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FEEDBACK CONTROL OF IRRIGATION CANAL SYSTEMS

BY

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FINAL REPORT

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FEEDBACK CONTROL OF IRRIGATION CANAL SYSTEMS
using the Harder-Smith control algorithm

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TABLE OF CONTENTS

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KEY WORD DESCRIPTORS

Irrigation canals, automatic control, computer programs.

ABSTRACT

A new computational algorithm is described, known as the Harder-Smith method, for stabilizing feedback control of irrigation canals. Assumed are a means of adjusting the position of the upstream gate of a canal reach, a means of sensing the water surface elevation at the downstream end of the reach, and the translation of these two variables to signals that can be processed by computer. The computer program uses the canal characteristics and a given value of offset of the water surface (between zero and full flow capacity) to determine the time constants and gain of a quasi-proportional controller. In addition, a transient suppressor program allows a near-instantaneous response to be made to changes in turnout flows at the lower end of the canal.

INTRODUCTION

This is the final report under contract 14-06-D-7674 with the Bureau of Reclamation (now the Water and Power Resources Service Agency) for work in the development of feedback control systems for irrigation canals.

The report contains background material on the need and purpose of the work, a brief history of earlier work at the Hydraulics Laboratory, University of California, Berkeley, and a description of the work under this contract. Details of computer programs and theory are given in appendices. Compputer decks for the programs are supplied separately.

BACKGROUND

Irrigation canals typically serve a large number of users whose demands for water must be carefully scheduled in advance so that the capacity of the canal can be fully used. At intervals along the canal check structures, which may be as simple as stop logs dropped into concrete slots, are used to raise the water level locally and to divert water through gates in the canal banks.

Modern canals are usually concrete lined and the water level is controlled at intervals with moveable gates, typically of the radial gate design. In addition to easing the water delivery, the maintenance of a near-constant water level is important to reduce stresses in the lining, since for economic reasons the lining is not usually reinforced with steel.

The moveable gates are motor-driven, and the water levels are sensed and recorded at each gate in a typical modern system. Thus the instrumentation and mechanical means to control the gates in an automatic way are already installed in most cases, and it remains only to develop the control algorithms to make automatic control a reality. This was the inspiration for a series of investigations at the Hydraulics Laboratory, University of California, Berkeley, and supported by the Bureau of Reclamation (now the Water and Power Resources Service Agency). This report is on a second main phase of this investigation.

A BRIEF HISTORY OF INVESTIGATIONS AT THE HYDRUALICS LABORATORY, UNIVERSITY OF CALIFORNIA

The initial concept (Shand, 1968, 1972) of automatic control was that the water level sensors at the downstream end of each reach would signal the gate at the upstream end so that a target water surface elevation would be closely maintained at the lower end. From a "target elevation" (usually that which would obtain at zero flow rate) the upstream gate would introduce flows proportional to the amount of offset of the water surface below the target. Were this relationship linear and were there no time delays in the system, this would be proportional control familiar to control engineering.

The delay time between the gate action and its being sensed at the lower end (which would be the surge travel time in hydraulic terms) alters the negative feedback to positive feedback for the frequency for which the canal length is half a wave length. Thus for this frequency the overall gain of the system must be less than unity. The analysis of this system proceeded by estimating the speed and attenuation of the surges, and their reflection coefficients at the lower end. There was incorporated a low-pass filter so that a higher gain could be used for lower frequencies, and the gain relationship made non-linear to take advantage of the fact that the attenuation of the surges was a non-linear function of the flow in the canal. A degree of "reset" could be incorporated to further increase the gain for frequencies approaching zero. With all the improvements made possible by a computer analysis of the system gain parameters, there was still the limitation imposed by stabilty on the speed of response of the system. This speed was on the order of the surge travel time.

In an attempt to improve the system response time, a new scheme was investigated: the Smith method, invented by Professor Otto J.M. Smith of the University of California, Berkeley. In this method, the lag time that creates the instability in the controlled system is compensated for by a simulated lag in the control system itself. The application to the feedback control of canal gates is as follows:

A desired steady-state relationship between the water level offset and the gate position upstream (which is related to the flow rate through the gate) is established; this can be the same as the relationship used for gain in the earlier Harder-Shand method. However, instead of introducing a low pass filter in the feedback loop, the upstream gate is operated immediately according to the desired relationship. This results in a reponse, consisting of a surge, that traverses the canal length and eventually arrives at the sensor at the lower end. In the meanwhile the control system operates its "lag memory" device to predict the magnitude and time-of-arrival of the surge. In the Smith method there is only one controlled variable; this is the water depth in the present instance. The speed and attenuation of the surge is calculated by a computer program. Ideally, at exactly the time that the water level sensor is detecting the canal surge, the control system has available a prediction of this surge. Thus, through a subtraction, the sensor can be "blinded" to the predicted surge and made not to respond to it. It will still respond to non-predicted transients and continue to control the upstream gate according to them.

This Smith method was developed in theory and computer simulations of its applcation to the Corning Canal were carried out (Shand, 1972). It was estimated that the system could be made to respond on the order of five times as fast as the simple filter-controlled system. No field tests were carried out.

This concluded contract 14-06-200-2337A. That contract was the basis of a number of reports and papers that are listed in the bibliography.

THE CURRENT CONTRACT

Contract 14-06-D-7674 (June, 1975) commenced Phase II of the investigations. This contract was intended to confirm and extend work on the Smith Predictor Method. During the contract negotiations, however, Professor Harder conceived of the new Harder-Smith Method. This seemed so promising that the direction of the investigation was enlarged to include the testing of this new concept.

The Smith Method assumes a single variable is to be controlled. Changes in the water depth are propagated, with a lag, from the upstream gate to the downstream water level sensor via a wave or surge. In the simplification initially employed, only the speed and attenuation of this surge was taken into account. Actually, open channel flow is governed by partial differential equations in two dependent variables, the water depth and the discharge (or the velocity), and an improved prediction can be made if the full equations are solved. This must be done numerically and requires the use of a digital computer.

Again the central problem is to predict the water surface elevation on the basis of past inputs that have produced transients, and to determine the significance of the difference between the predicted and measured values. But because of the fact that there are two dependent variables (the discharge would be the second) the computation is more complex. It can best be explained by an example. Consider the problem of determining the magnitude of a change in a turn-out diversion discharge at the lower end of one reach. This could result from the start up of a pump diverting water from the lower end of the canal, or from a sudden opening of the gate there.

Corresponding to the two unknowns at the gate, there are two equations: the characteristics equation from upstream (known as the C+ characteristic) and the boundary condition imposed by the flow under the gate and the amount of the turnout. In addition, there is the measured water surface elevation. It can be seen that there are theoretically enough relationships to predict the water surface elevation for comparison with the measured. Details of the computations are given in Appendix I.

If there should be a turnout at a location along the canal a similar set of equations will allow the water surface elevation to be calculated, and the turnout value inferred from the difference between the predicted and the measured. This does require a measurement at the turnout location, and may require the installation of additional measurement devices there.

Multiple reaches are joined by their joint boundary conditions as described in Appendix I.

The strategy for optimizing the control parameters for the gate operation is similar to that employed earlier, and is described in Appendix II.

EARLIER REPORTS AND RESULTS UNDER THIS CONTRACT

The initial strategy for solving the equations was based on the assumption that only a limited accuracy mini-computer would be available. Accordingly an explicit computational scheme was developed, and computer programs were devised both for the computation and for simulating the characteristics (limited accuracy and fixed point operation) of a typical minicomputer with only 16 bits. This required much scaling of the variables.

This program and its notes were forwarded to the Bureau on July 19, 1977. Included were the fortran compilations, a list of variables and their locations in common areas, a table of subroutines and the locations where they are used, and a general flow diagram of the computation. In addition, some examples of our preliminary computation applications were included.

EXPERIENCE AT THE BUREAU OF RECLAMATION

Mr. Clark Buyalski of the Bureau Denver Office worked heroically with this program, but eventually it was discovered that in making it adaptable to a mini-computer we had lost too much accuracy. The round-off errors due to the 16 bit accuracy accumulated and when differences were taken as a part of the computation they produced an instability in the prediction of the water surface elevation.

NEW PROGRAMS USING FLOATING POINT COMPUTATION

Since there were still funds, we asked for a no-cost extension and continued work on new programs that used floating point arithmetic. This report describes such programs. We returned to the use of the characteristics computational method for the controller predictor, and to simulate the canal we used a similar scheme but with a different and smaller time step, leading to an increased accuracy. These programs and illustrations for their use are contained in Appendix III.

CONCLUSIONS

With this final phase of the investigation, we believe it will be possible to test the new Harder-Smith method on the model canal system in the Hydraulics Laboratory of the U.S. Water and Power Resources Service Agency in Denver.

Alternatively, it will be possible to test the scheme on an actual canal system like the Tahema-Colusa Canal in Northern California. There the remote control lines and instrumentation that could be the basis for the new scheme are already being installed and tested (August, 1979).

APPENDIX I

COMPUTATION OF TURNOUT DIVERSION

The manner in which the turnout diversion is computed depends on the location along the canal system. The computer aided control system is able to establish the magnitude of the turnout diversion almost as fast as it occurs. It is easiest to describe a canal system composed of a single reach; a system of multiple reaches is slightly more complex.

SINGLE REACH MODEL

The single reach irrigation system shown in Figure 1 has a gate at the downstream end. The boundary conditions for this system are: a pumped discharge at the upstream end and a constant depth downstream from the gate structure. Assume that initially the system is flowing under steady state conditions.

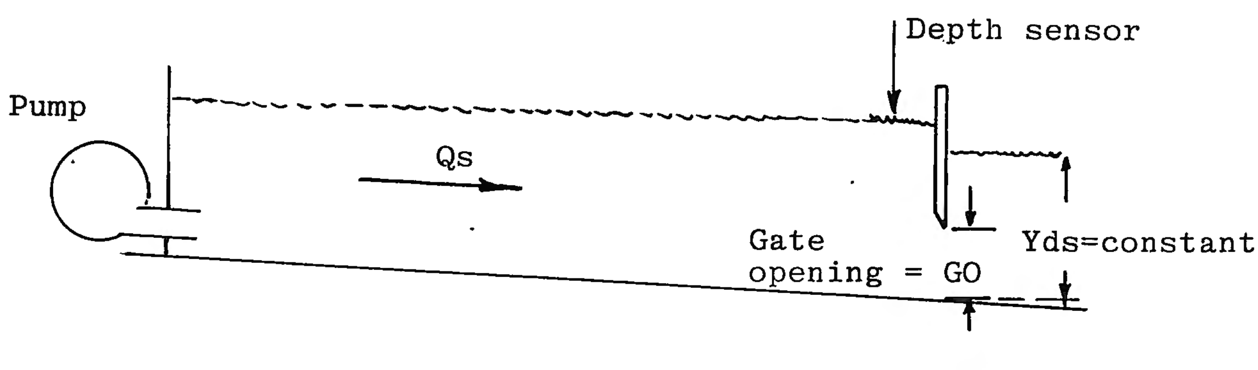

Figure 1: A single reach irrigation canal under steady state conditions

As a turnout diversion is begun just upstream from the gate, Yd1 will begin to drop with time and this will be reflected in the telemetered data. The turnout diversion can then be computed from the C+ characteristic and the gate energy equation.

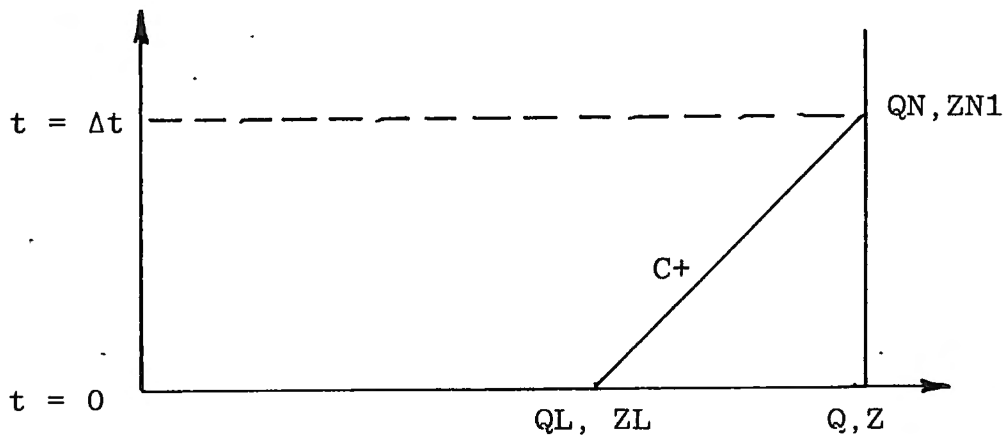

Figure 2: A diagramatic representation of the downstream boundary condition on the x-t plane

The C+ characteristic equation is:

$$QN-QL + B\ (-Q/A + C)\ (ZN1-ZL) = G\Delta t \qquad (1)$$

where: $G = (Q^2B/A^2\ So) - g\ A\ Se$

Se = friction slope computed from Manning's equation

A = cross-section area

B = bottom width

So = bed slope

g = gravitational acceleration

Q = flowrate at previous time step

C = celerity or gravity wave speed

Δt = time step used in computation

QN = flowrate at new time step

The gate energy equation is:

$$Qg = C\ (BR)\ (GO)\ \sqrt{2g\ (Yd1-Yds + DYSYF)} \qquad (2)$$

where: Qg = flowrate under the gate

C = gate discharge coefficient

BR = gate width

GO = gate opening

Yd1 = depth upstream of gate

Yds = depth downstream of gate

DYSYF = SYF - Qg^2 (SYFCO)

SYF = invert syphon drop

SYFCO = syphon loss coefficient

Equation (1) can be solved for QN, assuming ZN1 is known from a measurement:

$$QN = G\Delta t + QL-B\ (-Q/A+C)\ (ZN1-ZL) \qquad (3)$$

The difference between the flows calculated using equations 3 and 2 is then the turnout discharge.

$$DQT = QN-Qg \qquad (4)$$

where DQT = turnout discharge

With DQT known the pumped discharge at the upstream end can be incremented such that it will then deliver Q = QS + DQT, to compensate for the turnout diversion. In time the system will come to a new steady state, with the flow under the gate equal to the original steady state discharge value.

Note that the assumption that the canal was originally in a steady state is not essential to these arguments. The characteristic equation (1) is true for any flow condition.

JOINT BOUNDARY CONDITIONS AT THE JUNCTION OF TWO REACHES

At the junction of two reaches connected by a gate (and possibly also a syphon) there are four equations (see Figure 3).

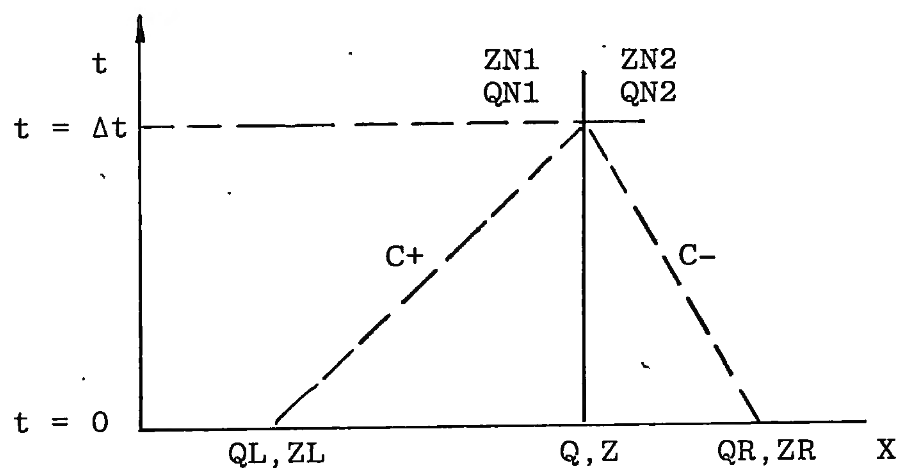

Figure 3: Variables and characteristics lines at the junction of two reaches

(a) C+ characteristic equation (1)

(b) Gate energy equation (2)

(c) Gate continuity

(d) C- characteristic equation (6)

The unknowns in this region are:

(i) Yd1 = downstream depth for reach 1

(ii) Qd1 = downstream flowrate for reach 1

(iii) Yu2 = upstream depth for reach 2

(iv) Qu2 = upstream flowrate for reach 2

This yields four equations in four unknowns and provides a means of interphasing reaches from the standpoint of the numerical solution algorithm.

On the other hand should a turnout diversion occur in reach 1, upstream from the gate, Yd1 will begin to fall and the magnitude of the diversion can be computed in the following manner:

C+ characteristic equation (ZN1 known from telemetered data)

$$QN1 = G\Delta t + QL - B \ (-Q/A+C)\ (ZN1-ZL) \qquad (5)$$

C- characteristic equation:

$$QN2 - QR + B(-Q/A-C)\ (ZN2-ZR) = G\Delta t \qquad (6)$$

Gate continuity states that flow at the new time interval downstream of the gate is equal that which flows under the gate:

$$QN2 = Qg \tag{7}$$

Gate energy

$$Qg = C(BR)\ GO1 \sqrt{2g\ (Yd1 - Yu2 + DYSYF)} \tag{8}$$

Again QN1 can be solved for explicitly from equation 5 since ZN1 is measured. Substituting equation 8 into equation 7 and subsequently into equation 6 and realizing that ZN2 = Yu2 + const, and equation is obtained in Yu2 which can be solved iteratively.

Once Yu2 has been determined to the desired accuracy, the flow under the gate is computed from equation 8 and the turnout diversion is then:

$$DQT = QN1 - Qg$$

With DQT known the pumped discharge at the upstream end can be incremented such that it will deliver Q = Qs + DQT.

GATE OPERATION

In the case of a multiple reach system, gate openings are continuously adjusted in the face of transient conditions. Gate openings appear in the gate energy equation at each gate location. The gate energy equation is employed on the basis of treating the flow in the region of gate structures as a succession of steady states. Each gate is considered to be an energy loss element which relates its discharge to the difference in head between its inlet and outlet. The inlet and outlet values of depth and discharge are then matched with the corresponding values at the beginning and end of adjoining channel section by simultaneously solving the unsteady flow equations and the gate energy equation, as described above.

APPENDIX II

OPTIMUM STRATEGY FOR GATE OPERATION

The Harder-Smith method essentially controls the flow rates by increments; any long-term errors will accumulate and lead to a gradual filling or emptying of the canal. Thus there must be an additional control system, which is fortunately already developed: the Harder-Shand method controls the absolute level of the water surface, and the implementation of this method provides an offset-gate opening relationship that is based on stability criteria. The stability is achieved by the incorporation of a low pass filter that attenuates the frequencies for which the system would be unstable.

The coordinated operation of the two control systems is illustrated with the help of Figure II-1. Therein the optimum discharge-offset relationship is given by the dashed line. (offset is the zero flow water surface elevation minus the target elevation)

Starting from a steady flow situation at "A" a sudden turnout discharge ΔQT results in a local decrease in the water surface elevation, thus increasing the offset as well as an increased flow in the canal. This is the condition at "B". The Harder-Smith controller detects this and operates the gate at the upper end to admit an equivalent amount of flow. This produces a surge, which will arrive some time later, and changes the offset to a lesser value (increases the water depth) such as B'. The discharge remains the same. The controller anticipates this surge, and prevents any reaction to it.

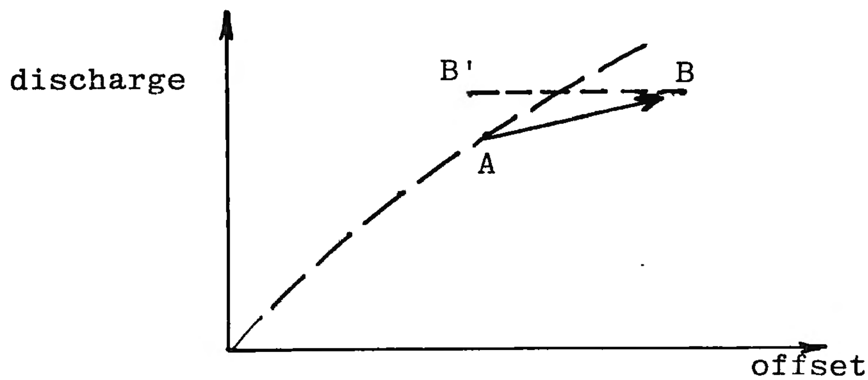

Figure II-1 Operation of the control system as described on the discharge-offset plane.

The surge travels back and forth until it dissipates.

Meanwhile, the average water surface elevation at the sensor is detected by the Harder-Shand controller, which averages it with its low-pass filter. If the average level departs from the optimum curve, it operates the gate to gradually bring the water surface elevation back to optimum.

The two controllers, each being a computer program, operate in conjunction; as far as the external world is concerned they are one integrated control system.

APPENDIX III

Appendix III consists of computer decks, computer print-outs, and other material that does not lend itself to binding in this report, and which are supplied separately to the U.S. Water and Power Resources Service Agency, Denver National Headquarters.