Cablegation: II. Simulation and Design of the Moving-Plug Gated Pipe Irrigation System

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ABSTRACT

THE hydraulics of a moving-plug gated pipe irrigation system are analyzed. A relationship was developed for predicting orifice discharge coefficients for a range of typical pipe flow velocity and head conditions. A simulation model was developed to predict the time distribution of orifice flows, the distribution of infiltrated water across a field, and runoff rates. The model can be used to design cablegation systems for fields having variable pipe slopes and variable furrow lengths. Orifice sizes are varied along the pipe line and the plug travel speed is varied in order to obtain optimum net water application for all furrows and to keep the furrow stream sizes within acceptable limits.

INTRODUCTION

Automated surface irrigation systems using enclosed pipe lines have been successfully demonstrated in recent years (Humpherys et al., 1975). The most common type of automated pipe system uses a buried pipe line for conveyance and surface gated pipes for distribution. An automatic valve and associated timers and controls are provided for each irrigation set. Commercial valves and timers are becoming available as the demand for this equipment increases. A second type of pipe system uses a single gated pipe for conveyance and distribution but requires an automatic valve on each furrow outlet. The initial cost of these systems is comparable to center pivot sprinkler systems and can be higher if extensive leveling is required.

A previous paper (Kemper et al., 1981) described a relatively low cost gated pipe system which requires little labor for operation. The system is called "cablegation" and consists of a surface pipe, laid on a slope at the head end of a field, which serves as both the conveyance and distribution pipe. The gates or holes are located near the top of the pipe, are left open, and may or may not be adjustable. Water is introduced at a flow rate less than the free surface flow capacity of the pipe. A plug obstructs the flow at some point downstream causing a head buildup, and water flows out the holes upstream from the plug.

The plug is restrained by a cable and is allowed to move down the pipe at a slow rate, thus moving the entire flow across the field. The cable follows the plug inside the pipe and is supplied from a reel located at the pipe inlet. The reel rotates at a predetermined rate and was governed by a gear motor in the intital field set up.

The objective of this paper is to analyze the hydraulics of the cablegation system, develop a simulation model including the distribution of infiltrated water over a field, and develop ways to optimize the design for specific field situations.

Hydraulic Analysis

Fig. 1 shows a schematic of the pipe with orifices placed near the top and the relationship of the energy grade line and hydraulic grade line to the pipe line and orifices. The piezometric head is measured from the center of the orifices. The analysis assumes that the orifices are located in the top of the pipe, although in the field system they were located 30 deg from center. Friction losses are computed based on full pipe flow.

The energy equation is used to determine the difference in piezometric head, h_{i+1}-h_i, between two adjacent orifices. Thus,

$$h_{i+1}-h_i = SW - h_f - h_o + (V_i^2 - V_{i+1}^2)/2g, \quad [1]$$

where

V_i = velocity in the pipe upstream from the ith orifice, m/s
g = gravitational constant, 9.81 m/s²
S = slope of the pipe line between the two orifices
W = orifice spacing, mm
h_f = loss of head due to friction, mm
h_o = loss of head due to branching flow at the ith orifice, mm

The friction loss, h_f in mm, as given by the Hazen-Williams equation is,

$$h_f = 6.08 \times 10^6 W (Q/C)^{1.85}/D^{4.865} \quad [2]$$

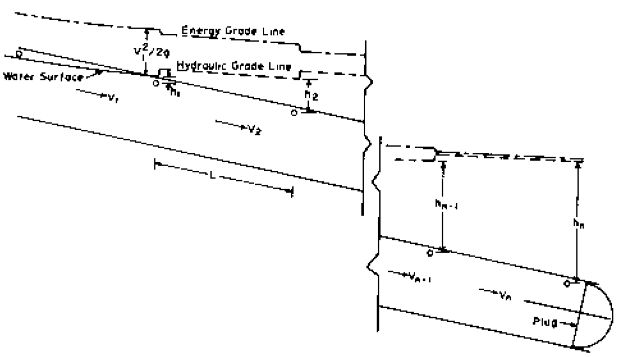

FIG. 1 Hydraulics of the moving plug system.

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where

Q = total flow in L/min

D = pipe inside diameter, mm

C = Hazen-Williams roughness coefficient.

Alternatively, friction loss can be calculated by the Darcy-Weisbach equation,

$$h_f = 1000\, fWV^2/2gD \qquad [3]$$

where the friction factor, f, is computed by the explicit relationship developed by Wood (1966) as follows,

$$f = u + x\, R_e^y \qquad [4]$$

where $R_e = VD/\upsilon$ is the dimensionless Reynolds number and υ is kinematic viscosity. The parameters, u, x and y, are functions of the relative roughness $E = e/D$, where e is absolute pipe roughness, mm.

$$u = 0.094\, E^{0.225} + 0.43\, E$$

$$x = 88\, E^{0.44}$$

$$y = 1.62\, E^{0.134} \qquad [5]$$

Most of the flow in gated pipes occurs at Reynolds numbers between 10^4 and 10^6. As the water approaches the plug in the gated pipe, velocity and the Reynolds number decrease and flow can enter the laminar region (R_e<2000) where neither equations [2] or [3] strictly applies. However, since the friction loss is small in this region, the error in computed head is negligible. Most manufacturers give values of the C factor to be used for their pipe in equation [2] and an equivalent value for e can be calculated. For plastic pipe, e ranges from 0.002 to 0.03 mm and for aluminum, 0.1 to 0.3 mm.

Many papers have been written analyzing the manifold flow problems, usually to determine how to obtain uniform flow from all orifices and to study the losses occurring at the branches. Keller (1949) assumed that the conversion of kinetic energy to pressure at each outlet was complete and thus the deceleration loss h_o is zero.

Van der hegge Zijnen (1951) suggested that the conversion of kinetic energy to pressure is about 90 percent, and that the deceleration loss could be calculated as a sudden expansion, in which case

$$h_o = (V_i - V_{i+1})^2/2g \qquad [6]$$

becomes small when outlet discharge << total pipe flow. The paper by McNown (1954) and subsequent discussions showed that when the outlet discharge is small relative to the total pipe flow, the apparent deceleration loss becomes negative. This is due to the fact that the outlet flow comes from a region in which the velocity is considerably below average, and the actual kinetic energy is slightly greater than that computed from the mean velocity. These results indicate that h_o is sufficiently small that it can be neglected.

Orifice Discharge

The discharge, q_i, from an orifice is given by the equation,

$$q_i = 0.0066\, C_d\, d_i^2\, h_i^{1/2} \qquad [7]$$

where

q_i = flow from the ith orifice, L/min

d_i = diameter of the orifice in mm

h_i piezometric head, mm

C_d = discharge coefficient.

The discharge coefficient is usually assumed constant. A value of C_d = 0.65 was used in the initial paper describing this system (Kemper et al., 1981). Previous papers such as those by Van't Woudt (1964) and Chu and Moe (1971) have used a constant discharge coefficient, but have recognized that C_d is not constant but is dependent upon the velocity in the pipe and somewhat dependent upon pressure head. In the cablegation system the flow condition near the plug is low velocity combined with high head. Moving upstream, the velocity gradually increases as the piezometric head approaches zero.

To more accurately predict the time distribution of furrow stream sizes, a laboratory test was conducted to determine the effect of flow velocity and head on the orifice discharge coefficient. A 200 mm inside diameter aluminum pipe was used in which orifices of 13, 19 and 28 mm diameter had been drilled. A 148 mm pipe with 17 and 28 mm orifices was also used. The total flow was controlled to produce average velocities of 0.2, 0.5, 1.0, 1.5 and 2.0 meters per second. At each of these velocity levels the piezometric head was varied from near zero to approximately 400 mm, and simultaneous head and orifice flow measurements were taken. The discharge coefficient was calculated for each discharge head measurement. For any given orifice, the coefficient C_d tended to approach a constant maximum value C_{do} as head increased or velocity decreased. The value of C_{do} varied from 0.62 to 0.65. A dimensionless parameters $h_r = h/(V^2/2g)$ was defined as the ratio of piezometric head to velocity head,

$$h_r = h/(V^2/2g) \qquad [8]$$

The ratio of C_d/C_{do} was plotted as a function of h_r as shown in Fig. 2. The data were grouped according to the ratio of orifice size to pipe size, d/D. There is no apparent effect of orifice size up to d/D = 0.2. The data in Fig. 2 were taken with the orifices set at an angle of 20 deg from vertical. Another set of tests was run with an orifice angle of 30 deg, resulting in a similar relationship

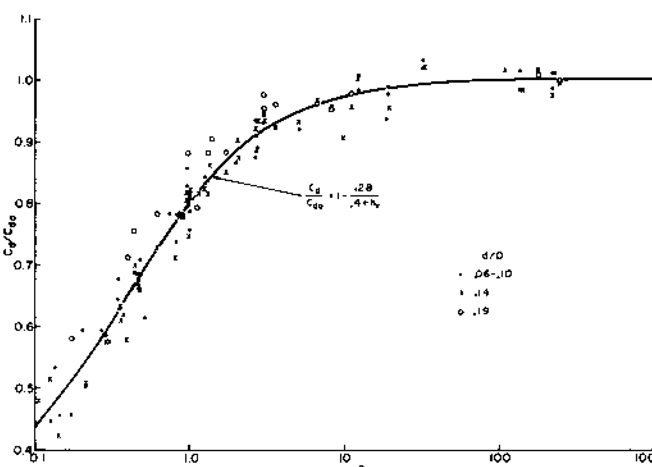

FIG. 2 Variation in orifice discharge coefficient with ratio of piezometric and velocity head in pipe.

but the data were more scattered. The significant result is that the discharge coefficient begins to decrease when the piezometric head is less than about 10 times the velocity head. For use in the model, the following relationship was chosen.

$$C_d/C_{do} = 1 - 0.28/(0.40 + h_r) \quad [9]$$

Equation [9] fits the measured data for $h_r > 0.05$. For values of h_r between 0 and 0.05, equation [9] may not be accurate; however, this region represents a very small portion of the distribution and should not affect the results.

Operation of the Pipe Flow Model

The parameters which are specified prior to computation are the pipe inside diameter and roughness, the orifice size(s) and spacing, the pipe slope(s) and the total inflow rate(s). The inflow rate may vary with time, but is limited by the flow capacity of the pipe when the friction slope is equal to the minimum pipe slope. As shown in Fig. 1, the piezometric head is measured from the center of the orifices and becomes zero at some point upstream from the plug. Since the point of zero head is unknown, a trial and error procedure is used to determine the hydraulic grade line. Starting at the downstream end, a value is assumed for the piezometric head h_n at the last flowing orifice. The orifice discharge and pipe flow are computed, and working upstream, the changes in head are computed by equation [1]. When the piezometric head becomes zero, the total accumulated flow is compared with the known inflow rate. The head h_n is then readjusted and the process repeated until the sum of the orifice flows is sufficiently close to the total inflow. This procedure applies after the plug has moved sufficiently far down the pipe such that the first orifice has stopped flowing. For the initial or start-up period the procedure must be modified. Three modes of operation are described for start-up as shown in Fig. 3.

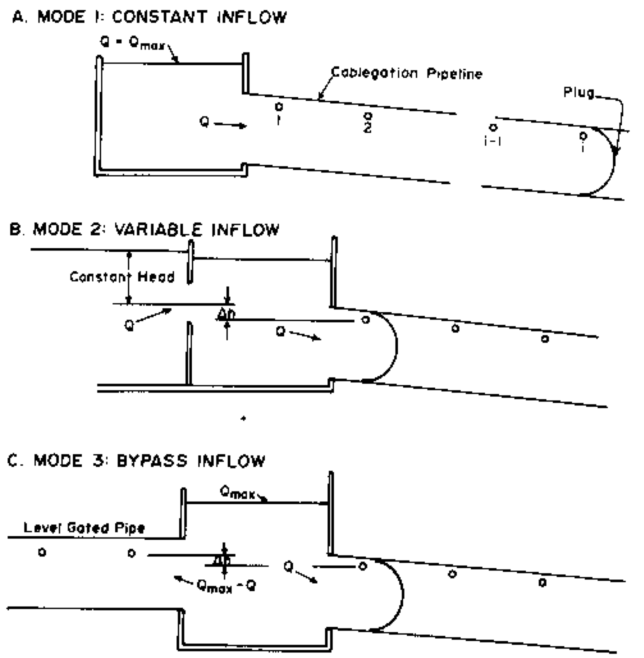

FIG. 3 Modes of operation for startup.

Mode 1. The plug is held stationary just beyond the ith orifice for a specified time, t_i, and then allowed to move at a constant rate (Fig. 3a). The inflow rate, Q, is constant from time zero. The intital orifice flows are constant until the plug begins to move, and then decrease to zero. The pipe slope for the initial set could be minimized to give a more uniform head distribution and allow more uniform hole sizes.

Mode 2. The plug starts moving at the first orifice from time zero. The total inflow rate is initially equal to the first orifice flow and gradually increases as the plug moves, opening up additional orifices, until a maximum specified flow is reached. The head at the first orifice gradually decreases to zero. The inflow rate is controlled by an inflow orifice of specified area with constant upstream head as shown in Fig. 3b.

Mode 3. The plug moves from time zero as in Mode 2. Initially most of the flow is diverted to a level gated pipe, or an equivalent system, which comprises an initial set. As the plug moves, the flow into the cablegation pipe increases until all flow is diverted to the cablegation side. The total area and elevation, Δh, of the orifice(s) in the level pipe are specified. Fig. 4 shows an example of the time distribution of inflow with Modes 2 and 3.

For all start-up modes, the calculation procedure is as follows. The piezometric head for the first orifice is assumed, the inflow rate is determined, and calculation proceeds downstream to the plug. The accumulated flow is compared with the inflow rate, upstream head is readjusted, the inflow rate is recalculated and the procedure repeated until the total flows balance. As the plug moves down the pipe, the head at the first orifice decreases and finally becomes zero. At this time, the calculation procedure is switched to the previously described method.

When the plug reaches the end of the pipe, there are three ways of completing the irrigation:

1 Inflow continues at the same rate until a desired gross or net application has been applied at the last furrow. It is difficult to obtain uniform net application with this method because the intake opportunity time for the last furrow is less than for furrows further upstream. Orifice sizes are usually increased near the lower end to produce rapid advance, and minimize the final set time required.

2 Inflow starts to decrease when the plug reaches the end. The inflow rate is decreased linearly to zero over a time period equal to the width of the flow distribution, divided by the plug speed when the plug reached the end. This method simulates the transfer of flow to a second cablegation system in which the second plug starts to move when the first plug reaches the end. This method allows more uniform orifice and stream sizes and results

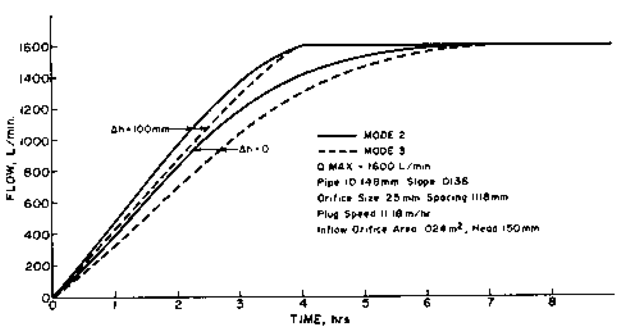

FIG. 4 Time distribution of inflow for startup Modes 2 and 3.

in a uniform net water application.

3 When the plug reaches the end, the outflow rate past the plug is allowed to increase from zero to the maximum rate, simulating the transfer of flow to a second plug system downstream. The resulting distribution is very similar to that obtained with the second method and a uniform net application could be easily obtained. This transfer can be accomplished by letting the plug move into a standpipe which is connected to a downstream pipe system, and allowing the flow to back up behind the second plug which then controls the flow. A special outlet is being developed which will allow a gradual release of flow from the end of the pipe into a wasteway or downstream pipeline.

The required speed of the plug is calculated to apply a specified gross amount of water on each furrow. If furrow lengths vary, the speed is adjusted by approximately an inverse proportion to the furrow lengths. If pipe orifice spacing changes, the cable speed is increased in direct proportion to the orifice spacing. The cable speed can be easily changed in practice by using adjacent reels of different diameters. The cable is wound up on the reels in reverse sequence. As the cable unwinds, the speed increases when the cable transfers to a larger diameter reel.

Multiple Plugs

As the pipe slope increases, the peizometric head at the plug increases. The head also increases as the orifice sizes are decreased to spread a given total flow over more furrows. Socks can be used on these outlets to dissipate this energy and control soil erosion. However, the large number and consequent cost of the socks needed often make other alternatives economically desirable. One alternative is to limit the piezometric head in the pipe. The use of multiple energy dissipating devices attached to the cable at selected distances upstream from the plug can accomplish this objective. The devices are attached as the plug is reeled out for the initial set. A plug with one or more holes or a baffle plate could be used as an energy dissipator. In general, the head loss relationship will be of the form

$$h_p = KV^2/2g \quad \text{[10]}$$

where

h_p = head loss across the device
V = pipe flow velocity immediately upstream from the device
K = loss coefficient

For simulation purposes, the distance from the plug to each head dissipator and a loss coefficient for each device are input to the program.

Optimizing Orifice Sizes

Orifice sizes are specified at key points and the remainder are computed by interpolation. An initial estimate of required orifice sizes can be obtained by first specifying the desired initial stream size(s). As the plug moves down the pipe, the piezometric head at the plug is known and the size of each new hole can be calculated from the known head and desired stream size. After this initial run, the orifice sizes are readjusted if necessary and the simulation is rerun until the desired distribution is obtained.

Orifice size can be varied to help compensate for differences in head-time distributions which occur at some orifices and thereby obtain more uniform distribution of applied water across a field with the moving plug system. Orifice sizes usually should be increased at both ends of the field to compensate for shorter time duration of flow. Changes in pipe slope or furrow length also require variations in orifice size to optimize water application uniformity. The following procedure is used to determine the orifice sizes required to produce a uniform gross water application. Orifice sizes are initially specified as constant or follow an assumed distribution. A complete irrigation is simulated and gross applied volume, V_i for each furrow is calculated. An adjusted orifice size, d_i', is calculated by the equation,

$$d_i' = d_i\,(\text{V}/\text{V}_i)^{1/2} \quad \text{[11]}$$

where

d_i = initial orifice size
V = desired volume.

The simulation is repeated and orifice sizes readjusted until the calculated distribution of applied volumes agrees with the desired distribution.

Some Results From the Pipe Flow Model

Fig. 5 shows an example of the distribution of piezometric head with uniform orifice size. Results using the Darcy and Hazen-Williams equations are compared. The effects of pipe roughness and water temperature are shown. The distribution of Reynolds number is also shown.

It appears that the Hazen-Williams equation is adequate to describe friction losses, and this equation will be used in the remainder of this paper.

Fig. 6 shows an example of the distribution of orifice flow with total flow at various fractions of pipe flow capacity. The solid lines were calculated with the use of equation [9] and the dotted line was calculated for $Q/Q_{cap} = 0.95$ using a constant orifice discharge coefficient of 0.65 throughout. The effect of the reduction in discharge coefficient on the shape of the flow distribution curve is evident. It is also apparent that in order to terminate the flow rapidly at the upstream end, the total flow should be less than about 90 percent of the flow capacity.

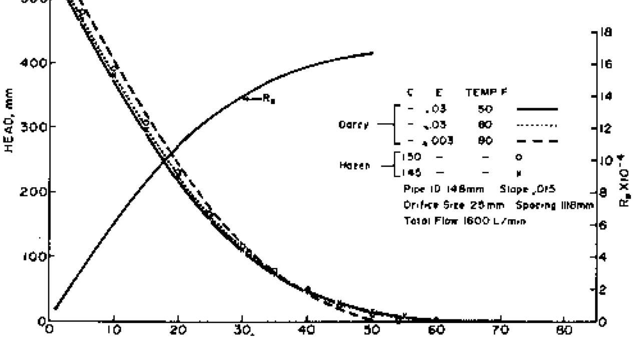

FIG. 5 Effect of friction loss parameters on head distribution.

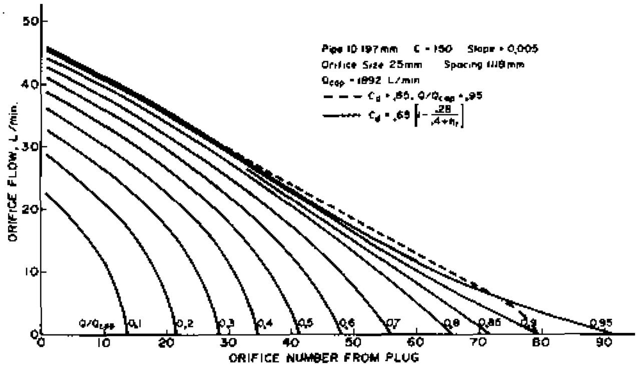

FIG. 6 Distribution of orifice flow at various fractions of pipe flow capacity.

Furrow Infiltration Model

The discussion thus far has been concerned with modelling the pipe distribution system and predicting the time distribution of inflow to each furrow. Since the inflow distribution can be different for each furrow, it is desirable to compute infiltration for each individual furrow and then determine the distribution of infiltrated water over the entire field. The infiltration model must be as simple as possible in order to limit computation time and to keep the number of parameters to a minimum. A time-based function will be used in this analysis. The volume of intake per unit length of furrow can usually be described by,

$$Z = aT^b + cT \quad [12]$$

where

Z = volume of intake, L/m

T = time in hours since the beginning of wetting

a, b, and c are constants

Equation [12] can take several forms depending on the values chosen for the constants. If $c = 0$, the equation becomes a simple power function, and if $b = 1/2$, the equation becomes the Philip (1954) equation. If T is dropped from the second term on the right side, the equation is identical to the USDA-SCS (1974) equation. Equation [12] can be made to describe a wide range of intake conditions and can be used to approximate other mathematical functions if data for other functions are available. The following procedure can be used with any time-based infiltration equation.

The distribution of infiltrated water is computed by first dividing a furrow into N equal segments of length, F. In this model, surface storage is assumed to be negligible, (the fields used in this study were relatively steep and surface storage was small) and all applied water is added directly to infiltration or total runoff. The volume of inflow for the first furrow increment during the jth time increment is,

$$V_1 = q_j\,\Delta T \quad [13]$$

where

q_j = average inflow rate (L/min) for the jth time increment

ΔT = time increment (hours).

The inflow volume V for the ith furrow increment is equal to the inflow volume for the previous furrow increment minus the volume infiltrated in the previous furrow increment. For each successive furrow increment, the potential or maximum infiltrated volume is given by,

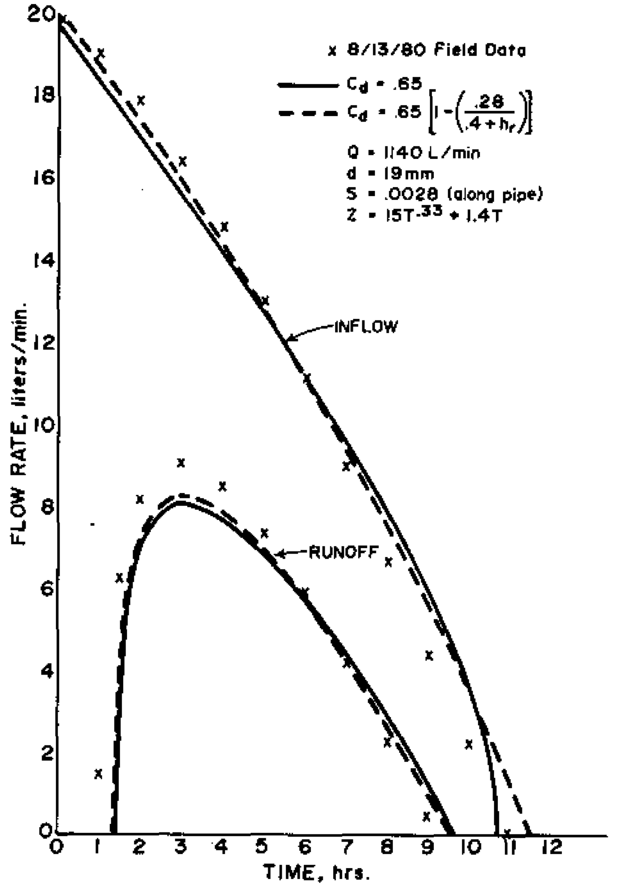

FIG. 7 Computed inflow and runoff rates compared with measured rates (average of 8 furrows).

$$Z_{i,j} = a(T_o + \Delta T)^b + c(T_o + \Delta T) \quad [14]$$

where T_o is the effective opportunity time for the ith furrow increment, computed by solving for T in the infiltration equation with $Z = Z_{i,j-1}$. If $c = 0$, for example, then

$$T_o = \left(\frac{Z_{i,j-1}}{a}\right)^{\frac{1}{b}} \quad [15]$$

In general, equation [12] requires a trial and error procedure to solve for T, given Z. However, for the cases $c = 0$, $b = 1/2$, $b = 1/3$, or $Z = aT^b + c$, T_o can be solved explicitly as a function of Z

If the inflow volume for a furrow increment exceeds the potential infiltration, then the actual infiltration equals potential and the excess is the inflow to the next increment. However, if the inflow is less than potential infiltration, then the actual infiltration is equal to the available inflow volume and the infiltration for all succeeding increments is zero. The wetted portion of the furrow is thus advanced by increments until all N increments are wetted and then all excess inflow is added to runoff volume. As the inflow rate decreases, the runoff rate also decreases and finally ceases. The wetted portion of the furrow then begins to recede until inflow ceases, thus completing the irrigation for that furrow.

Field Evaluation of the Complete Model

A field test setup of the cablegation system was described in the first paper (Kemper et al., 1981). The measured and calculated orifice flow distribution was compared. Detailed measurements of orifice and furrow flows were made on August 13, 1980 during an irrigation (Goel et al., in process). The pipe size was 197 mm (8 in.), orifice spacing was 762 mm, orifice size was 19 mm, and pipe slope was 0.0028. Furrow length was 108 m and total inflow was 1140 L/min. Eight furrows were selected (furrows 60, 70 . . . . 130) and inflow and runoff rates were measured on these furrows for the complete irrigation. Advance time varied from 0.7 to 1.6 h. Fig. 7 shows average inflow and runoff hydrograph data for the eight test furrows. The average advance time for these furrows was about one hour. Furrow intake rates were calculated by subtracting runoff from inflow rates. An average intake curve was fitted to these data and values for the intake parameters (a = 15, b = 0.33, c = 1.4 for units given in equation [12]) were determined.

Two simulations were run using the above intake parameters. The solid line in Fig. 7 shows hydrographs computed using a constant orifice discharge coefficient (C_d = 0.65). The dashed lines were computed using equation [9] for C_d. The use of equation [9] appears to improve the agreement between field measured and computed flows. This relationship will be used for the remainder of the simulations in this paper.

Design Examples

Use of the model for simulating field situations will be illustrated by three examples. The values of the parameters used in these examples are given in Table 1. System 1 is a rectangular field with constant pipe slope. Inflow is Mode 1 with 50 furrows flowing initially. Orifice sizes were first adjusted to obtain nearly uniform gross

TABLE 1. INPUT PARAMETERS FOR DESIGN EXAMPLES.

A. *System 1. Rectangular field, constant pipe slope*
Pipe I. D. = 197 mm C = 150 Slope = 0.0028
Furrow length increment = 20 m, Total length = 200 m

Total number of furrows	200
Furrow and orifice spacing	762 mm
Intake	$Z = 15T^{0.33} + 1.4T$
Inflow Mode 1, Total flow	1140 L/min
Cable speed	3.42 m/hr

B. *System 2. Variable pipe slope*

Pipe I. D. = 201 mm	C = 150
Furrow length increment	30 m
Furrow and orifice spacing	1118 mm
Intake	$Z = 15T^{0.33} + 1.4T$
Total flow	1703 L/min
Inflow Mode 3	Δh = 450 mm

Section	Furrows	Length increments	Pipe slope	Cable speed, m/hr
1	160	11	0.004	3.16
2	90	11	0.020	3.16
3	158	5	0.014	6.95
4	92	6	0.014	6.95

C. *System 3. Variable furrow length and orifice spacing*
Pipe I. D. = 152 mm C = 150 Pipe slope = 0.0045

Furrow length increment	30 m
Furrow spacing	762 mm
Intake	$Z = 10T^{0.66} + 5.3$
Inflow Mode 1, Total Flow	852 L/min

Section	Furrows	Length increments	Orifice spacing, mm	Cable Speed Run 1, m/hr	Cable Speed Run 2, m/hr
1	60	10	762	1.70	1.70
2	104	9	879	2.18	2.56
3	42	7	2286	5.80	5.79
4	44	5	859	5.80	3.75

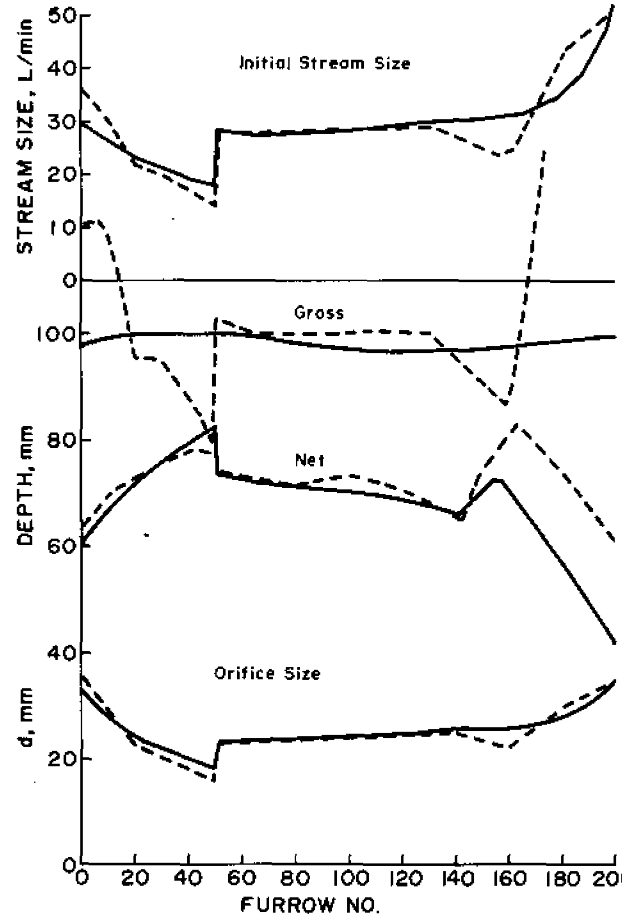

FIG. 8 Orifice sizes, furrow flows, gross and net water application, System 1.

water application using the previously described method, with the results shown in Fig. 8. The solid lines show the distribution of orifice sizes, initial furrow stream size, and net water application (average infiltrated depth for each furrow) when gross application was maintained between 97 and 100 mm. A second run was made with orifice sizes adjusted to obtain more uniform net application, and the results are shown as dashed lines in Fig. 8. It was necessary to increase the gross application drastically at the upper and lower ends to increase the net application. Fig. 9 shows the distribution of infiltrated depth for the second run. The "Christiansen" uniformity coefficient for this run was 0.91, calculated by including infiltrated depth for all furrow segments in the field.

In this example, the inflow rate remained constant after the plug reached the end, and inflow continued until gross application reached 100 mm for the first run and until net application reached 60 mm for the second run.

System 2, shown in Fig. 10, involves variable furrow lengths and variable pipe slopes. Inflow Mode 3 is used with the intital set of furrows being irrigated prior to the start of the cablegation. The field is divided into four sections (excluding the initial set) in which the pipe slope and furrow lengths are constant. Since the furrow lengths are integral multiples of the furrow length increment, an irregularly shaped field is approximated by a series of rectangles.

The simulation was run with the cable speed calculated to apply a gross application of 100 mm of water on the first two field sections. The cable speed was

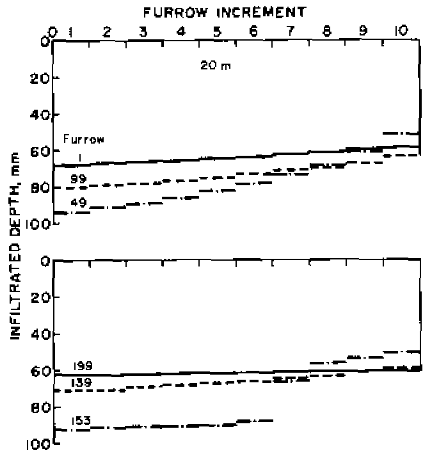

FIG. 9 Distribution of infiltrated water in selected furrows, System 1.

increased for the last two sections. Fig. 11 shows the distribution of orifice sizes obtained for System 2 after several runs. The gross and net water application and initial furrow stream sizes are also shown. The dashed lines in Fig. 11 show the results obtained when the inflow rate was decreased linearly to zero after the plug reached the end. Orifice sizes were kept nearly constant for the last field section and net intake was maintained at a constant level.

The effect of an increase in pipe slope can be seen in Fig. 11 where the slope increased by a factor of 5 at furrow 160. It was necessary to increase the orifice sizes in the vicinity of the slope change to obtain uniform net application along the pipe. The effect of the slope change could be reduced if a smaller pipe could be used on the higher slopes. A change of pipe size could be accomplished by transferring the flow to a second plug system as previously described.

The third example involves the situation shown in Fig. 12 where the head end of the field is odd shaped. The pipe is laid on a constant slope and the orifice spacing changes so that the outlets will coincide with the furrows. Table 1C lists the parameters used in simulating System 3. The inflow is handled by Mode 1 with 24 furrows in the initial set. The plug speed was calculated to apply approximately 100 mm gross application, and increases in successive field sections in direct proportion to the orifice spacing and in inverse proportion to the furrow lengths. The speed for sections 3 and 4 was taken as the average for the two sections since there is considerable overlap of the flow between sections. The results of simulating this system are shown in Fig. 13. For the initial set, the orifice size was decreased from orifice 1 to 24 as shown in order to compensate for the decreased intake opportunity time at the upper end. After the intital set, the orifice sizes are constant to about midway through the second section and then gradually decrease due to the need for smaller stream sizes on the shorter furrows. This distribution at the lower end is similar to that of System 2 when the inflow was kept constant until the last furrow had the required net intake. It was necessary to reduce the runoff to almost zero in the upper part of the final set and allow a large amount of runoff at the lower end in order to obtain fairly uniform net application. The distribution problem of the final set would be reduced if the furrows were of constant length, or better still, incresing in length rather than decreasing toward the lower end because in-

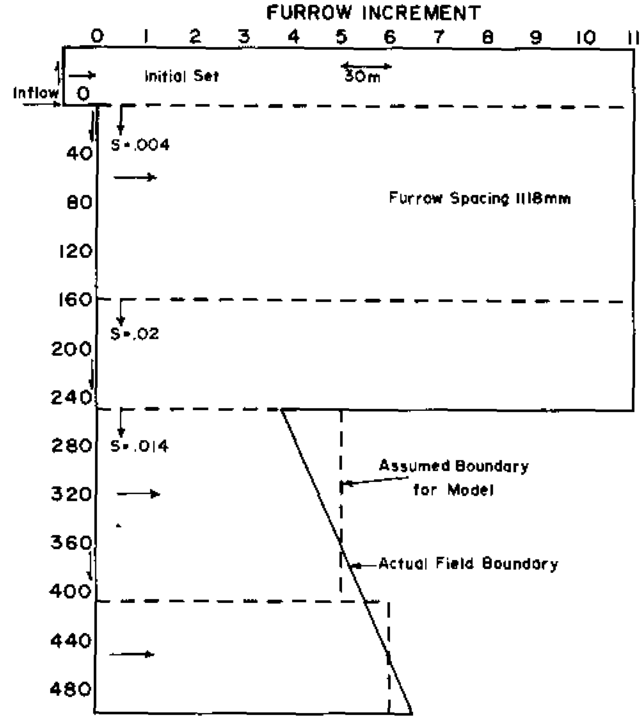

FIG. 10 Field layout for System 2.

creasing stream sizes would be desirable.

The dashed lines in Fig. 13 (run 2) show the results of decreasing the inflow when the plug reached the end, and gradually decreasing the orifice sizes toward the end of the pipe. It was necessary to change the cable speeds for this run because the orifice spacing changed drastically. The average runoff was low for this system indicating that larger stream sizes probably should have been used.

The sharp changes in gross and net applied water in Figs. 11 and 13 are largely due to the abrupt changes in furrow length assumed in the model which do not occur in the field. The minor fluctuations can thus be ignored and the design adjusted according to major trends in the water distribution.

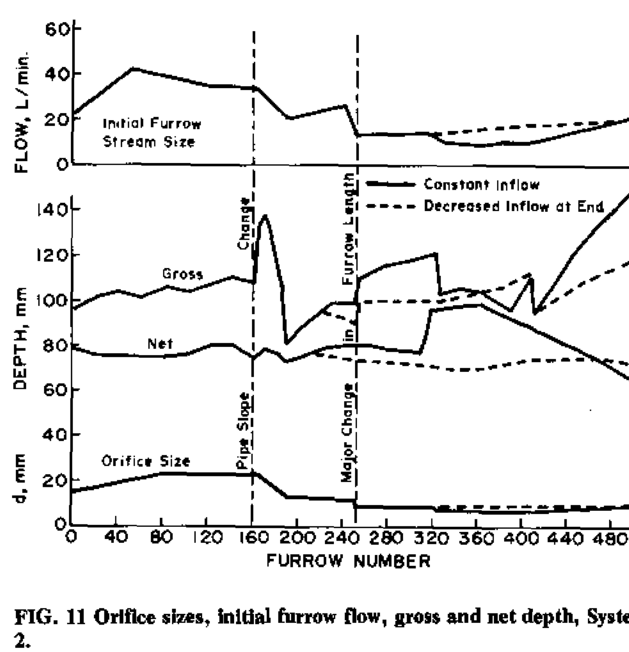

FIG. 11 Orifice sizes, initial furrow flow, gross and net depth, System 2.

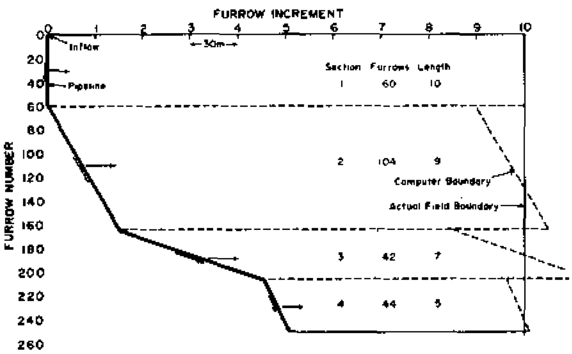

FIG. 12 Field layout, System 3.

DISCUSSION AND CONCLUSIONS

Design limitations for the cablegation system have not been worked out to the present time. The pipe slope must be greater than zero, but beyond that, any limitations on pipe slope depend upon the total flow and maximum stream sizes, orifice size limits or maximum pipe head and other factors. Systems have been successfully designed and operated with pipe slopes between 0.0026 and 0.022. Steeper pipe slopes may be feasible as energy dissipating outlets become available. On fields requiring very large stream sizes, the minimum pipe slope may be larger than indicated above. The process of selecting furrow stream sizes for given furrow slope, length and infiltration rates, is similar to other furrow systems except that higher initial stream sizes may be used due to the cutback nature of the flow. The simulations need to be run over a wide range of conditions to develop design criteria and determine attainable efficiencies.

The cablegation system is particularly well suited to cases where the furrow set width is narrow relative to the total width of a field. The problems of distribution uniformity are mostly related to the initial and final sets (when orifice flows are constant and the plug is stationary) and the portion of the field affected by these conditions. The end conditions can be eliminated by gradually increasing the flow initially and gradually decreasing the total flow at the conclusion of the irrigation when the plug is at the lower end. Thus it is desirable to operate these systems in sequence so that the flow is automatically transferred from one field to another. This also maximizes the benefit from the cutback furrow flow inherent in this system. Operation in Mode 2 would be facilitated by using a constant-head float valve such as the "Harris valve" to supply water from a reservoir or canal.

This paper has not addressed the problem of intake rates varying during an irrigation season. For high intake rates which usually occur during the first irrigation, larger stream sizes will probably be required. Small changes in stream sizes can be made by increasing or decreasing total flow, and multiple plugs (on the cable inside the pipe) can be used to decrease stream sizes by decreasing the head on the orifices. Large changes in stream sizes will require changing orifice sizes. Low cost polyethylene gates are available which can facilitate such changes. The simulations can be run with maximum expected range of intake constants to determine the range of orifice sizes to obtain an acceptable distribution.

The model has made use of a time based infiltration equation which does not account for the effect of variable

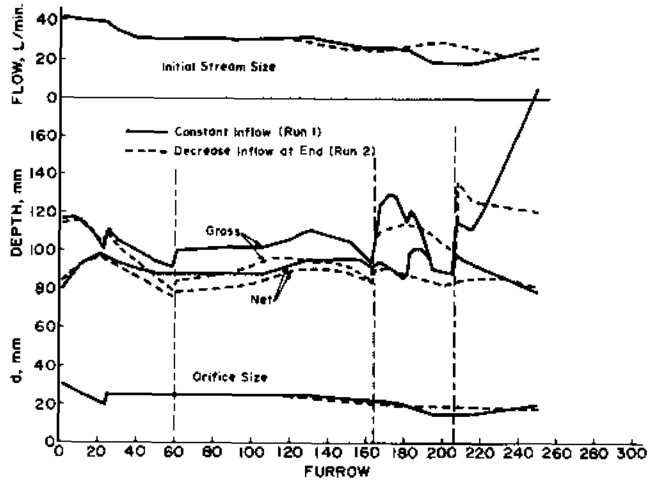

FIG. 13 Orifice sizes and distribution of gross and net water application and stream sizes, System 3.

furrow flow rate or wetted perimeter on infiltration rates. The use of a more detailed infiltration model would be desirable and may shown that larger stream sizes partially compensate for reduced intake opportunity times. If this compensation is appreciable, designing cablegation systems for more uniform gross application may result in more uniform net application than is predicted by our present model. The infiltration-advance model may need to be modified to include surface storage on flatter furrow slopes. However, the potential error in predicting infiltration rates is likely to far exceed the error caused by neglecting surface storage.

The simulations have thus far shown that for systems with changes in pipe slope and furrow length, the distribution is quite sensitive to small changes in orifice size, and trial and error adjustments in the field may prove to be frustrating. For fields with complications of the type shown in systems 2 and 3, farmers may need guidelines developed on a computer model to help them make the orifice size changes most efficiently to accommodate changed infiltration rates.

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