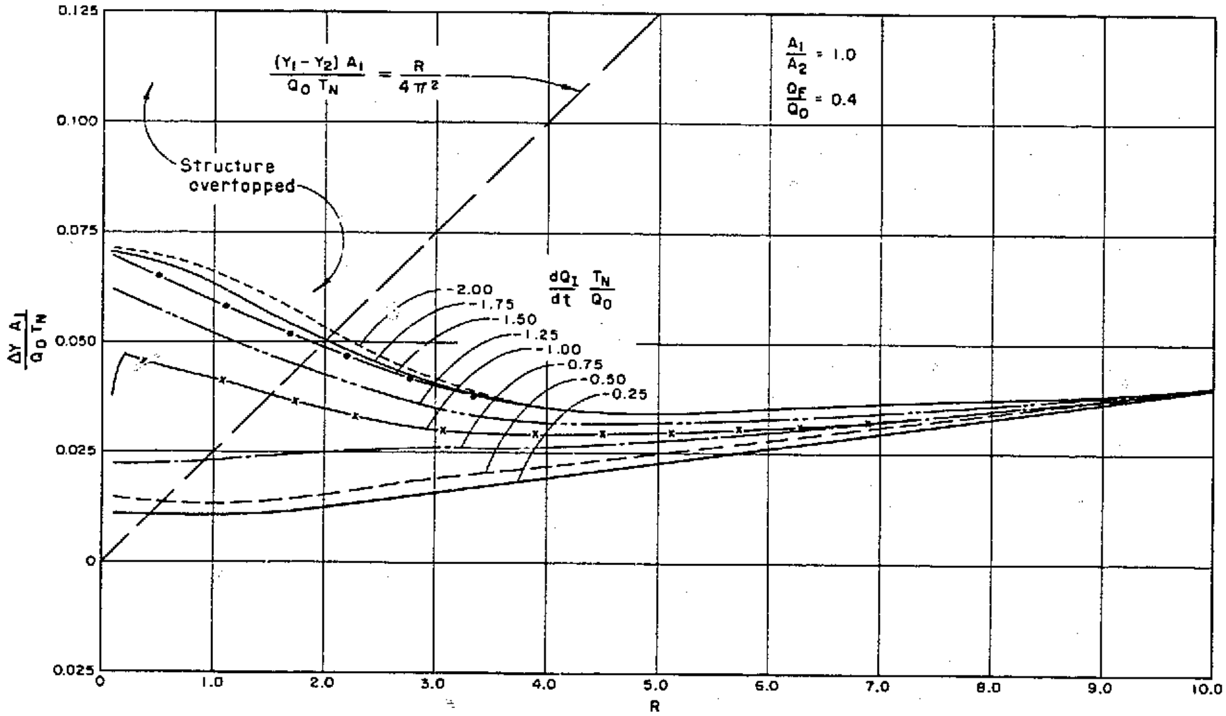


Figure 25.

Surge amplitudes as a function of the resistance coefficient.



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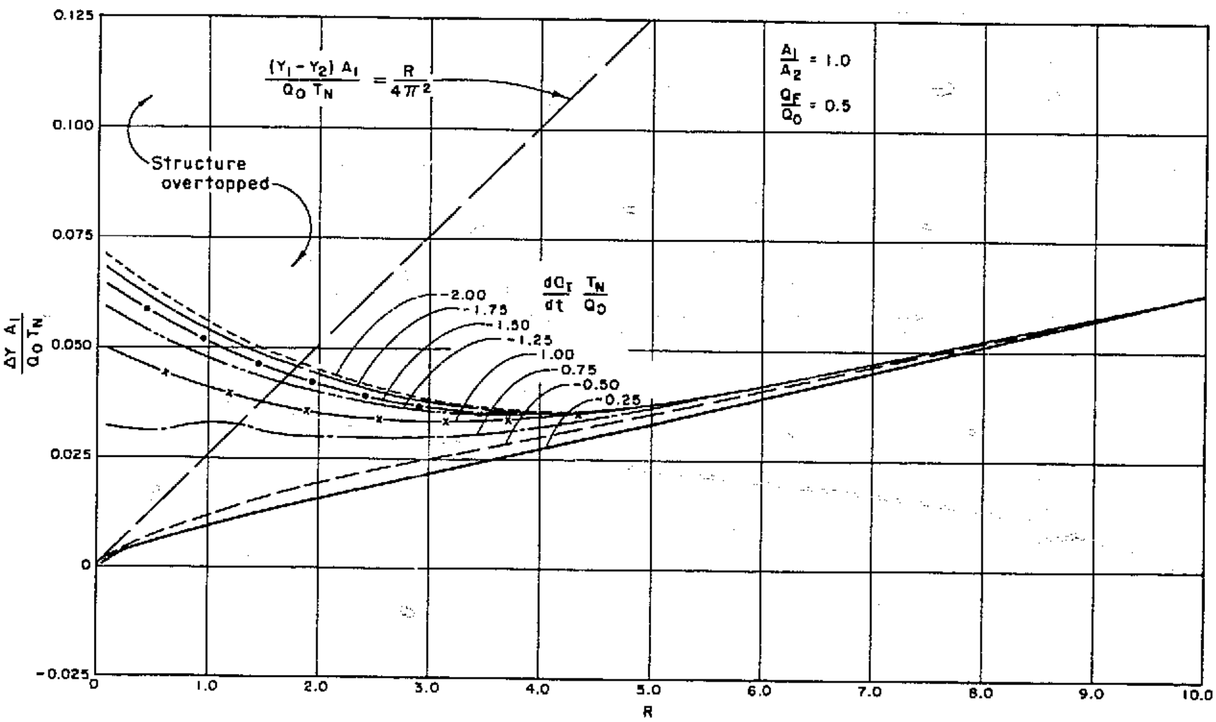

Figure 27.

Surge amplitudes as a function of the resistance coefficient.

APPENDIX

COMPUTER PROGRAMS FOR DIMENSIONLESS AQUADUCT PARAMETERS AND AQUADUCT SURGE CHARACTERISTICS

BUREAU OF RECLAMATION ENGINEERING COMPUTER SYSTEM

CLASSIFICATION - HYDRAULICS

PROGRAM DESCRIPTION - PRC 1532 - PIPAP

PROGRAM BY

J. J. CASSIDY

DOCUMENTATION BY

H. T. FALVEY

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

GENERAL RESEARCH

HYDRAULICS BRANCH

ENGINEERING AND RESEARCH CENTER

DENVER, COLORADO

04/24/72

T A B L E   O F   C O N T E N T S

(A)

PROGRAM TITLE

DIMENSIONLESS AQUEDUCT PARAMETERS

PURPOSE

THE PROGRAM CONVERTS BASIC DIMENSIONS OF A U-TUBE SHAPED PIPELINE OR AQUEDUCT REACH TO DIMENSIONLESS QUANTITIES. THE QUANTITIES ARE USEFUL IN EXAMINING SURGE CHARACTERISTICS OF THE AQUEDUCT SYSTEM AS OUTLINED IN REPORT REC-ERC-72-XX BY J.J.CASSIDY. THE QUANTITIES ARE ALSO INPUT VARIABLE TO PRO-1532 AQSURG.

METHOD

THE PROGRAM COMBINES THE GIVEN QUANTITIES INTO DIMENSIONLESS PARAMETERS GIVEN IN THE ABOVE REFERENCE USING ELEMENTARY MATHEMATICAL OPERATIONS.

INPUT-OUTPUT

THE INPUT CONSISTS OF: THE STATION, DISCHARGE, RESISTANCE COEFFICIENT, PERTINENT ELEVATIONS AND PIPELINE DIAMETERS.

THE OUTPUT CONSISTS OF: THE STATION, DESIGN DISCHARGE, THE SIGNIFICANT DIMENSIONLESS PARAMETERS.

LIMITATIONS

THE PROGRAM IS NOT LIMITED BY THE NUMBER OF INPUTS. HOWEVER, THEY MUST ALL BE NON ZERO.

BUREAU OF RECLAMATION ENGINEERING COMPUTER SYSTEM

CLASSIFICATION - HYDRAULICS

USER'S MANUAL - PRO 1532 - PIPAR

PROGRAM BY

J. J. CASSIDY

DOCUMENTATION BY

H. T. FALVEY

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

GENERAL RESEARCH

HYDRAULICS BRANCH

ENGINEERING AND RESEARCH CENTER

DENVER, COLORADO 04/24/72

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(A)

PROGRAM TITLE

DIMENSIONLESS AQUEDUCT PARAMETERS

GENERAL INFORMATION

IN GENERAL. TWO FLOW CONDITIONS NEED TO BE EXAMINED WHEN ANALYZING A PIPELINE OR AQUEDUCT SYSTEM FOR SURGING. THESE ARE A SINUSOIDAL INPUT AND A RAMP INPUT. CURVES FOR BOTH OF THESE CONDITIONS HAVE BEEN PREPARED AND THEY ARE APPLICABLE TO ANY PIPELINE SYSTEM WHICH HAS U-TUBE SHAPED REACHES. IN ORDER TO USE THE CURVES. THE DATA DESCRIBING THE PHYSICAL PROPERTIES OF THE REACHES MUST ALSO BE IN DIMENSIONLESS FORM.

THIS PROGRAM USES THE BASIC PIPELINE GEOMETRY TO COMPUTE THE NECESSARY DIMENSIONLESS PARAMETER TO BE USED IN THE ANALYSIS

INPUT

THE INPUT DATA ARE PUNCHED ON TWO CARDS. A HEADER OR PROJECT TITLE CARD AND A DATA DESCRIPTION CARD.

HEADER CARD

THE HEADER CARD IS FILLED OUT AS FOLLOWS:

COLUMNS 1-80 THE TITLE OR DESCRIPTION OF THE PROJECT IS ARRANGED TO BE CENTERED IN THE 80 SPACES ON THE CARD.

DATA DESCRIPTION CARD

THE DATA DESCRIPTION CARD IS FILLED OUT AS FOLLOWS:

COLUMNS 1-6 STATION OF THE UPSTREAM LEG OF THE PIPE, STA

7-12 DESIGN DISCHARGE IN REACH, Q

13-18 FRICTION LOSS COEFFICIENT FROM MOODY DIAGRAM, RK

19-24 MAXIMUM UPSTREAM ELEVATION, ELA

25-30 MINIMUM UPSTREAM ELEVATION, ELB

31-36 MINIMUM DOWNSTREAM ELEVATION, ELC

37-42 MAXIMUM DOWNSTREAM ELEVATION, ELD

43-48 UPSTREAM VERTICAL PIPE DIAMETER, D1

47-54 LENGTH OF HORIZONTAL PIPE WITH DIAMETER D2, P2

53-60 HORIZONTAL PIPE DIAMETER, D2

61-66 LENGTH OF HORIZONTAL PIPE WITH DIAMETER D3, P3

67-72 HORIZONTAL PIPE DIAMETER, D3. IF THERE IS ONLY ONE DIAMETER OF HORIZONTAL PIPE, THEN P3=0. THE DIAMETER OF THE DOWNSTREAM VERTICAL PIPE IS SUBTITUTED FOR D3.

SUBMITTAL INSTRUCTIONS

THE FIRST CARD IN THE DATA SET SHOULD BE THE HEADER CARD. THE NEXT N CARDS ARE THE DATA DESCRIPTION CARDS. WHERE N IS ANY POSITIVE INTEGER.

OUTPUT

THE DATA OUTPUT IS IN THE FORM OF A TABLE WHERE THE COLUMN HEADINGS ARE DEFINED AS FOLLOWS:

STA = STATION OF VERTICAL CENTERLINE AT UPSTREAM LEG OF REACH

Q = DESIGN DISCHARGE IN REACH

SUM(L/A) = SUMMATION OF ALL LENGTH TO AREA RATIOS IN REACH

SUMK = SUMMATION OF FRICTION COEFFICIENT TIMES LENTH TO AREA

RATIOS RELATIVE TO THE HORIZONTAL PIPE AREA IN THE REACH.

A1/A4 = AREA RATIO BETWEEN UPSTREAM AND DOWNSTREAM VERTICAL LEGS.

R = DIMENSIONLESS DAMPING(FRICTION) FACTOR

TN = NATURAL PERIOD OF THE REACH

Y1M = MAXIMUM UPSTREAM WATER SURFACE IN DIMENSIONLESS FORM

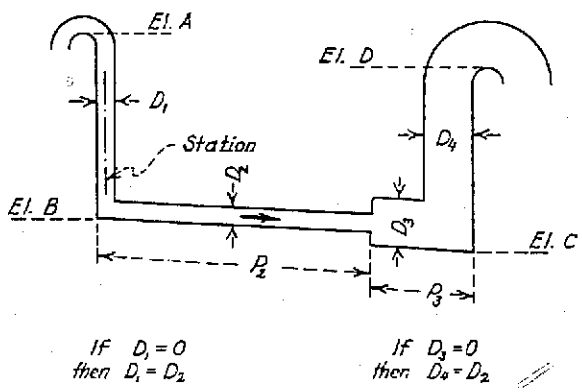
Y1B = MINIMUM UPSTREAM WATER SURFACE IN DIMENSIONLESS FORM

Y2M = MAXIMUM DOWNSTREAM WATER SURFACE IN DIMENSIONLESS FORM

Y2B = MINIMUM DOWNSTREAM WATER SURFACE IN DIMENSIONLESS FORM

APPENDIX A

DEFINITION SKETCH OF PIPE REACH



APPENDIX B

FORTRAN LISTING

```
C
C     DIMENSIONLESS AQUEDUCT PARAMETERS
C
      DIMENSION T(16)
      READ(2,1)(T(N),N=1,16)
    1 FORMAT(16A5)
      CALL EOF(NO)
      IF(NO.EQ.1)CALL EXIT
      WRITE(3,200)(T(N),N=1,16)
  200 FORMAT(1H1,1X,16A5)
      WRITE(3,201)
  201 FORMAT(66H  STA.  SUM(L/A)  SUMK  A1/A4  R     TN     Y1M     Y1B
     1Y2M     Y2B//)
    7 READ(2,100)STA,QO,RK,ELA,ELB,ELC,ELD,D1,P2,D2,P3,D3
  100 FORMAT(2F6.0,10F6.1)
      P1=ELA-ELB
      P4=ELD-ELC
      A2=3.1416*D2*D2/4.
      F2=1./(2.*ALOG10(D2/RK)+1.14)**2
      IF(D1)3,2,6
    2 D1=D2
    6 A1=3.1416*D1*D1/4.
      F1=1./(2.*ALOG10(D1/RK)+1.14)**2
      IF(D3)3,3,4
    3 D4=D2
      A4=A2
      F4=F2
      SUMLA=P1/A1+P2/A2+P4/A4
      SUMK=F1*(A2/A1)**2*P1/D1+F2*P2/D2+F4*(A2/A4)**2*P4/D4
      GO TO 5
    4 A3=3.1416*D3*D3/4.
      D4=D3
      A4=A3
      SUMLA=P1/A1+P2/A2+P3/A3+P4/A4
      F3=1./(2.*ALOG10(D3/RK)+1.14)**2
      SUMK=F1*(A2/A1)**2*P1/D1+F2*P2/D2+F3*(A2/A3)**2*(P3+P4)/D3
    5 TN=6.2832*SQRT(A1*SUMLA/32.2)
      R=SUMK*TN*QO/(2.*A2*A2*SUMLA)
      PAR=A1/(QO*TN)
      Y1M=(ELA+D2-ELD)*PAR
      Y1B=(ELD-ELB)*PAR
      Y2M=D4*PAR
      Y2B=(ELD-ELC)*PAR
      AR=A1/A4
      WRITE(3,202)STA,QO,SUMLA,SUMK,AR,R,TN,Y1M,Y1B,Y2M,Y2B
  202 FORMAT(F6.0,F8.0,2F8.1,3F7.1,4F8.4)
      GO TO 7
      END
```

BUREAU OF RECLAMATION ENGINEERING COMPUTER SYSTEM

CLASSIFICATION - HYDRAULICS

PROGRAM DESCRIPTION - PRO 1532 - AQSURG

PROGRAM BY

J. J. CASSIDY

DOCUMENTATION BY

H. T. FALVEY

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

GENERAL RESEARCH

HYDRAULICS BRANCH

ENGINEERING AND RESEARCH CENTER

DENVER, COLORADO

04/24/72

TABLE OF CONTENTS

(A)

PROGRAM TITLE

AQUEDUCT SURGE CHARACTERISTICS

PURPOSE

TO SOLVE THE NONLINEAR SURGE EQUATION OF FLOW IN A U-TUBE SHAPED PIPELINE SYSTEM FOR A GIVEN DECREASE IN THE INFLOW RATE.

METHOD

THE PROGRAM SOLVES THE ONE-DIMENSIONAL EQUATION OF MOTION AND THE CONTINUITY EQUATION SIMULTANEOUSLY BY A RUNGE-KUTTA METHOD OF NUMERICAL INTEGRATION.

INPUT-OUTPUT

THE INPUT CONSISTS OF TWO CARDS WHICH CONTAIN: DIMENSIONLESS AQUEDUCT PARAMETERS, AND DIMENSIONLESS FLOW PARAMETERS.

THE OUTPUT CONSISTS OF: FLOW AND WATER SURFACE PARAMETERS AT DISCRETE TIME INTERVALS.

LIMITATIONS

THE SIZE OF THE TIME INTERVAL MUST BE CHOSEN TO BE LESS THAN 0.2 TIMES THE NATURAL PERIOD OF THE PIPELINE REACH BEING ANALYZED.

BUREAU OF RECLAMATION ENGINEERING COMPUTER SYSTEM

CLASSIFICATION - HYDRAULICS

USER'S MANUAL - PRC 1532 - AQSURG

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T A B L E   O F   C O N T E N T S

(A)

PROGRAM TITLE

AQUEDUCT SURGE CHARACTERISTICS

GENERAL INFORMATION

LOW PRESSURE PIPELINES OR AQUEDUCTS ARE DESIGNED TO OPERATE WITH THE HYDRAULIC GRADE LINE ROUGHLY PARALLEL TO THE GROUND SURFACE AT DESIGN DISCHARGE. AVAILABLE INFORMATION INDICATES THAT THESE PIPELINES WILL UNDERGO UNDAMPED FLUID MOTION WHEN OPERATED AT FLOW RATES LESS THAN THE DESIGN VALUE. AN ANALYSIS OF THE SURGE PROBLEM IS GIVEN IN REPORT REC-ERC-72-XX BY J.J.CASSIDY.

THIS PROGRAM CAN BE USED TO PREDICT THE FLUID MOTION IN A PIPELINE REACH DELINEATED BY UPSTREAM AND DOWNSTREAM CHECK STRUCTURES WHEN THE INFLOW IS UNIFORMLY DECREASED FROM ONE DISCHARGE VALUE TO ANOTHER. THE SIGNIFICANT REACH PARAMETERS ARE INSERTED INTO THE PROGRAM IN DIMENSIONLESS FORM.

SINCE THE SOLUTION INVOLVES THE SIMULTANEOUS SOLUTION OF THE EQUATION OF MOTION AND THE CONTINUITY EQUATION BY A RUNGE-KUTTA METHOD, THE METHOD DOES HAVE A LIMITATION. THE TIME INTERVAL SHOULD BE GREATER THAN 0.2 TIMES THE NATURAL PERIOD OF THE REACH. THIS LIMITATION INSURES THAT SUFFICIENT POINTS ARE USED TO DEFINE THE OSCILLITORY MOTION. PRO-1532 - PIPAR IS USED TO PREPARE THE INPUT DATA.

INPUT

TWO CARDS ARE USED TO INPUT THE DATA IN COLUMNS 1-80.

FIRST CARD

THE FIRST CARD IS FILLED OUT AS FOLLOWS:

COLUMNS 1-10 THE DIMENSIONLESS INFLOW, QI

11-20 THE INITIAL WATER SURFACE ELEVATION AT THE DOWNSTREAM END IN DIMENSIONLESS FORM, Y2I

21-30 THE MAXIMUM POSSIBLE UPSTREAM WATER SURFACE ELEVATION IN DIMENSIONLESS FORM, Y1M

31-40 THE MAXIMUM POSSIBLE DOWNSTREAM WATER SURFACE ELEVATION IN DIMENSIONLESS FORM, Y2M

41-50 THE FINITE TIME INTERVAL, DT

51-60 AREA RATIO OF A1/A2, AR

61-70 MAXIMUM TIME OVER WHICH THE ANALYSIS IS MADE, TMAX

SECOND CARD

THE SECOND CARD IS FILLED OUT AS FOLLOWS:

COLUMNS 1-10 THE DIMENSIONLESS CUTBACK RATE D(QI/QO)/D(T/TN), DQI

11-20 THE FINAL DIMENSIONLESS DISCHARGE AFTER CUTBACK, QF

21-30 THE DIMENSIONLESS DAMPING(FRICTION) COEFFICIENT, R

31-40 THE MINIMUM POSSIBLE UPSTREAM DIMENSIONLESS WATER SURFACE ELEVATION, Y1MIN

41-50 THE MINIMUM POSSIBLE DOWNSTREAM DIMENSIONLESS WATER SURFACE ELEVATION, Y2MIN

SUBMITTAL INSTRUCTIONS

THE NUMBER OF SETS OF DATA SUBMITTED IS NOT LIMITED. HOWEVER, EACH SET MUST CONTAIN TWO DATA CARDS IN THE ORDER MENTIONED ABOVE.

OUTPUT

THE BASIC INPUT VARIABLES ARE PRINTED. THIS IS FOLLOWED BY A

LIST OF DISCHARGES AND WATER SURFACE ELEVATIONS FOR THE TIME INTERVAL WHICH WAS CHOSEN.

APPENDIX A

FORTRAN LISTING

```
C  PROGRAM FOR SOLUTION OF SURGE EQUATION.
C
  102 READ(2,100)QI,Y2I,Y1M,Y2M,DT,AR,TMAX
  100 FORMAT(7F10.2)
      CALL EOF(J)
      IF(J.EQ.1) CALL EXIT
      READ(2,100)DQI,QF,R,Y1MIN,Y2MIN
C  COMPUTE PARAMETERS FOR DIFFERENTIAL EQUATION
      S=4.*3.1416*3.1416
      Q=QI
C  DETERMINE STEADY STATE UPSTREAM WATER ELEVATION.
      Y1I=R*Q*Q/S+Y2I
      IF(Y1M.LE.Y1I)Y1M= Y1I
C  WRITE OUT HEADINGS.
  101 WRITE(3,203)QI,DQI,QF,R,AR,Y1I,Y2I,Y1M,Y2M
  203 FORMAT(1H1,4H QI=,F5.2,7H   DQI=,F5.2,6H   QF=,F4.2,5H   R=,F6.2,9
     1H   A1/A2=,F7.2//,5H Y1I=,F5.2,7H   Y2I=,F5.2,7H   Y1M=,F5.2,7H
     2Y2M=,F5.2///)
      WRITE(3,201)
  201 FORMAT(51H       T           Q          QI          Y1          Y2//)
C  SET INITIAL CONDITIONS
   18 Y2=Y2I
      Y1=Y1I
      T=0.
C  WRITE OUT CONDITIONS.
    1 WRITE(3,200)T,Q,QI,Y1,Y2
  200 FORMAT(3F10.2,2F10.3)
      Z2=Y2
C  CHECK TO SEE IF SOLUTION IS COMPLETE.
      IF(T-TMAX)2,2,102
C  SOLVE
    2 QB=Q
      DYT1=QI-Q
      DQT1=S*(Y1-Y2)-R*ABS(Q)*Q
      IF(QI-QF)14,14,13
   13 QI=QI+DQI*DT
      IF((QI-QF).LE.0.)QI= QF
      GO TO 15
   14 QI=QF
   15 DYT2=QI-Q
```



```
      Y1=Y1+(DYT1+DYT2)*DT/2.
      IF(Y1.LE.Y1MIN)Y1=Y1MIN
      DQT2=S*(Y1-Y2)-R*ABS(Q)*Q
      Q=Q+(DQT1+DQT2)*DT/2.
      T=T+DT
    3 Y2=Y2+DT*AR*(Q+QB)/2.
      IF(Y2.LE.Y2MIN)Y2=Y2MIN
      IF(Y2-Y2I)5,5,20
   20 Y2=Y2I
    5 IF(Y1-Y1M)7,7,6
    6 Y1=Y1M
    7 GO TO 1
      END
```

7-1750 (3-71)
Bureau of Reclamation

CONVERSION FACTORS–BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, E 380-68) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given in the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg, that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Where approximate or nominal English units are used to express a value or range of values, the converted metric units in parentheses are also approximate or nominal. Where precise English units are used, the converted metric units are expressed as equally significant values.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
LENGTH		
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
Inches	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
Feet	0.3048 (exactly)*	Meters
Feet	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
Miles	1.609344 (exactly)	Kilometers
AREA		
Square inches	6.4516 (exactly)	Square centimeters
Square feet	*929.03	Square centimeters
Square feet	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	*0.40469	Hectares
Acres	*4,046.9	Square meters
Acres	*0.0040469	Square kilometers
Square miles	2.58999	Square kilometers
VOLUME		
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
CAPACITY		
Fluid ounces (U.S.)	29.5737	Cubic centimeters
Fluid ounces (U.S.)	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
Liquid pints (U.S.)	0.473166	Liters
Quarts (U.S.)	*946.358	Cubic centimeters
Quarts (U.S.)	*0.946331	Liters
Gallons (U.S.)	*3,785.43	Cubic centimeters
Gallons (U.S.)	3.78543	Cubic decimeters
Gallons (U.S.)	3.78533	Liters
Gallons (U.S.)	*0.00378543	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
Gallons (U.K.)	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	*764.55	Liters
Acre-feet	*1,233.5	Cubic meters
Acre-feet	*1,233,500	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
MASS		
Grains (1/7,000 lb)	64.79891 (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
Short tons (2,000 lb)	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
FORCE/AREA		
Pounds per square inch	0.070307	Kilograms per square centimeter
Pounds per square inch	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
Pounds per square foot	47.8803	Newtons per square meter
MASS/VOLUME (DENSITY)		
Ounces per cubic inch	1.72999	Grams per cubic centimeter
Pounds per cubic foot	16.0185	Kilograms per cubic meter
Pounds per cubic foot	0.0160185	Grams per cubic centimeter
Tons (long) per cubic yard	1.32894	Grams per cubic centimeter
MASS/CAPACITY		
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
BENDING MOMENT OR TORQUE		
Inch-pounds	0.011521	Meter-kilograms
Inch-pounds	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
Foot-pounds	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
VELOCITY		
Feet per second	30.48 (exactly)	Centimeters per second
Feet per second	0.3048 (exactly)*	Meters per second
Feet per year	$*0.965873 \times 10^{-6}$	Centimeters per second
Miles per hour	1.609344 (exactly)	Kilometers per hour
Miles per hour	0.44704 (exactly)	Meters per second
ACCELERATION*		
Feet per second^2	*0.3048	Meters per second^2
FLOW		
Cubic feet per second (second-feet)	*0.028317	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
FORCE*		
Pounds	*0.453592	Kilograms
Pounds	*4.4482	Newtons
Pounds	$*4.4482 \times 10^5$	Dynes

Table II—Continued

Multiply	By	To obtain
WORK AND ENERGY*		
British thermal units (Btu)	*0.252	Kilogram calories
British thermal units (Btu)	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	*1.35582	Joules
POWER		
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
HEAT TRANSFER		
Btu in./hr ft^2 degree F (k, thermal conductivity)	1.442	Milliwatts/cm degree C
Btu in./hr ft^2 degree F (k, thermal conductivity)	0.1240	Kg cal/hr m degree C
Btu ft/hr ft^2 degree F	*1.4880	Kg cal m/hr m^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	0.568	Milliwatts/cm^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	4.882	Kg cal/hr m^2 degree C
Degree F hr ft^2/Btu (R, thermal resistance)	1.761	Degree C cm^2/milliwatt
Btu/lb degree F (c, heat capacity)	4.1868	J/g degree C
Btu/lb degree F	*1.000	Cal/gram degree C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
Ft^2/hr (thermal diffusivity)	*0.09290	M^2/hr
WATER VAPOR TRANSMISSION		
Grains/hr ft^2 (water vapor) transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

Multiply	By	To obtain
Cubic feet per square foot per day (seepage)	*304.8	Liters per square meter per day
Pound-seconds per square foot (viscosity)	*4.8824	Kilogram second per square meter
Square feet per second (viscosity)	*0.092903	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	*35.3147	Millicuries per cubic meter
Milliamps per square foot	*10.7639	Milliamps per square meter
Gallons per square yard	*4.527219	Liters per square meter
Pounds per inch	*0.17858	Kilograms per centimeter

ABSTRACT

Review and analysis of available information regarding surging in low-pressure pipelines indicated that such lines, designed to operate with the hydraulic gradeline roughly parallel to the ground surface at design discharge, will undergo a fluid motion which is underdamped when operating at flow rates below the design value. An analytical study of the dynamics of surging flow enumerated the significant design and operation parameters. Numerical values of these parameters were obtained and are tabulated in the report. Through use of these tabulated values, low-pressure pipelines may be designed to minimize the amplitude of flow oscillations.

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REC-ERC-72-28
Cassidy, J J
CONTROL OF SURGING IN LOW-PRESSURE PIPELINES
Bur Reclam Rep REC-ERC-72-28, Div Gen Res, September 1972. Bureau of Reclamation, Denver, 37 p, 27 fig, 3 tab, 8 ref, append

DESCRIPTORS—/ *pipelines/ *surges/ closed conduit flow/ fluid flow/ fluid mechanics/ hydraulics/ oscillations/ *Water pipes/ momentum
IDENTIFIERS—/ *pipeline surges/ Canadian River Project, Tex

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