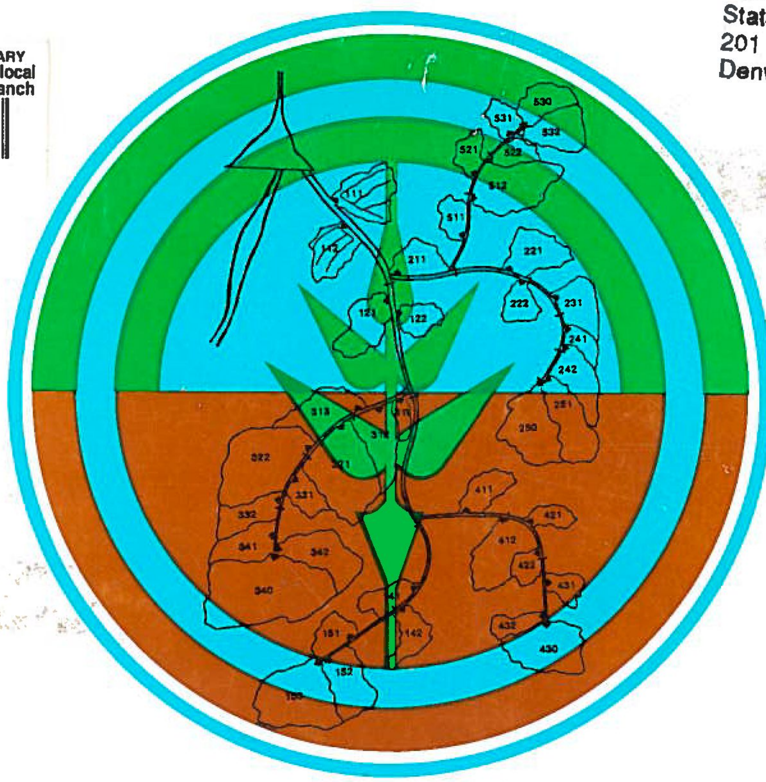

DEVELOPMENT OF A BRANCHING CANAL NETWORK HYDRAULIC MODEL

WATER MANAGEMENT SYNTHESIS II PROJECT
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PREFACE

This study was conduted as part of the Water Management Synthesis II Project, a program funded and assisted by the United States Agency for International Development. Utah State Universty, Colorado State University, and Cornell University serve as co-lead universities for the project.

The key objective is to provide services in irrigated regions of the world for improving water management practices in the design and operation of existing and future irrigation projects and give guidance for USAID for selecting and implementing development options and investtiment strategies.

For more information about the Project and any of its services, contact the Water Management Synthesis II Project.

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ABSTRACT

The performance of canal networks can be improved through physical upgrading of the system and through changes in management and operation. This study provides irrigation professional and novices with a tool to address issues related to the interaction between design and operation of the conveyance and distribution system.

This report illustrates the development of a mathematical model based on solving the integrated form of Saint Venant's equations to simulates canal filling, operating, and draining phases, bulk lateral outflow or inflow into the section being modeled, and control structure scheduling (gate-stroking) of a branching canal network. The model can be used in evaluating unlimited "What if ..." questions on the planning, design, management, and operational issues.

The model developed represents a unique set of integrated modules that can be used to better assess the reality in dealing with flow conditions prevailing in canals with the aim to identify constraints and opportunities to increase manageability of the system. The model highlights are:

1. The model simulates closely the behavior of existing canal networks making it acceptable by operating staff;

2. The model input, output, and operation meets the needs of different categories of users;

3. The computer program optimizes the computation and memory allocation giving the software the highest possible level of simulation performance on microcomputers; and

4. The computer program has state-of-the-art algorithms and modules that prevent hydraulic simulation errors, numerical instability, and divergence of the solution.

CHAPTER I

INTRODUCTION

Background

Irrigated agriculture has a vast appetite for water. In 1975, irrigation accounted for over 80 percent of all water withdrawal in the world (Framji, 1984) and more that 90 percent in the western United States (Bredehoeft, 1984). An evaluation of 61 federal irrigation projects covering approximately one million acres, by the U.S. Department of the Interior, noted that on the average only 44 percent of the water diverted reached the farm. They envisaged that the average could be increased to 61 percent and about two thirds of this improvements would be accomplished through better operation of the conveyance and distribution systems (U.S. Department of the Interior, 1978).

These poor performances can be attributed to substandard system management resulting from: (1) over-simplistic approaches to planning and design which lead to projects that have constrained capacities, poor maintenance, and inequitable distribution of water and income; and (2) the rigid approaches to system operation and management that fail to satisfy crop demands with respect to time and quantity of water delivered. Replogle (1980) observed that

> Rigid schedules often result in low project irrigation efficiencies, create drainage problems, leach soil nutrients, and waste labor on the farm. (p. 320)

To optimize the crop production environment, a flexible and reliable delivery system is required in order to increase operational flexibility to respond to changes in weather, cropping patterns, and socio-economic factors. Strong incentives to improve the performance of conveyance and distribution systems are provided by the: (1) high cost of developing, operating, and maintaining conveyance systems; (2) growing water shortage; and (3) high potential for improvement.

The advent of computers and developments in systems analysis technology is facilitating the development of comprehensive and inter-disciplinary models. Systems analysis techniques are expected to augment the managers' experience in decision making by: (1) providing a broader information base; (2) creating a better understanding of the system and its numerous components; (3) predicting the consequences of a given course of action; and (4) selecting suitable actions required to achieve predetermined objectives (Biswas, 1975). In an irrigation water system, these methodologies can be used to ensure that all of the links in the chain that provide for the capture, storage, transportation, utilization and disposal of excess water are equally strong and that the delivery of an adequate and reliable water supply is achieved.

Statement of the Problem

Kraatz and Mahajan (1975) observed that due to difficulties in planning and designing irrigation water conveyance systems (canal dimensions; number, type, and location of water control structures; and appropriate canal layouts), it is sometimes advantageous to install temporary structures at first and then replace them as more knowledge and experience in operating the system became available. It can, therefore, be argued that the present knowledge base is barely sufficient to improve irrigation water delivery decision making. Bottrall (1981) noted that

> .... it [water allocation and distribution research] has been astonishingly neglected, both by academic researchers and professional practitioners. ...there was recognized to be an immense potential, so far largely untapped, for improving current water distribution practices. (p. 2)

Rangeley (1983) also identified water distribution as one of the irrigation research topics that should receive priority. Burt (1987) noted that because designers rarely consider unsteady flow conditions in their design analysis, the following operational issues should be addressed in order to improve the performance of irrigation projects:

1. With the present or planned locations and type of control structures, what will be the flow rate fluctuations;
2. How should the structures be operated;
3. How does the adjustment of one structure affect the flow conditions; and
4. How often should control structures be adjusted.

Thus, in gravity irrigation projects, a major problem is "How can irrigation canal networks be designed, operated, and managed in a dynamic manner to ensure a reliable, adequate, and timely water delivery to the crops?"

This study focuses on the development of conveyance system (main system) management software that can be used for analysis of planning, design, and operational issues. It was justified by: (1) the current neglect and poor performances of open-channel conveyance systems; (2) the complexity and pervasiveness of open-channel conveyance technique; (3) the high potential for improvement; and (4) the dire need for a comprehensive multi-disciplinary methodology for irrigation water management.

Objectives

The primary purpose of this study was to formulate, develop, and implement a methodology for managing unsteady, non-uniform flow regimes in an irrigation canal network. This was accomplished by the following tasks:

1. Illustration of the structure, functioning and working order of an irrigation water conveyance system;
2. Formulation of a theoretical analysis of unsteady, non-uniform flow in branching canals network with a wide range of flow control structures and diverse water control rules;
3. Development of computer software to solve the mathematical problem developed in (2) above;
4. Generalization of the software to make it readily transferable and user-friendly; and
5. Verification of the computer solution against field data to demonstrate its utility in generating technically feasible and socially acceptable solutions of the operation and management problems in an irrigation water conveyance system.

Scope of the Study

The work reported here deals exclusively with an open channel water conveyance and distribution subsystem and its relationship with the storage, allocation, and command area subsystems. This research explored ways and means of how to operate the subsystem as a dynamic entity to better cope with the variations in demand and supply, control canal transients, and minimize operational losses.

The model is a solution of the Saint Venant equations of open channel flow. Consequently, it is limited to the conditions under which these equations were derived. Piped sections, inverted siphons, and long drop structures or chutes, and other critical or super-critical transitions are treated as boundary conditions with a time delay factor. The model does not consider the effects of canal bends nor the non-prismatic nature of the canal.

The resulting model enables operational and management issues of irrigation conveyance systems to be analyzed from different viewpoints. In each case the less viable options and alternatives are eliminated, thereby allowing the model user to finally focus on selected feasible solutions. The specific uses of the model are to:

1. Provide an analytical tool to answer a wide range of "What if..." questions related to planning, design, and management of irrigation water conveyance systems;
2. Provide immediate and specific demonstration of what can be achieved by improved management techniques of a particular system; and
3. Increase the understanding of certain principles and relationships affecting quality of irrigation water delivery.

The model is also intended to provide incentives for improved water accounting and system monitoring, and to aid system operators in making the right decisions. These benefits should lead to reduced social conflicts and provide incentives for improving on-farm water management, ultimately leading to increased farm incomes.

CHAPTER II

POLICIES AND STRATEGIES IN MAIN SYSTEM MANAGEMENT

Issues

The main objective of the conveyance system is to convey water to the users in a predictable and reliable manner so that they can make productive use of it. An ideal conveyance system is one in which the series of operations necessary to ensure that crop water requirements are met at the right time and quantity are in place and functional. Variation in flow rate at the farm level makes it impossible for farmers to control the flow without additional investments and makes it difficult to determine how much water has been applied--a major cause of over and under-irrigation (Clemmens, 1983). There is widespread consensus that enormous scope exists for improving irrigation systems. (Bottrall, 1981; Rangeley, 1983; Burt, 1983; and Keller, 1986). Chambers (1984) noted that the interventions that dominate our thinking rarely focus on what can be achieved immediately with little or no additional expenditure. He identified the following approaches that the system managers can implement without farther delay:

1. Rotations and reallocating water from head to tail;
2. Utilizing or saving irrigation water at night;
3. Responses to rainfall, including consideration of rainfall probability;
4. Better information to farmers about water supplies;
5. Better information and communications for managers about operation and performance; and
6. Farmer involvement in decisions and management.

Attempts to improve the performance of a conveyance system should address the following crucial questions in the operation of an irrigation water system.

The physical system issues:

1. Does the project have adequate physical infrastructure and is it functional?
2. Does the physical setting favor head- or tail-enders?
3. What physical infrastructure is required in order to improve project performance? and
4. Is the physical infrastructure structurally safe against system malfunction, improper operation, earthquakes, etc?

Management issues:

1. How does the system react to unexpected rainfall that forces irrigators to reject the water in-transit that is allocated to them?
2. How is the water routed during unusually high demand periods to ensure equity in water distribution?
3. What operational adjustments should be made to cater to the changes in cropping patterns that the farmers plan to implement in the following season (the cropping pattern may not yet be known)?
4. What is the most appropriate delivery schedule?
5. How can water supply reliability be improved?
6. How can the system be operated to minimize water level fluctuations and operational and seepage losses?
7. How much water is being delivered to various outlets? and
8. To what extent is poor maintenance responsible for reduced system capacity, increased lag time and flow depth, and high losses and turnout discharges?

and finally, institutional issues:

1. Susceptibility of agency personnel to user influence;
2. Water users' apathy and mistrust of the operation and maintenance agency;
3. Poor communication between the agency and the water users;
4. Perception of agency personnel that the farmers are ordering more water than the actually need; and
5. Failure on the part of water users to order water at the right time and quantity or use it when delivered.

Water Management

The irrigation water demands change throughout the irrigation system spatially and temporally due to changes in the weather, cropping pattern, crop growth stage, and irrigation practices. In order to satisfy this fluctuating demand the canal system experiences changes in flow rates resulting from changes in control structure settings and water levels in the canal. Irrigation frequency depends on the climatic factors, crop consumptive needs, the soil water reservoir, and the flexibility of the physical and non-physical sub-systems. Water management activities at the conveyance and distribution level can be divided into two activities, water allocation and delivery and flow regulation.

Allocation and Delivery Rules

Management activities in sharing irrigation water fall into two categories--the allocation rules and the delivery schedules.

Water allocation rules may be based on anticipated crops to be grown, physical system characteristics, social-cultural requirements, political considerations, required system flexibility, and the nature of supply and demand. These rules mainly come into play when the demands exceed the supplies (Eisele, 1988). In allocating water for a particular growing season, the water resources are estimated from reservoir carry-over storage and a prediction of the watershed yield in the subsequent months. An estimation of the reservoir releases can be obtained from reservoir operation model studies, but actual water resources available during the irrigation season are determined as the season progresses. Once an estimation of the available water resources is available, determination of the cropping pattern and acreage to be planted for each crop are established. Day to day irrigation water demands are determined by the ditch riders who serve as a link between the agency and the water users. This information is used to determine the reservoir releases and the routing procedure.

Water delivery rules define the way in which the water is scheduled and delivered in space and time over the command area (Replogle, 1984). The water delivery rules are established and enforced by the water control bureaucracy in order to attain its water allocation objectives. The water delivery rules, sometimes referred to as "scheduling rules", determine the flexibility of the conveyance system in meeting the highly variable water demands. Special facilities required to execute the deliveries are (Walker, 1986):

1. Sufficient canal capacity;
2. Regulatory storage close to the project to reduce the lag time between orders and deliveries;
3. Measurement of the quantity delivered;
4. Data base on water orders and amounts delivered; and
5. Trained personnel for the operation and maintenance of the project.

Most irrigation schemes adopt a scheduling strategy which utilizes a combination of continuous flow, rotation, and demand scheduling. The rules should be flexible to suit different physical and social-economic settings and to different levels of water shortage. Replogle (1984) stated that,

> Most schedules have been in place for several decades and were chosen for reasons usually valid at the time. It does not necessarily follow that those reasons are still valid and many projects need to closely examine their scheduling policies. (p. 120)

Replogle (1986) suggested introduction of multiple scheduling policies as an institutional measure to improve system performance. He observed that:

1. The most flexible schedule that can be supported on a section of the canal should be adopted;
2. Farmers adjacent to large main canals should be assigned limited rate demand schedules to take advantage of the residual storage and bypass flow capacities in large canals; and
3. More difficult areas may be assigned a rigid schedules.

He also noted that these schemes have not been considered due to shortsightedness. Theoretically, areas served by demand schedules would release management's attention for concentration on areas with less desirable schedules. He envisaged that some or all the following features are required in order to implement workable multiple scheduling policies:

1. Total reconstruction to cater for new policies;
2. Repair and replacement with increased capacity;
3. Use of automated systems;
4. Using freeboard capacity and canal storage volume to create regulatory reservoirs. (This complicates the operation in that changing canal water levels in turn requires corresponding farm turnout changes to compensate for the changing water levels in order to maintain stable turnouts flows);
5. Level top canal operations. (Canals could be laid at a grade and raising the freeboard at the lower ends so that one ends up with a level top. This construction allows for zero flow operations. Due to extra storage capacity the water level in the reach can be maintained even with a varying outflow); and
6. In-line reservoirs to reduce the canal capacity requirements, need for canal automation, lag time, and operational losses.

Flow Regulation Activities

Management of flows involves the hydraulic execution procedures that are required to distribute water in accordance with the allocation plans. Improved control of water will increase the farmers' productivity per unit of land, water, labor, and capital as well as create conditions conducive to long term sustenance of irrigation by alleviating the problems of environmental degradation, system deterioration, loss of productivity, and social conflicts. Water control activities are extremely important and should consider both long and short term contexts. Long term activities require realistic forecasting so water can be stored at strategic locations (canal storage or regulating reservoirs) and be made available when and where it is needed. Forecasting storage requirements, in case the irrigators reject the water they ordered, should also be considered. Short term activities deal with day to day operations. The main problems in managing the regulating flow are attributed to the poorly understood behavior of flow regulators, unexpected variations in demand, and the unpredictable time lag peculiar to each canal section. The basic questions in the management of flow are: (1) Where and when are flow control changes required? and (2) How and when should the changes be made?

Burt (1983) identified three motivations for improvements in canal control logic and hardware:

1. The Canal operators have the potential to properly determine water delivery schedules based upon an assessment of agronomic needs;
2. The desire to make adjustments of water levels and gates easier, while still maintaining the same delivery schedule criteria; and
3. The canal control improvements would enable canal systems to automatically respond to user's demands, possibly without advance notice for receiving or turning off water.

Flow Regulation Concepts

The main purposes of open-channel regulation are to:

1. Raise water as high as economically feasible to serve the command area by gravity; and
2. Control water level and discharge so as to minimize turnout flow fluctuations, canal lining deterioration due to changes in hydrostatic pressures, and canal breaching.

Flow regulation can be accomplished by manual or automatic control depending on: the nature of farm water demand; size of the system; availability of funds, electric power, communication systems, and skilled artisans; skill and mobility of canal operators; local traditions and a variety of other cultural and social influences. Over the years, several water control methods have been developed. They are upstream control, downstream control, combination of upstream and downstream control and dynamic regulation.

Upstream Control

Upstream control is the oldest flow regulation method. It aims at maintaining a constant water level upstream of each structure thereby maintaining a constant head on the turnouts in the reach. This method requires that adjustments of flows at the headworks and flow depths at various points in the canal be made so as to keep the canal reaches full at whatever discharge. This method of control is often established using straight, oblique or duck-bill weirs. Use of long-crested weirs controls upstream water level within narrow limits thereby making flow depths in the reach more or less independent of the variations in flow rate. Gates and pumps are also used. Gates have an advantage over weirs due to their low head loss and minimal sedimentation behind the structures. Pumps are used where water has to be raised to a higher level canal.

Operation of an upstream control system involves the discharge into the network of a predetermined flow rate that is based on the project's water allocation. A rate of flow controller is required at the headworks to ensure that the inflow matches the demand, wedge storage, and operational losses (see Fig. 1). The water is subsequently distributed to various branches and reaches in accordance to the water delivery schedule. The control structures (weirs, stop-logs, slide or radial gates, pumps, etc.) are designed and operated to maintain a constant volume in the reach. In a manually operated system such an operation is very complex and is normally based on the operators' experience and knowledge of water levels throughout the system and the water demands being delivered. In projects where large fluctuations in demand are commons, the operator is required to make numerous visits to adjust the control structures as changes in demand occur.

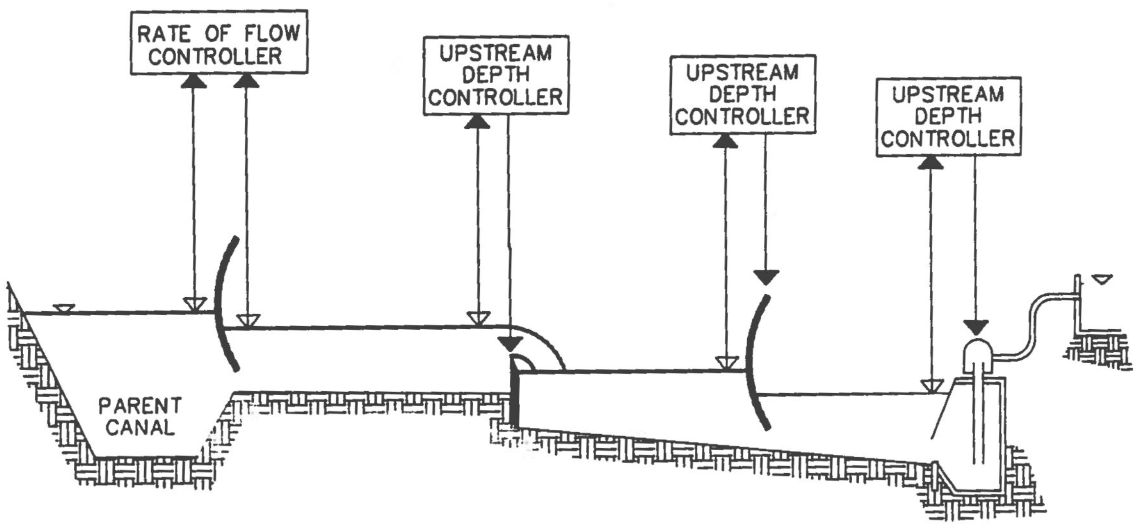

Figure 1. Diagrammatic Representation of Upstream Control System

Figure 2. shows the additional storage required when reach flow rate is increased from Q_1 to Q_2. This additional reach storage is partly responsible for the high time lag in transmission of water. To increase discharge at the lower end, a certain amount of discharge is stored in successive upstream reaches. Conversely, decreasing downstream demand leads to unavoidable losses because decreases in headworks inflows do not have any effect on downstream flows until the water level has dropped to the equilibrium level for that discharge.

The response time is a function of flow rate, change in demand, number of control structures, slope of the canal and distance from the source to the demand point. Burt (1987) stated that:

> Upstream control is by nature a control system which passes all of the problems to the downstream end on the system. Water levels can be controlled on the majority of the canal, but all of the errors show up at the downstream end. At that point, it is often a case of "feast or famine" for water users. (p. 89)

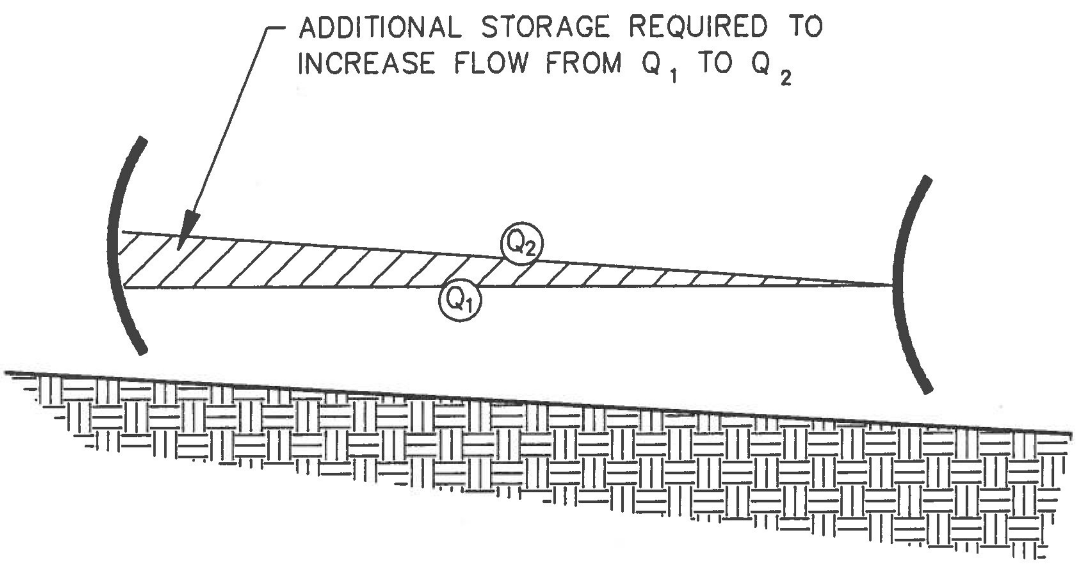

Figure 2. Water Surface Profile for Upstream Control Network.

Downstream Control

Downstream control facilitates total automation because it allows for control of water levels and also adjustment of flow rates to meet the demand. Downstream controlled systems are equipped with control structures whose settings are controlled by the water level in the downstream reach (see Fig. 3). Each change in flow depth is transmitted to the upstream gate where corresponding adjustments are made. This step-by-step transmission of change in flow conditions causes the overall supply to the network to be adjusted to suit the demand.

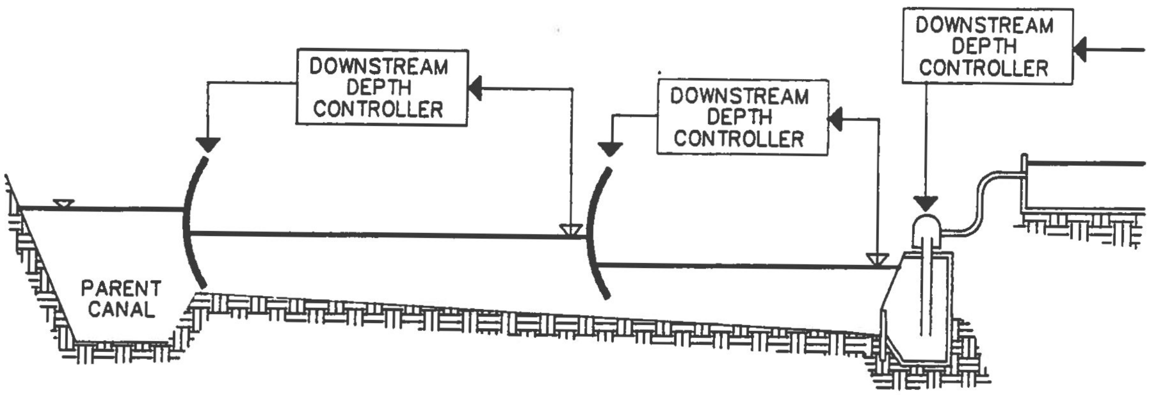

Figure 3. Diagrammatic Layout of Downstream Control Network

Figure 4 shows the water surface profile when the flow in the reach is decreased from Q_2 to Q_1. It can be observed that the change in flow depth is higher at the downstream end of the reach and therefore level top canals are required. When the demand is reduced no water is wasted because the additional amount that may be added into the reach during the gate closure time is stored in the reach where it remains for subsequent withdrawals. This system therefore, requires larger canals to contain the volume of water corresponding to zero flow which are much higher that those at maximum flow.

The quest for further improvements of the downstream control concept led to the development of the BIVAL, the EL-FLOW, Zimbelman, and CARDD control techniques. BIVAL technique employs two water level sensors situated at both ends of the reach. This technique, patented by Sorgreah of France provides downstream control and minimizes the canal works because the canal bank does not have to be horizontal at the upstream end. When the flow rate changes, the water level curve pivots around a given axis situated in the reach at a point determined by the proportion of the upstream and downstream flow depth changes. Consequently, the canal bank at the upstream end can remain parallel to the bottom of the canal but the downstream part must be kept horizontal (see Fig. 5). This technique maintains nearly constant reach storage regardless of the flow rate because the pivot point is normally at the mid-point of the reach. It has a good hydraulic stability because additional flow is not required to increase wedge storage when demand increases (Chevereau and Schwarte-Benezeth, 1987).

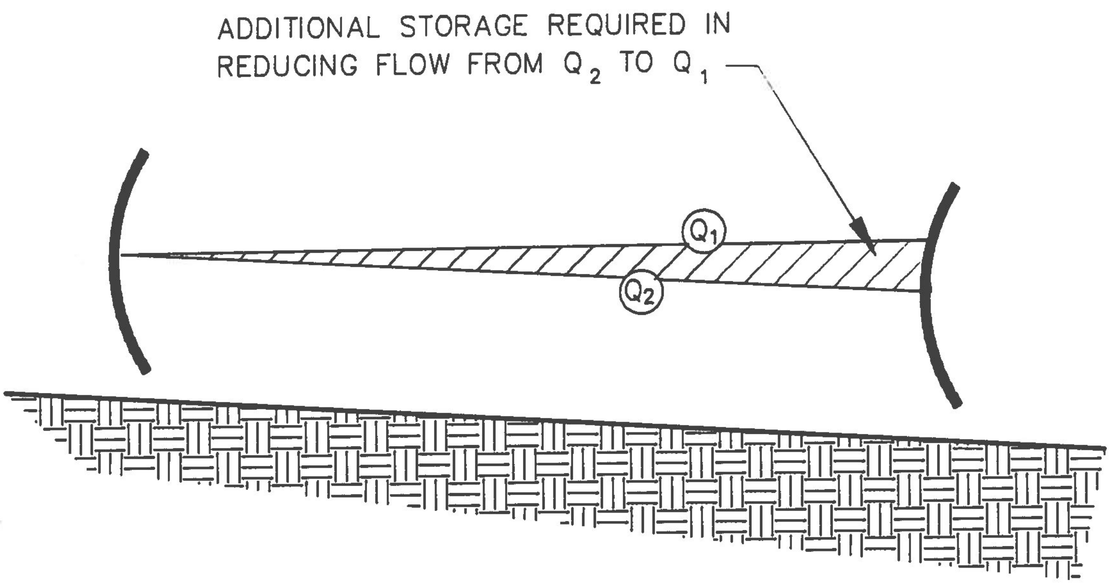

Figure 4. Water Surface Profile for Downstream Control Network.

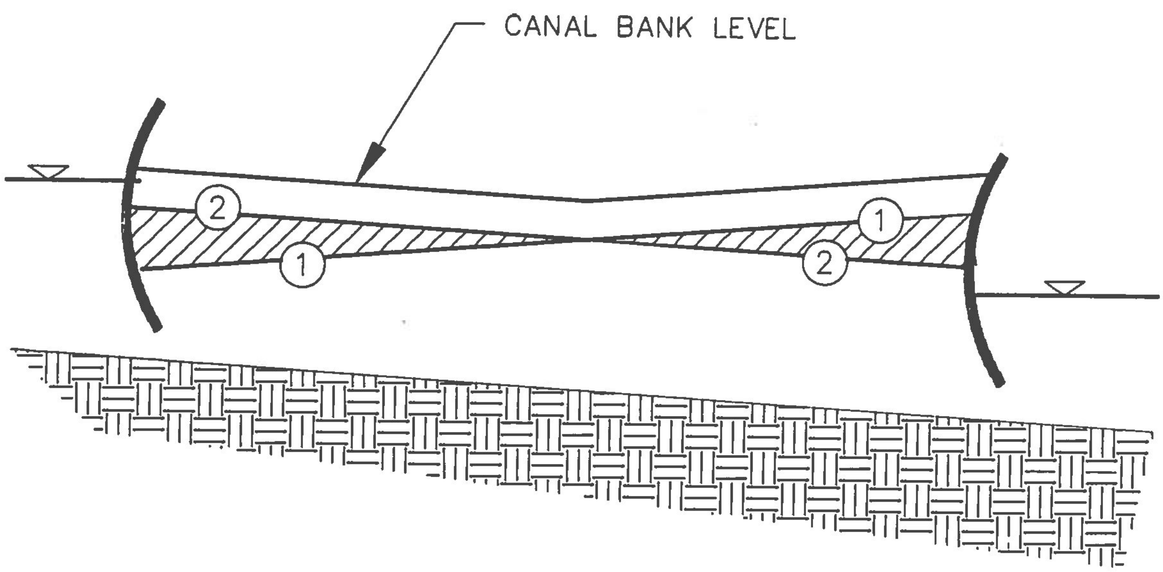

Figure 5. Water Surface Profile for BIVAL Downstream Control

EL-FLO plus Reset algorithm was designed by the U.S. Bureau of Reclamation (Buyalski and Serfozo, 1979). The algorithm is based on control theory concepts and is built into an analog computer. The resulting electronic time delay circuit superceded the cumbersome hydraulic filter. It offers a great deal of versatility and flexibility of operating automated flow regulation by smoothly regulating changes. It requires only one downstream depth sensor (Buyalski and Serfozo, 1979).

Zimbelman (1981) developed a logical control algorithm which requires one sensor at the downstream end and determines the upstream gate movement based on the deviations of water level from the target and a time-adjusted rate of change. Burt (1983) developed the CARDD (Canal Automation for Rapid Demand Deliveries) algorithm. This algorithm requires three to five water level measurements within the reach. The rationale of using multiple water level sensors was to reduce the time lag between upstream gate action and downstream response. CARDD method of downstream control is based on the following hypothesis:

1. The water levels within a reach reflect the flow rate balance into and out of the reach;
2. A complete canal system with local controllers can respond quickly to a change in one reach and if a local controller responds quickly to a change anywhere in the reach it monitors, the hydraulic connection between reaches will have the same effect as electrical connection between controllers;
3. A controller can be developed, without expensive theoretical analysis to have the proper timing and magnitude of gate response thereby reducing the water level fluctuations and achieving flow stability within a reach and within the canal network; and
4. Downstream control can be implemented on canals with sloping banks (Burt, 1983).

Combined Upstream and Downstream Control

Although downstream control ensures a demand oriented delivery, it presents the following disadvantages:

1. Water users consumption cannot be effectively checked. Consequently, in times of water shortage the upstream users are deprived of water whilst the downstream users continue drawing their full demands; and
2. Breakdown in the system that results in failure to pass water from one reach to the next, such as blockage of gates or pump breakdown, would lead to rapid drawdown in the lower reaches.

These disadvantages led to the development of composite gate and a longitudinal combination of upstream and downstream control concepts to take advantage of opportunities presented by both methods.

The composite gate system employs NEYRPIC hydro-mechanical gates which act as downstream control gates when the demand is equal to the supply and as upstream control when the supply exceeds the demand. When demand exceeds supply, the gates close before the upstream reaches are drained thereby allowing the available flow to be shared out fairly among the users. This technique is implemented in the main canals while the secondary canals are equipped with downstream control structures. Composite control gates function as compensating reservoirs by absorbing differences between demand and supply that may be caused by:

1. Fluctuating demands supplied by constant supply;
2. Storm water inlets into the system; and
3. Canal breaches or other breakdowns.

Longitudinally combined system employs downstream control for the main canals in order to minimize the response time, save water, and facilitate laying canals on mild slopes. Secondary canals are equipped with upstream control to prevent over-drawing upstream reaches and enable water consumption to be checked (Kraatz and Mahajan, 1975).

Dynamic Flow Regulation

Kraatz and Mahajan (1975) defined dynamic regulation as

> a means of seeking and implementing the regulation optimum in relation to a set of conditions existing at a given moment and in line with a given number of criteria. In this context, the set of conditions refers to water levels, flows, gate positions, valve opening, etc., and the criteria may be consumption forecasts, physical and economical constraints and safety margins. (p. 24)

In dynamically regulated systems, all the reaches of the canal take place in meeting demands and absorbing the deviations between supply and demand. This is accomplished by simultaneous measurement of water level and control structure settings at various sections in the canal and evaluating and implementing required structural setting adjustments to meet the induced change within the shortest lag time while at the same time minimizing hydraulic transients. Thus, unlike the downstream control which is blind to what happens in other parts of the system except the reach downstream, dynamic regulation is sensitive to flow condition changes throughout the network (Rogier, Coeuret, and Bredmond, 1987).

Dynamic control bring about considerable savings in civil works compared to downstream control because of:

1. Canal banks don't have to be kept horizontal;
2. A reduction in volume of balancing reservoir brought about by

more accurate regulation; and

3. A reduction in overall size of canal reaches and structures (Kraatz and Mahajan, 1975).

Two of the widely and familiarly known aqueducts that use dynamic regulation tactics are the California Aqueduct in United States and the Canal de Provence in Southern France. In the California aqueduct the demands are known in advance. The aqueduct is controlled by information from two computer programs (Amorocho and Strelkoff, 1965). The first program calculates the contemplated flow rates in the reach. The second program calculates gate openings based on unsteady flow simulation of the system. The gates are operated in a "timed gate operation" mode in which one or all the gates at each control can be operated and timed to start and stop several times before reaching the predetermined gate position to reduce canal transients (Dewey and Madson, 1976). Water flows from all upstream reaches into downstream reaches simultaneously thereby reducing the time lag in the system. Dynamic regulation of Canal de Provence is based on simultaneous sensing of water levels at many points within the network. A control center is used to collect, check and interpret the data and monitor execution. A mathematical model is used to determine gate movement and pumping schedules (Rogier, Coeuret, and Bremond, 1987).

Hydraulic Structures

Open-channel conveyance and distribution networks require many different types of structures to effectively and efficiently convey, regulate and measure the canal discharges and also to protect the canal from storm water runoff. They can be grouped into conveyance, regulation, measurement, protective, and structural components and appurtenances (Aisenbrey, 1983).

Table 1. Canal System Hydraulic Structures

Purpose	Structure
Conveyance	Canals, inverted siphons, road crossings, bench flumes, drop structures
Regulating	Headworks, cross-regulators (weirs, gates, etc), pumps, turnouts, diversions, wasteways, regulating reservoirs
Measurement	Flumes, weirs, gates, orifices
Protective	Cross-drainage structures, wasteways
Appurtenances	Transitions, energy dissipators, pipes and pipe appurtenances

For the purposes of flow routing modeling the structures can be divided into two broad categories, conveyance and control structures. Conveyance structures influence the transfer of water from the source to the demand point and establishes the depth, discharge, and reach storage relationships. The principles governing the relationship between depth and discharge depend on the rate of energy dissipation due to friction, the slope of the channel, and the transient nature of the flow. Control structures break the conveyance into discrete reaches. The type, size, and settings of the control structure influences the water surface profile in the reach and determines the relationship between the reach flow conditions and those existing in the downstream reach.

Control Structures

In this study, control structures are divides into two categories: (1) the inline or cross-regulators; and (2) turnout structures. Inline structures regulate the water surface profile in the reach thereby establishing the distribution of incoming flow between outflow and reach storage. Turnout structures include all structures that facilitate lateral bulk outflow. They include secondary canal off-takes, wasteways, and farm turnouts.

Control structure equations. In irrigation canals, discharge through the control structures take the form of overflow or underflow or a combination of both. The flow regime may be free flow or submerged flow. Free flow, also referred to as modular flow, occur when downstream flow conditions have no effect of discharge through the structure. The equations of discharge cannot be exactly determined due to: (1) variations in flow patterns from one structure to another and from one discharge to another; and (2) the fact that the number of variables involved defy rigorous analytical approach (Brater and King, 1976). The approximate equations generally used are derived from the Bernoulli equation.

Overflow structures include weirs, stop-logs and flow measuring flumes. The equation of discharge over a sharp-crested weir is:

$$Q = C_d W (2g)^{0.5} h_u^{1.5} \tag{1}$$

where Q = discharge (L^3T^{-1}); C_d = coefficient of discharge that combines the effect of vena contracta, head loss, velocity of approach, and kinetic-energy correction factor in the Bernoulli equation (dimensionless); W = effective crest width (L); $h_u = (y - h_s)$ height of water above the crest in the approach channel (L); y = flow depth in the approach channel (L); h_s = sill height (L); and g = acceleration due to gravity (LT^{-2}).

Skogerboe et al. (1986) presented general rating formulas based on free and submerged flow conditions (see Fig. 6):

$$Q_f = c_f h_u^{n_f} \tag{2}$$

$$Q_s = \frac{c_s (h_u - h_d)^{n_f}}{(-\log S)^{n_s}} \tag{3}$$

where f = subscript denoting free flow conditions; s = subscript denoting submerged flow conditions; Q_f = free flow discharge (L^3T^{-1}); h_u = head upstream of the critical section (L); n_f = free flow exponent (dimensionless); c_f = free flow coefficient; Q_s = submerged flow discharge (L^3T^{-1}); h_d = head downstream of the structure (L); n_s = free flow exponent (dimensionless); c_s = free flow coefficient; and S = h_d/h_u, submergence (dimensionless).

Walker (1987) presented the following alternative submerged weir flow equation:

$$Q_s = Q_f \left[1 - \left[\frac{h_d}{h_u} \right]^{1.5} \right]^{0.385} \tag{4}$$

The terms are as defined above.

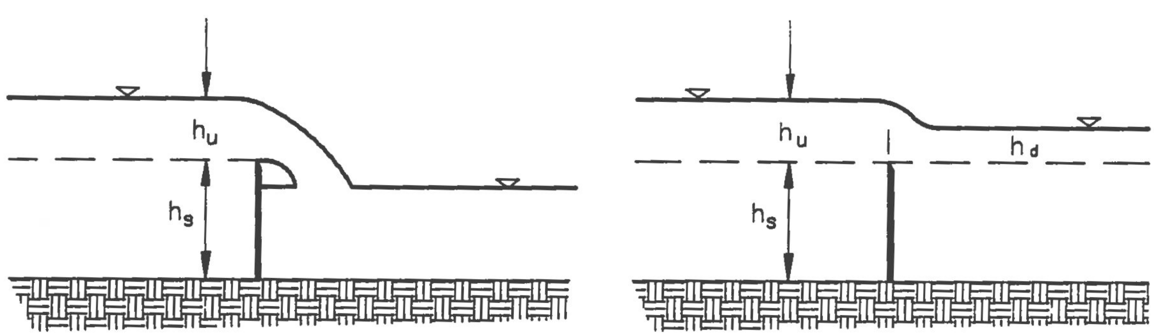

Figure 6. Flow through an Overflow Structure

Any type of opening in which the upstream water level is higher than the top of the opening can be classified as an underflow structure. Underflow structures include orifices, culverts, and gates as illustrated in Fig. 7. The general equations governing flow through an orifice are:

$$Q_f = C_d A (2gh_u)^{0.5} \tag{5}$$

$$Q_s = C_d A [2g(h_u - h_d)]^{0.5} \tag{6}$$

where A = b*W, area of the gate (L^2); b = gate opening (L); W = gate width (L); and the other variable are as previously defined.

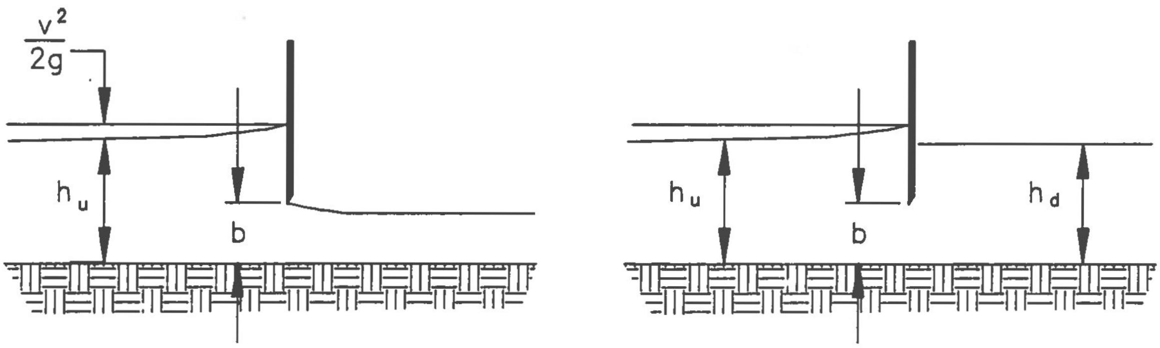

Figure 7. Flow through Orifice Type Structures

Flow through a culvert depends on the part of culvert that exerts primary control, the inlet or the outlet. Inlet control exists when the ability of the culvert to pass flow is constrained by the inlet. The inlet ability is controlled by the upstream water depth; the entrance geometry; barrel shape and cross-sectional area; and the type of inlet edge (Skogerboe et al, 1986). Figure 8 shows inlet control flow for submerged and free entrance flow conditions. The occurrence of critical depth under free entrance conditions permit the determination of discharge when upstream flow depth is known using Eq. 2. Discharge under submerged flow conditions can be evaluated using Eq. 3.

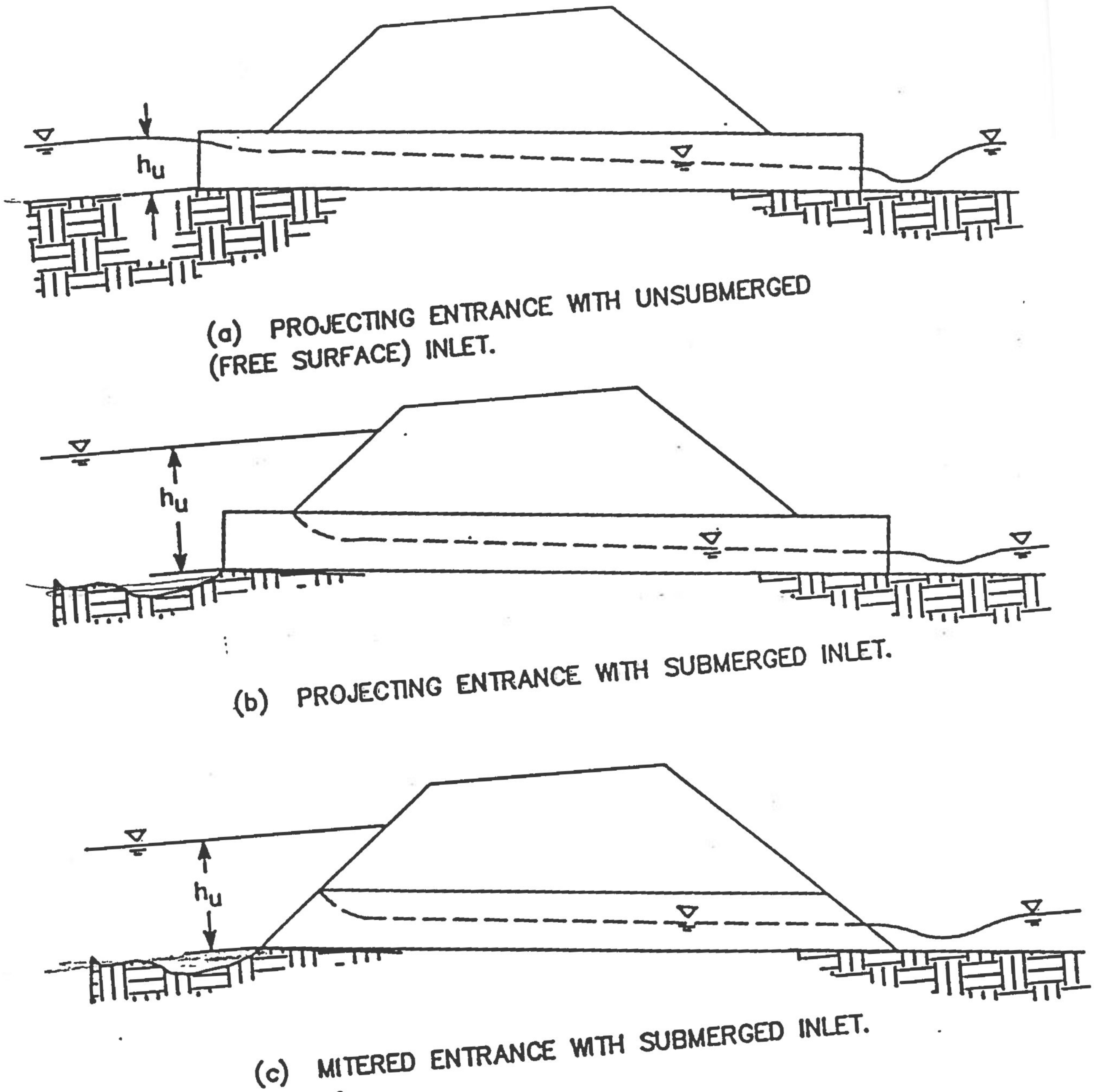

Figure 8. Inlet Control Flow Conditions (Skogerboe et al., 1986)

Outlet control exists when the ability of the culvert to pass flow is constrained by the conditions of the outlet. Figure 9 shows four possible flow conditions. Free surface subcritical flow exists when flow depths at any section of the culvert are greater than critical depth. Under such flow conditions both upstream and downstream flow depths measurements are required. Skogerboe et al. (1986) presented a procedure for preparing dimensionless plots of parameters describing stage discharge relationships and developed the following equations:

For free-surface outlet conditions:

$$Q_s = \frac{c_s(h_u + z - h_d)^{n_f}}{(-\log S)^{n_s}} \tag{7}$$

where $S = \frac{h_d}{h_u + z}$; and z = drop in elevation between culvert invert between the inlet and outlet (L);

For submerged outlet control:

$$Q_s = c_{so}(h_u + z - h_d)^{n_{so}} \tag{8}$$

where c_{so} = submerged outlet control coefficient which is the value of Q when the operating head is equal to one; and n_{so} = submerged outlet control flow exponent, which is the slope of the submerged outlet control flow rating when plotted on logarithmic paper.

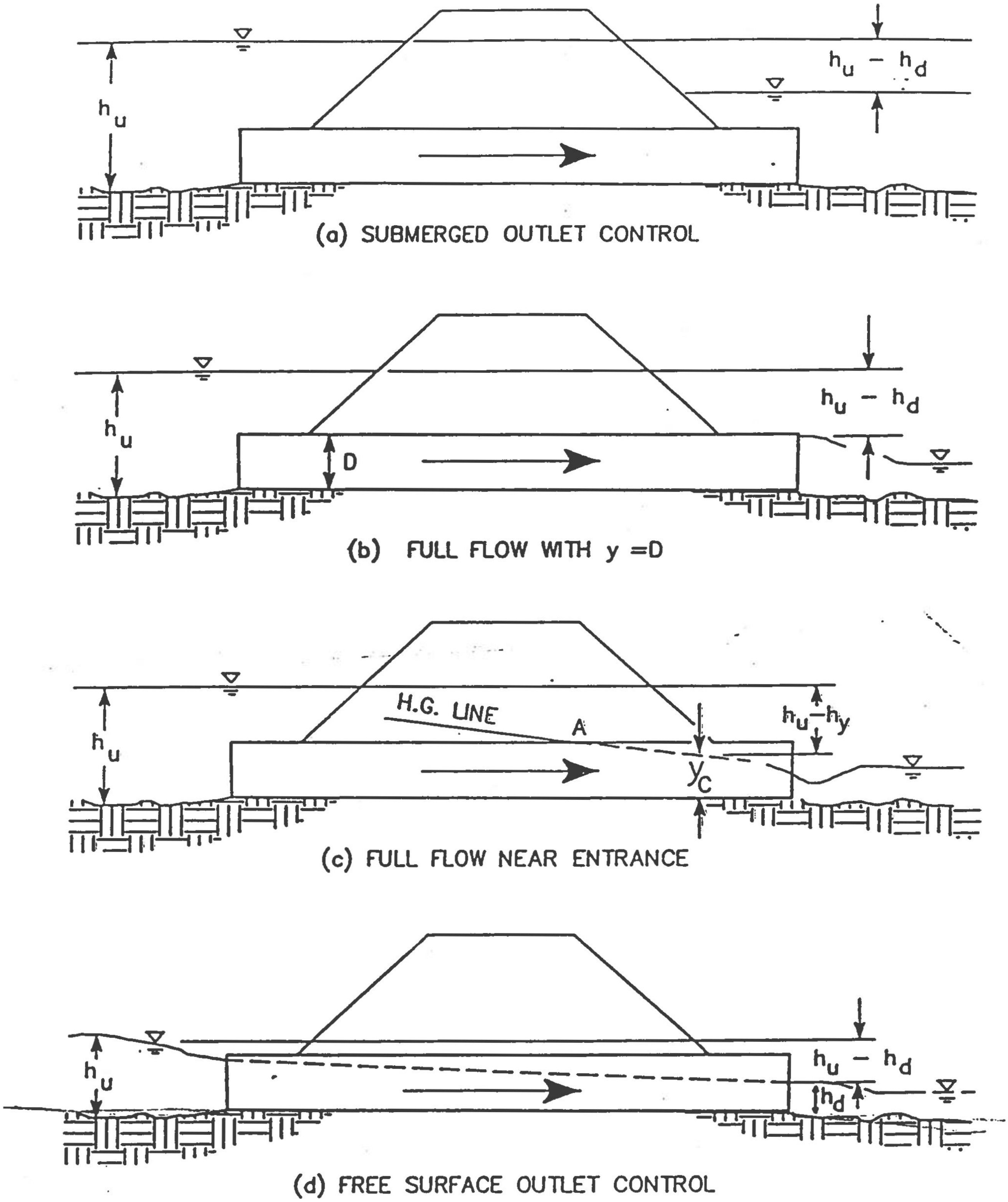

Figure 9. Outlet Control Flow Conditions (Skogerboe et al., 1986)

Location of structures. Location of inline control structures depends on the canal slope, the flow control concept used, number and size of turnouts along the canal reach, and the desired flexibility in operating the system. Burt (1987) noted that control structures should be located where they provide the following:

1. Sufficient head on all the turnouts even during low flow conditions; and
2. Minimize water level fluctuations that result from inflow and/or outflow rate changes.

Figure 10 shows the effect of reducing gate spacing by half. This reduction minimizes the water level fluctuations at the upper end which results in less turnout delivery variation and less damage to concrete lining and also reduces the wave travel speed due to reduced wedge storage.

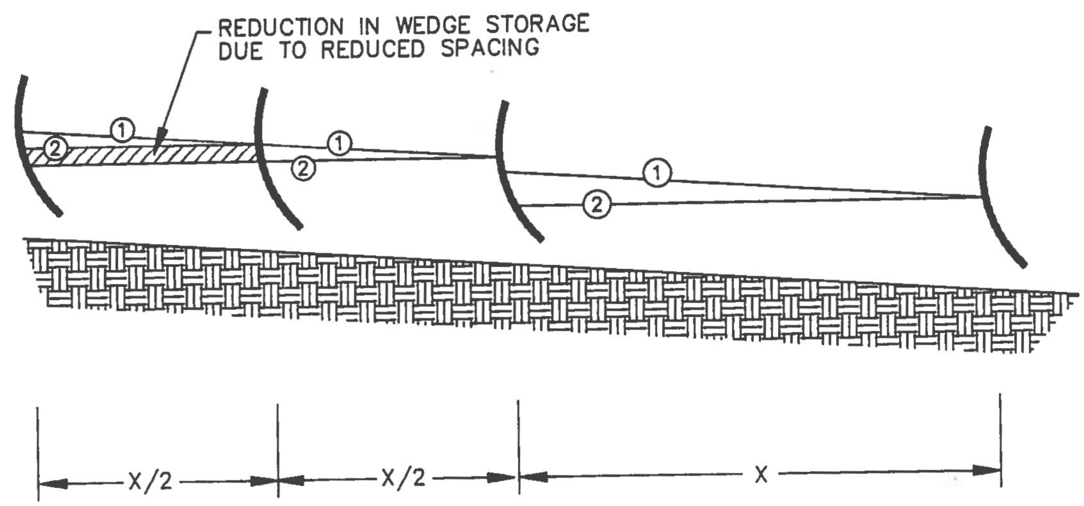

Figure 10. Inline Structure Spacing on Upstream Control System (Burt, 1987)

The location of secondary canal off-takes and farm turnouts is affected by the topography, command area location, canal network layout, and the water control method used. Figure 11 shows the preferred zone for locating turnout structures in order to minimize turnout delivery fluctuations without resorting to use of additional flow regulating structures to ensure that turnout discharges are independent of the water variation in the parent canal (Kraatz and Mahajan, 1975).

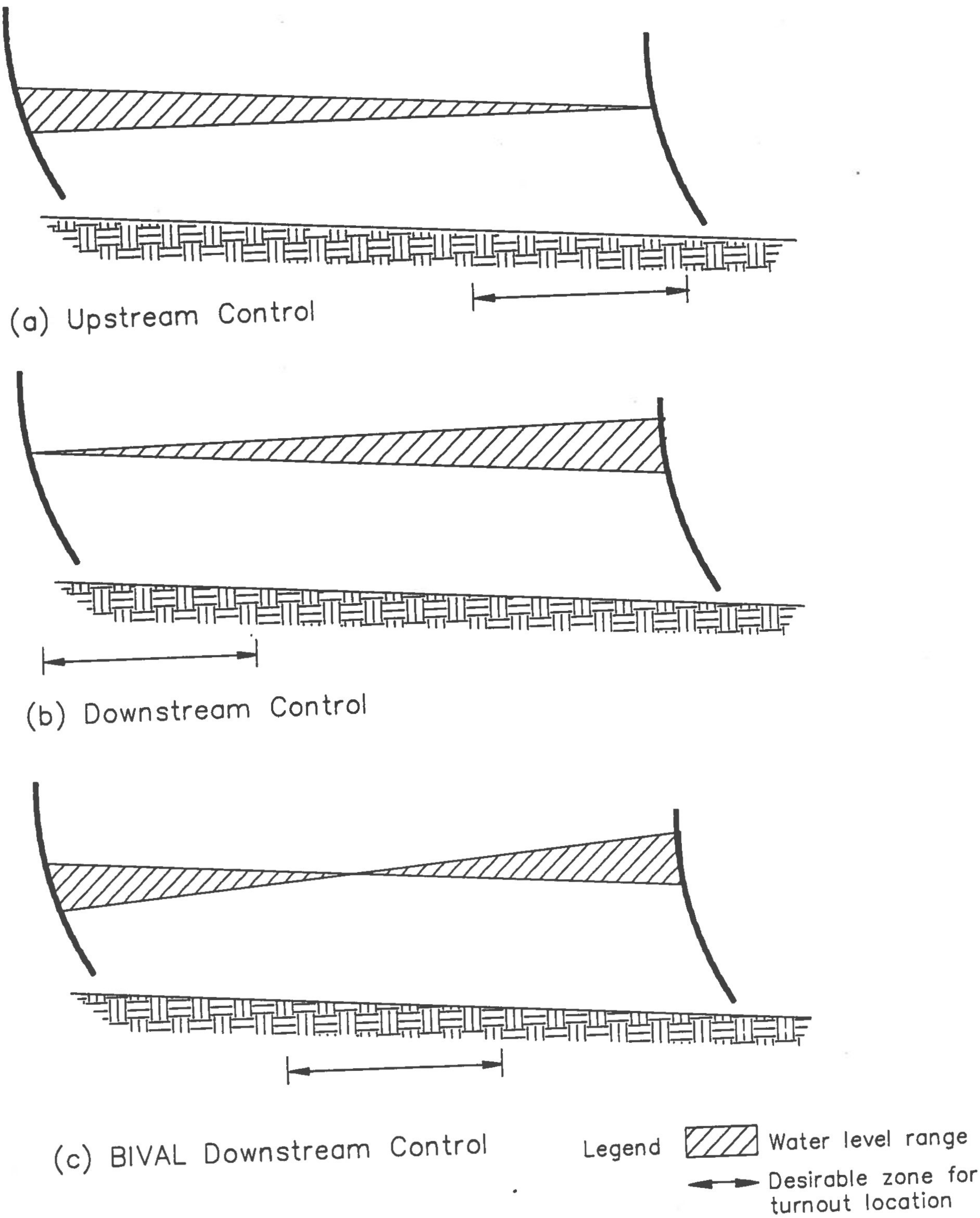

Figure 11. Location of Turnout Structures

Choice of structures. Choice of structures is influenced by the hydraulics of the structure and related flow conditions and the operational requirements. From a hydraulic point of view, the structures selected should minimize flow variation through the turnouts while at the same time meeting the required demands. Walker (1987) presented an analysis on the relationship between inline and turnout control structures. He reported that an ideal combination of a canal inline structure and turnout structures should accommodate large flow rate changes in the canal, as much as 40 to 50 percent, and at the same time result in small flow rate changes through the turnout in order to minimize re-regulation of the structures.

Figure 12 shows the percentage change in flow for $\pm$ 0.05 change in head. The overflow structures have a consistently higher deviation in discharge. Consequently, the effect of varying canal discharge on turnout deliveries can be minimized by using:

1. High heads for turnout deliveries; and
2. Low heads for inline control structures.

Long crested overflow structures or a combination of overflow and underflow structures are normally employed to minimize water level fluctuations in canals that experience huge fluctuations in flow rate. Underflow structures operating under high head provide ideal means of regulation turnout flows.

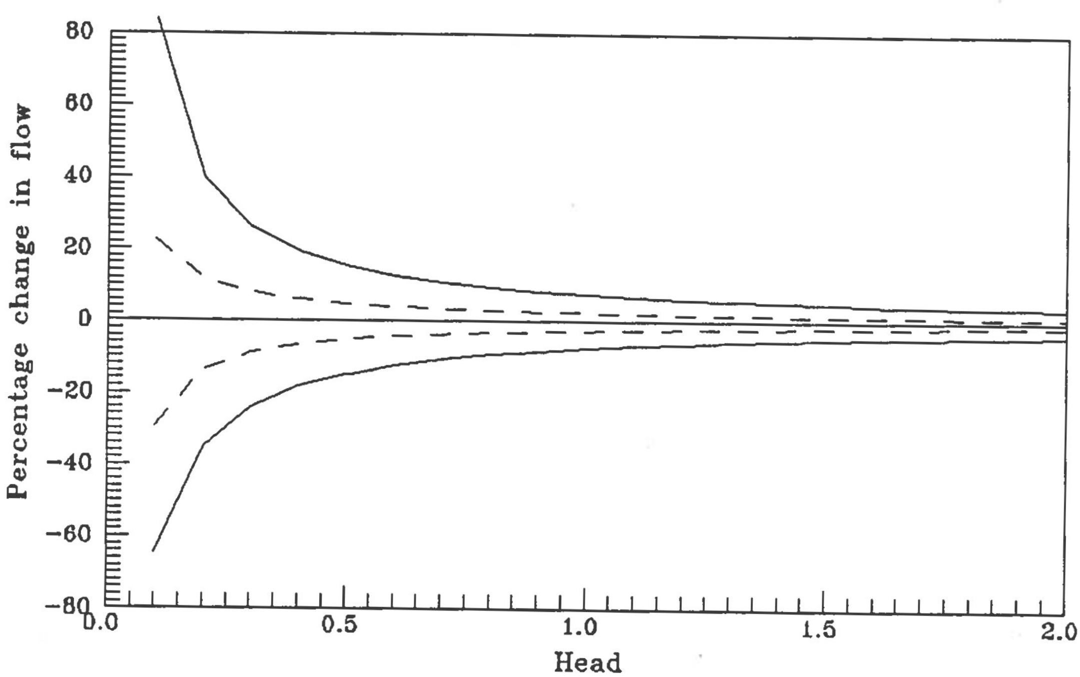

Figure 12. Flow Rate Changes as a Function of Head

Horst (1987) argued that

> Systems which are designed with the aim of being highly flexible and efficient through relatively sophisticated technological means, without taking into account operational capabilities and farmer participation, might actually lead to situations with little flexibility at the farm level and with overall efficiencies lower that expected. (p. 1)

In comparing fixed, open/closed, and gradually adjustable structures (see Fig. 13) he concluded that systems designed for high flexibility and efficiency require sophisticated gradually adjustable structures, measurement devices and a considerable number of highly qualified operating staff that are not readily available. He observed that the flexibility may be nullified by:

1. Creating complex operational policies that are not easily adhered to;
2. Introducing more sources of errors and malfunctioning of regulating structures; and
3. Increasing susceptibility of canal operators to farmers' influence because many of the control structures adjustments are hidden and/or incomprehensible to the farmers.

Storage Facilities

Storage facilities are required for long and short term storage. Long term storage reservoirs store large volumes of water for use during supply shortfalls. Their locations and size is largely dictated by the topography. Supply to the reservoirs is constrained by the available water and canal capacities. Regulating reservoirs provide short term storage for balancing supply and demand thereby minimizing delivery fluctuations associated with difficulties in operating the system. They are operated to ensure that in time the inflow volumes balance the outflow volume and more important that there is sufficient storage capacity to receive rejected deliveries and enough stored water to supply unexpected water demand increase.

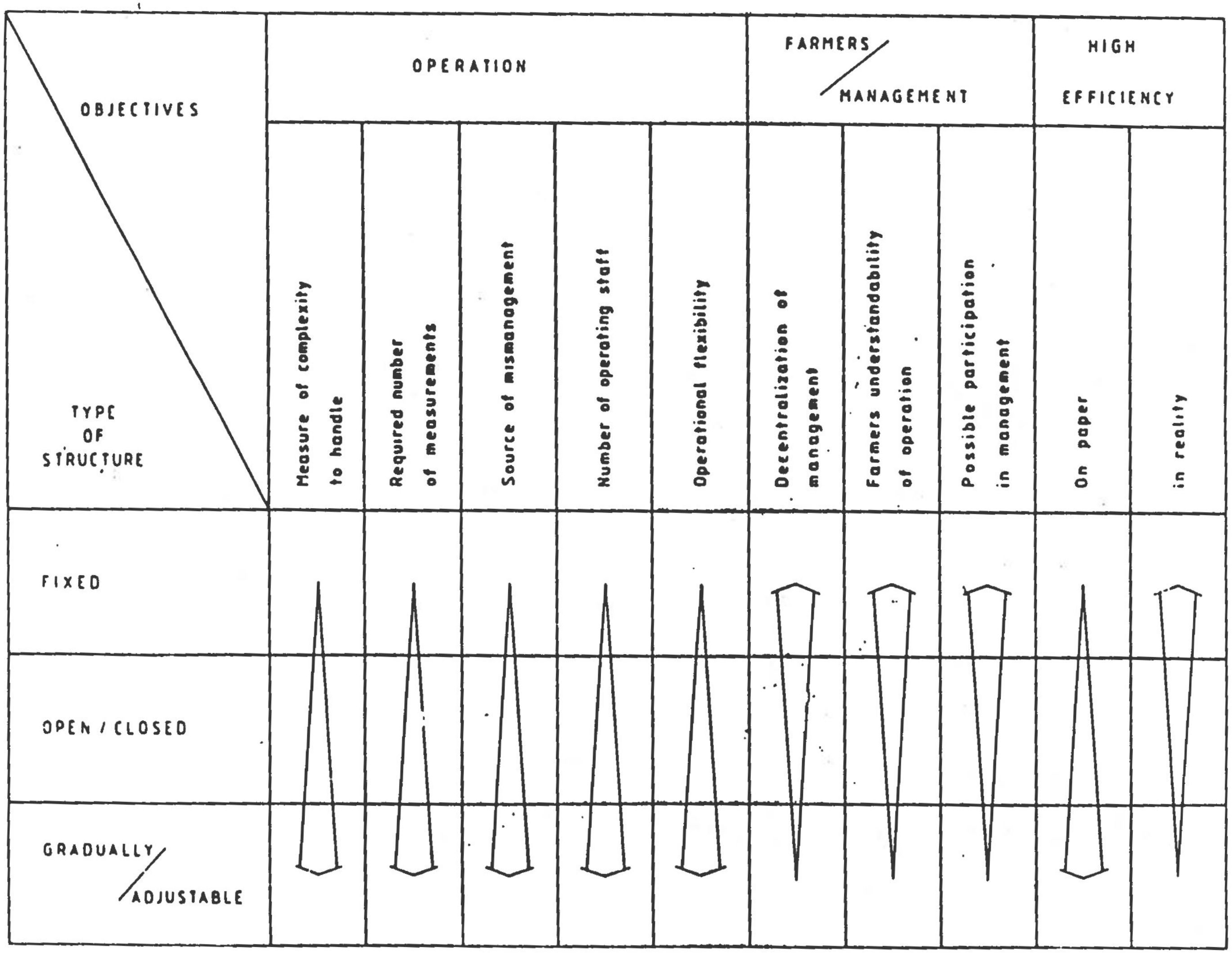

Figure 13. Relation between type of structure and objectives (Horst, 1987)

Mathematical Modeling

General Concepts

Simulation approaches to problem solving involve conceiving and building a model of the real system, and using the model for experimentation to gain some insights into the real world problems. When the model is mathematical, the system is represented by mathematical equations and parameters. Development of good models depends on a thorough knowledge and understanding of the system and ability to discern vital variables and their relationships. In utilizing models as a decision support tool, expert intuition and judgement are required. Biswas (1975) observed that, "while models cannot substitute experience, they can augment it."

The major task of this section is to provide a basis for identifying the essential features of model development and use. The models surveyed do not provide a complete inventory of the existing numerical models, but do encompass those related to irrigation water conveyance and distribution systems management. From the survey it is apparent that the major effort in modeling has been concentrated in a few developed countries. This does not, however, reflect the geographic scope of model applications since a large percentage of irrigation projects are in developing countries.

In discussing the utilization of numerical groundwater models for water resource management, Bachmat et al. (1978) noted the following gaps which also apply to modeling irrigation water systems:

1. Most models developed are not well publicized and readily distributed to prospective users. Consequently, individuals needing models are often unaware of what models are available, nor do they know where they can turn to find this information. This results in unnecessary duplication of efforts as various users embark on developing their own models;

2. Most models are poorly and inadequately documented. Model documentation should at the very least include a description of the models general characteristics, its structure, underlying assumptions, degree of reliability of the results, capability, range of application, specific technical instructions that may assist in the utilization of the model, listing of the code, and a users' manual;

3. A major factor that has hampered the use of models in decision making is the lack of confidence on the part of the users. This is aggravated by the poor communication between the model user and its developer. Also, the model output is in most cases not presented in a way that is meaningful and compatible with decisions that must be made; and

4. Inadequate and/or insufficient data are limiting factors in the use of models for decision making. The reliability of the output of a model cannot exceed the reliability of the input data, thus the model user is confronted with a major question "What degree of accuracy in prediction is required?", so as to invest appropriately in data collection.

Design and Operation Models

Efforts in developing a better understanding of open channel flow have existed since the dawn of man. In the 19th century, the theoretical hydraulic equations governing overland flow, attributed to Barre de Saint Venant, were formulated. Prior to the advent of high-speed computers and numerical analyses, simplified forms of these equations were used to solve specific problems because no closed form solution of the equations is possible. Graphical solutions were mainly used (Chow, 1959). Development of digital computers provided an impetus to the development of rapid, numerical solution techniques.

Earlier applications were mainly confined to overland and river flow problems. Development of large aqueducts, needed for water control and saving techniques, and concern for aqueduct safety spurred application of these equations to open channel conveyance and distribution networks. The design and development of the operational procedure of the California Aqueduct employed extensive computer simulation (Amorocho and Strelkoff, 1965, and Fredericksen, 1969).

Strelkoff (1969) presented the complete one-dimensional hydrodynamic equations of unsteady flow in a fixed-bed open channel of an arbitrary form and alignment and set the stage for subsequent enhancement of overland flow modeling. Wylie (1969) utilized the method of characteristics to solve the Saint Venant equations and applied it to determine gate motions required to produce a desired water surface profile. The model analytically determined the motions of the control structures in the canal so that transient conditions emanating from canal discharge fluctuations could be mitigated. The model analyzed the transition from one steady state condition to another and prescribed the operation of gates so that the final steady flow is established in a minimum of time and that neither the specified flow depth variation nor the rate of change of flow depth would be exceeded.

Mozayeny and Song (1969) used the explicit finite-differencing technique on the characteristic equations in a semi-infinite rectangular open channel to study the effects of initial flood stage height, channel slope and Manning's friction coefficient on an unsteady flow regime. Chaudhry and Contractor (1973) employed the implicit method to simulate surges on open channels. Their program was applied mainly to study river flows where the effects of different values of time and distance averaging parameters were observed for their effect on the diffusion of the wave front.

Ponce et al. (1978) studied the convergence of the four-point implicit numerical models of shallow waves using a linearized version of the Saint Venant equations. They concluded that for dynamic waves, in which friction and inertia dominates, the accuracy is highly dependent on the correct value of distance and time averaging coefficients. Corringa et al. (1979) presented a lumped parameter model which accounted for time delays due to the propagation of perturbations. Falvey and Luning (1979) developed a "gate stroking" model and installed it on the Granite Reef Aqueduct, Central Arizona Project. Their modeling concept followed Wylie's original concept but included turnouts, siphons, and free flowing tunnels. This model is limited to steady state initial conditions. Zimbelman (1981) and Burt (1983) developed algorithms for automatic downstream control of irrigation canals as enhancements for the unsteady state model (USM) developed by Falvey and Luning.

Joliffe (1984) developed a model for simulating flows in dendritic and looped channel networks. This model was based on the implicit solution of Saint Venant equations where partial derivative terms are evaluated analytically and the equations solved for the complete network as a single problem for each time step. Manz (1985) developed a similar model mainly for evaluating the performance of irrigation conveyance systems. Hamilton and Devries (1986) also developed a canal operation model for non-branching canal systems.

Gaps in Canal Design and Operation Modeling

The models developed to date have had significant impact on the improvements of the operation and management of irrigation water conveyance systems. The current models' limitations are:

1. High computational time (Hamilton and DeVries, 1985; Falvey and Luning, 1979);
2. Application to only non-branching systems (Hamilton and DeVries, 1985; Falvey and Luning, 1979; Strelkoff, 1969);
3. Limited range of water control structures (Chaudhry and Contractor, 1973; Hamilton and DeVries, 1985; Ponce et al., 1978);
4. Intermediate turnouts not included (Chaudhry and Contractor, 1973; Corringa et al, 1979);
5. Not available on microcomputers (Chaudhry and Contractor, 1973; Corringa, 1979; Ponce, 1978; Strelkoff, 1969; Falvey and Luning, 1979);
6. Requires steady state initial conditions (Hamilton and DeVries, 1985; Manz, 1985; Strelkoff, 1969; Falvey and Luning, 1979); and
7. No simulation of the canal filling phase (Hamilton and DeVries, 1985; Manz, 1985; Strelkoff, 1969; Falvey and Luning, 1979).

No efforts have been made to develop comprehensive multi-disciplinary methodologies for use in planning, design, operation, and management of the open-channel conveyance system that is easy to install, use and cover a wide range of physical and operation scenarios. The model developed in this work was aimed at overcoming a lot of the above-mentioned limitations. It is intended for use by canal operators, trainees, planners, and multi-disciplinary teams to answer unlimited "What if..." questions on conveyance system planning, design, and operational issues.

CHAPTER III

MODEL DEVELOPMENT

Modeling Strategy

General

The principal objective of this study was to develop a model that can be used to evaluate the hydraulic response of the conveyance network to changes in inflow, type and setting of control structures and channel physical features as well as to determine the best combination of controllable parameters to meet predetermined system performance goals. To accomplish the above objectives, the author began by focusing on understanding the system hydraulics to facilitate application of theoretically sound simplifying assumptions. The simplifying assumptions enabled a mathematical representation of this complex system, and to develop a model that closely approximates the field conditions. In developing the model attempts were made to:

1. Describe the physical processes on a sound theoretical basis so as to make the model readily transferable from one prototype to another;
2. Describe all the system components and their inter-dependencies;
3. Ensure numerical accuracy and stability;
4. Include all phases of main system operation (filling, transient flows and draining);
5. Cater to a wide range of physical configurations and operational scenarios;
6. Achieve rapid execution of the computations and trap as many input and execution errors as possible; and
7. Ensure user-friendliness so that the software can be used by persons with minimal computer, hydraulics, and main system operation skills.

Model features

The canal hydraulic simulation model (CAHSM) is a mathematical model based on sub-critical open-channel flow equations of continuity and momentum. It is capable of mimicking actual hydraulic conditions in a canal network and of technically optimizing the performance of the system by determining the optimal inflow and control structure operation. The model can accurately simulate flow under the following conditions:

1. A branching canal network with a wide range of physical configurations including regulating reservoirs;
2. Submerged and free flow conditions for a wide variety of control structures;
3. Empty canal filling, previously computed hydraulic status, or user-specified initial starting conditions;
4. Three operation modes--(a) user-specified inflow hydrograph and control structure settings, (b) control structure scheduling based on demand and upstream flow depth control, and (c) control structure scheduling based on demand and downstream flow rate control.

Modeling Approach

Hydraulic and hydrologic linkage: Open-channel water conveyance and distribution systems consist of conveyance canals that extend from the water source to the various outlets that supply water to a group of users (see Fig. 14). In modeling such a network, it is necessary to break it up into hydraulic units separated by control structure. Each unit is linked hydraulically to the one upstream and downstream (if submerged flow conditions exist) and hydrologically to the one upstream. These units are called reaches. A series of reaches that are hydrologically linked to the reach upstream by flow through the inline control structure form a branch. The uppermost branch is hydrologically linked to the network headworks, whereas the others are linked to the branch, reach, and turnout that feed them. Figure 15 illustrates how the canal network in Fig. 14 is sub-divided. Table 2. shows the hydrologic linkages of all the branches. Branch 0, reach 0, and turnout 0 represent the network water source.

Table 2. Branch Hydrologic Linkage

	Branch Inflow Source		
Branch	Branch	Reach	Turnout
1	0	0	0
2	1	1	3
3	1	2	3
4	1	3	1
5	2	1	2

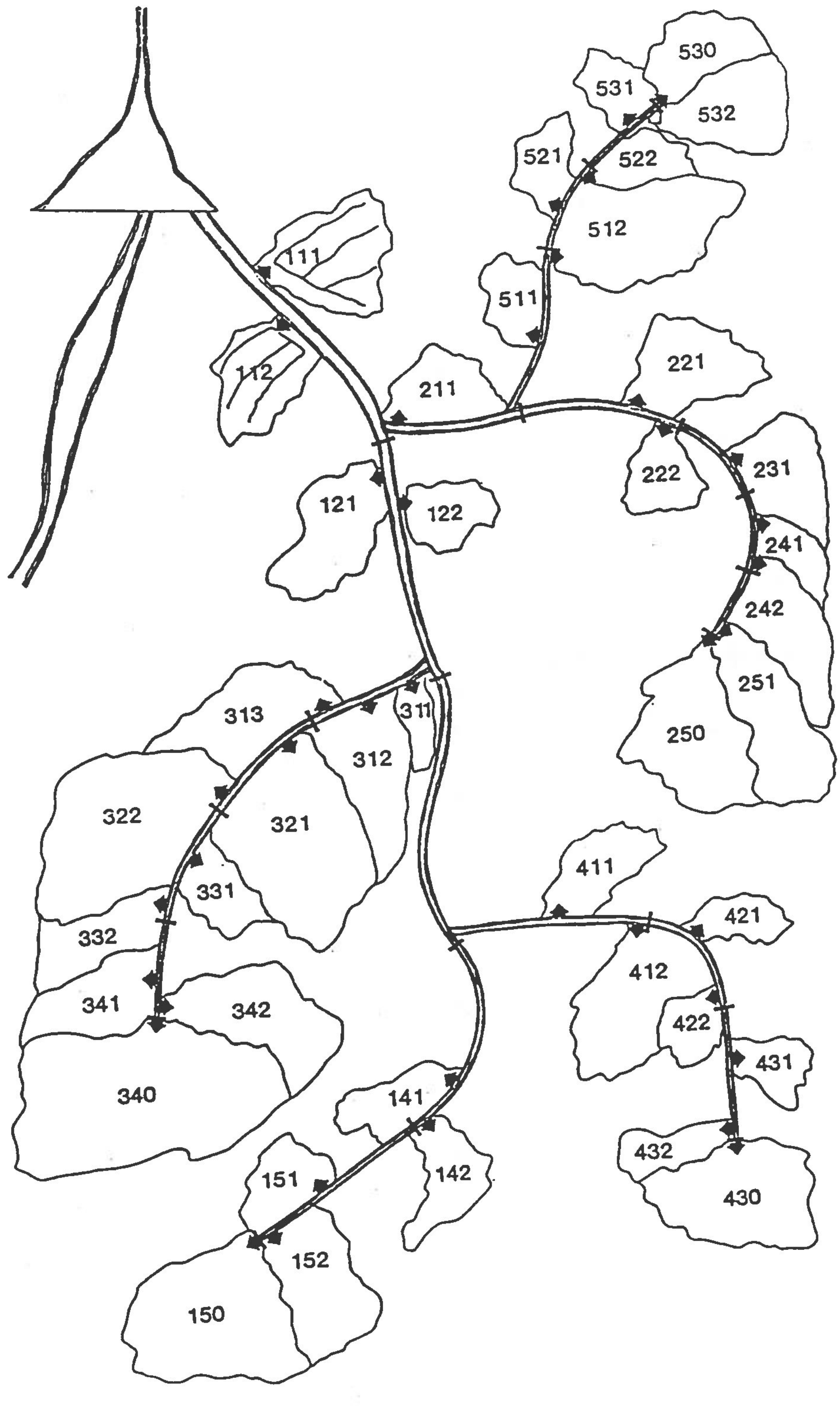

Figure 14. Branching Canal Network Showing Command areas

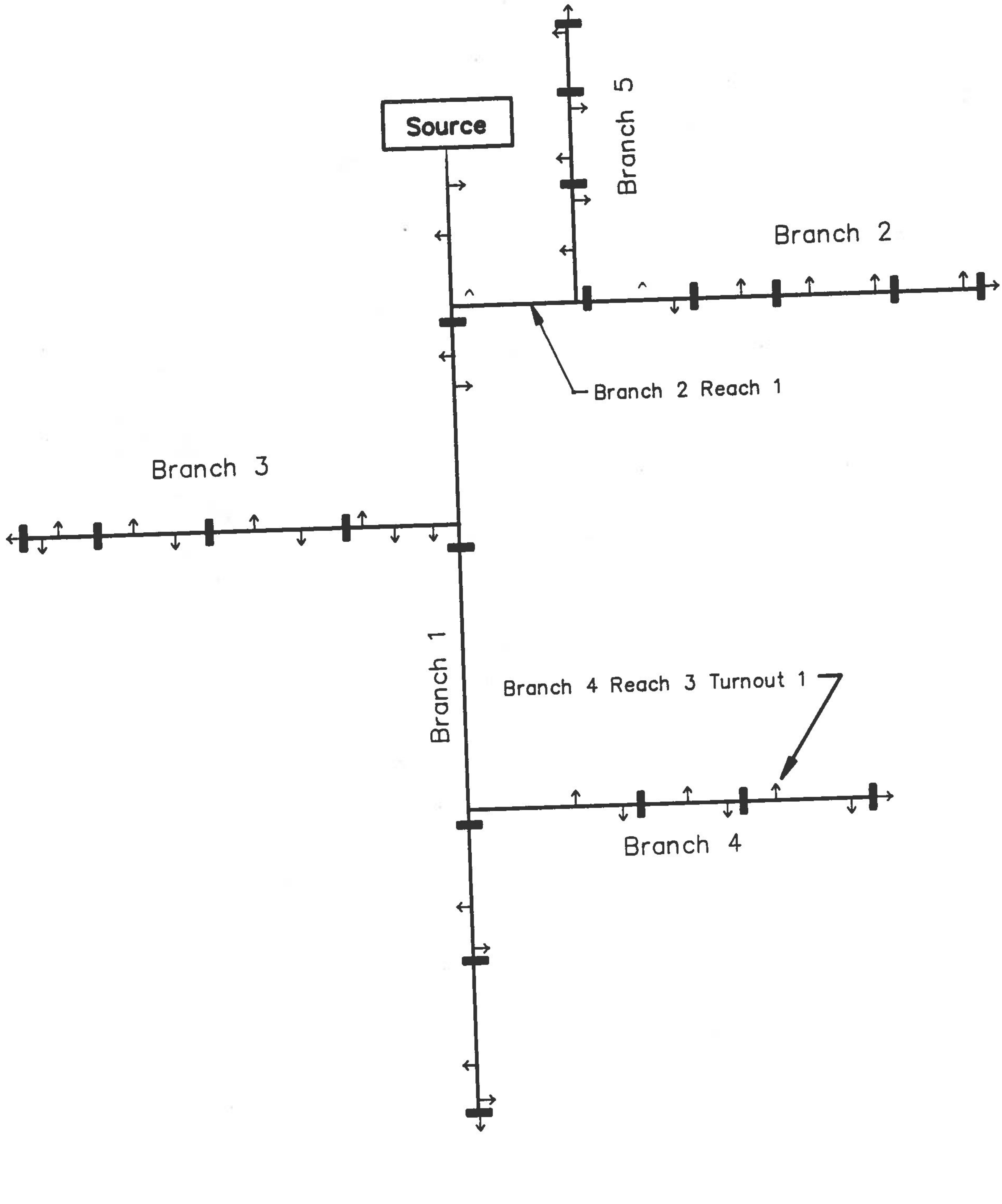

Figure 15. Schematic Representation of Branch Linkage

<u>The operations</u> : The model simulates all phases of conveyance system operation. At the beginning of the irrigation season, most irrigation canals are normally empty and need to be filled. This may also happen in distributary canals, where a rotation delivery rule is practiced or where night irrigation is not practiced and the canals are, therefore, drained at the end of the day. During this phase of operation, there is a need to know how early canal filling or draining should begin to ensure that the water users can get water at the requested time and amount and also to minimize seepage and operational losses. The advancing front analysis is equivalent to the advance phase of furrow irrigation although occurring more rapidly. And as one might expect, the ability to model this phase stems from the successful solution of the advance in surface irrigation (Walker and Skogerboe, 1987).

The post-filling operations of a main system depends on the water delivery rules (continuous, rotation, or on-demand), physical facilities and the social-organizational set-up. When the flow reaches the end of the reach, it may experience ponding when the control structure is an overflow type or if the gate is closed. For overflow structures, the flow depth builds up until it reaches the sill height when overflow starts. During the ponding phase the discharge at the downstream node is zero. In the post-filling phase, there are three possible operating scenarios, namely: (1) predetermined inflow and control structure settings; (2) upstream control operation (3) downstream flow discharge control. The requirement for modeling operation of structures to satisfy these scenarios is discussed in the next section.

Control Structure Scheduling Concept

The operation of the control structures depends on the water control concept being used and the transient nature of flow deliveries. Streeter (1967) presented a "valve stroking concept", in closed pipe systems, in which valve settings changes are made as the flow rate changes so that allowable pressures are not exceeded. He stated that:

> The design or synthesis approach in which certain allowable pressure fluctuations are specified and the changes in boundary conditions that causes the changes in flow to take place are calculated so that steady-state flow conditions are established on cessation of boundary movement (p. 81)

Similarly, in open-channel flow, changes from one discharge to another always create fluctuation in water levels in the reach. The magnitude of the fluctuations can be controlled by the manner in which the control structures are operated. Wylie (1969) developed a method to control canal hydraulic transients by properly varying the reach downstream boundary conditions. This water surface control method is called "gate stroking" and relies on a continuous or a series of discontinuous gate motions to produce predetermined water surfaces. It has not been used extensively because it requires a centralized operation schedule (O'Loughlin, 1972 and Gientke, 1974). Falvey and Luning (1979) adapted this technique to suit unique conditions (turnouts, siphons, free flowing tunnel, pumping plants, etc.) found in U.S. Bureau of Reclamation projects. Application of this method on the Granite Reef Aqueduct, Central Arizona Project, enabled the canal to be operated at nearly the design capacity--a situation with little margin for error because the aqueduct is designed without wasteways or re-regulating reservoirs--and minimized pumping cost (Falvey and Luning, 1979).

In this study, the concept has been extended to cover a wide range of control structures (overflow and underflow, inline and turnout) and upstream and downstream water control concepts. The term "Control Structures Scheduling" is therefore used in this study.

Upstream Control

In upstream control systems, the objective is to minimize the water level fluctuations on the upstream side of the control structure. The structures are therefore operated in response to the flow depth changes upstream of the structure. In modeling this control strategy, an attempt is made to fix the flow depth at the control structure in some pre-determined manner. This is achieved by varying the control structure setting to pass the excess flow when the flow depth attempts to increase or to reduce the downstream discharge is the flow depth reduces.

Downstream Discharge Control

This concept is analogous to the downstream flow depth concept discussed in Chapter II. It is based on the following hypothesis:

1. When modeling the operation of an irrigation water conveyance system, the demand rather than the flow depths are known; and
2. For a particular reach, changes in demand result in corresponding changes in flow depth and in order to minimize the fluctuations in water level, discharge into the reach should be adjusted.

With adequate adjustable water control structures, communication facilities and a sufficient, able and willing operational staff, downstream discharge control can facilitate operational improvements that result in performances comparable to those obtainable under dynamic regulation.

Specified Versus Calculated Control Structure Setting

When modeling the hydraulics of a canal network, upstream and downstream control can be achieved by specifying how the structure are operated or by letting the model determine the required settings. When the user specifies how the control structures are to be operated and the inflow hydrograph, the model calculates the corresponding flow profiles and supplies. In the second option, the user specifies the demand hydrograph of all the delivery points in the network. The model then, determines the technically optimum inflow hydrograph and control structure settings that are required to minimize the deviations between demand and supply based on the control concept used--upstream depth control or downstream discharge control. In both cases the model checks to ensure that the drawdown criteria of the canal network are not violated.

Flow Profile Computation

Governing Equations

Continuity and momentum equations: The water conveyance system consists of the conveyance channels that extend from the reservoir or diversion works to the various outlets. The conveyance capacity, therefore, tapers as water is withdrawn by the upstream command areas. The canal sections can be modeled as open-channel flow sections with sub-critical flow regimes. Unsteady, non-uniform flow in open channels can be simulated accurately using the one-dimensional Saint-Venant equations that are derived from the conservation of mass and momentum in the flow (Chow, 1959; Strelkoff, 1969; Joliffe, 1984; and Walker and Skogerboe, 1987).

The continuity equation is:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + q_o = 0 \tag{9}$$

and the momentum equation is:

$$\frac{1}{gA}\frac{\partial Q}{\partial t} + \frac{2Q}{gA^2}\frac{\partial Q}{\partial x} + (1 - F^2)\frac{\partial y}{\partial x} = S_0 - S_f \tag{10}$$

where Q = discharge (L^3T^{-1}); A = flow cross-sectional area (L^2); y = flow depth (L); q_o = net lateral inflow or outflow in (L^2T^{-1}); x = distance (L); t = time (T); g = acceleration due to gravity (LT^{-2}); S_o = channel slope (dimensionless); S_f = friction slope (dimensionless); and F = Froude Number (dimensionless).

The underlying assumptions in the derivation of these equations are:

1. Hydrostatic pressure distributions are normal to the canal bed;
2. The cosine of the angle of inclination equals 1;
3. The channel has a rigid boundary and is straight and prismatic;
4. A uniform velocity distribution exists;
5. Shear forces can be estimated with the Manning's equation;
6. Effects of surface friction due to wind are negligible;
7. Changes in flow depth are not so rapid as to create a hydraulic bore;
8. Inflows to the channel enter it with a zero velocity in the direction of flow; and
9. The momentum associated with the water that infiltrates from the control volume is negligible.

Equations (9) and (10) are hyperbolic and non-linear with no closed form solution. The numerical solution of these equations is based on the concept of a deformable control volume which is described in detail by Walker and Skogerboe (1987). These solutions were developed as part of efforts to simulate the surface hydraulics of furrow, border, and basin irrigation systems. The modifications necessary to apply the solution to canal networks have dealt primarily with the boundary conditions.

The integrated form of Eq. (9) is:

$$\{\theta[Q_l-Q_r]+(1-\theta)\,[Q_k-Q_m]\}\delta t/\delta x -$$

$$\{\phi[A^l - A_k + Z_l - Z_k]+(1-\phi)\,[A_r - A_m + Z_r - Z_m]\} = 0 \qquad (11)$$

in which Q = flow across the respective cell boundaries (L^3T^{-1}); A = cross-sectional flow area (L^2); z = infiltrated volume per unit length and is equal to the product of channel losses and wetted perimeter, (L^2); δx = length of the cell (L); δt = time step size in seconds (T); θ = time averaging coefficient to account for the non-linear variation in the flow profile over time (dimensionless); ϕ = time averaging coefficient (dimensionless); k and m subscripts identifying physical parameters at time j-1 for the left and right boundaries of the cell respectively; and l and r are subscripts at the left and right cell boundaries at time j.

If bulk lateral outflow conditions exists in the control volume, the lateral outflow is a function of the flow depth in the control volume, the type of turnout structure, and its setting. In the case of a side-discharging weir, the bulk outflow can be determined by Eq. 12.

$$q=0.5C_dW(2g)^{0.5}\ [(H_{n-1})^{1.5} + (H_n)^{1.5}] \tag{12}$$

where q is the bulk lateral outflow in (L^3T^{-1}); C_d = coefficient of discharge for the side weir (dimensionless); W = width of weir crest (L); H = operating head (L); and subscript n = computational node number. The control volume including the turnout is constructed with a length equal to the width of the turnout structure. By adding the bulk outflow term in Eq. 12 to Eq. 11, we obtain:

$$\begin{aligned}&\{\theta[Q_l - Q_r - q^j]+(1-\theta)\ [Q_k - Q_m - q^{j-1}]\}\delta t/\delta x - \\ &\{\phi[A_l - A_k + Z_l - Z_k]+(1-\phi)\ [A_r - A_m + Z_r - Z_m]\} = 0\end{aligned} \tag{13}$$

in which the superscripts indicate the beginning and end of the time step.

Integration of Eq. 10 yields:

$$\begin{aligned}&\{\phi[Q_l - Q_k]+(1-\phi)[Q_r - Q_m]\}/g\delta t + \\ &\theta\{[P + Q^2/gA]_r - [P + Q^2/gA\}^l\}\delta x + \\ &(1-\theta)\ \{[P + Q^2/gA]_m - [P + Q^2/gA]_k\}/\delta x - \\ &S_0\{\theta[\phi A^l = (1-\phi)A_r] + (1-\theta)\ [\phi A_k + (1-\phi)A_m]\} + \\ &\theta[\phi D_l + (1-\phi)D_r] + (1-\theta)\ [\phi D_k + (1-\phi)D_m] = 0\end{aligned} \tag{14}$$

where Q = flow across the respective cell boundaries (L^3T^{-1}); A = cross-sectional flow area (L^2); Z = infiltrated volume per unit length (L2); D = product of area and frictional slope (L^2); P = h_cA (L^3); h_c = vertical distance from water surface to the centroid of the cross-sectional area (L); S_0 = longitudinal slope (dimensionless); and g = acceleration due to gravity in (LT^{-2}).

Downstream boundary equations: As noted in Chapter II, the control structures have a major influence on flow conditions within the reach. These control structures influence water surface profiles as well as the discharge out of the reach. The structures that are explicitly modeled include weirs, stoplogs, checks, flumes, gates, transitions, culverts, flow dividers, farm turnouts, emergency spillways, and pumping plants. For each of these structures, a stage-discharge relation determines the interaction of water control and conveyance structure hydraulics. The remaining array of structures such as, drop chutes, tunnels and pipes flowing full are not explicitly modeled but their effects on flow are incorporated in the solution by estimating the travel time in the conveyance section and the effect on upstream and downstream flow depths.

The hydraulics of control and regulating structures in the network are handled through unique stage-discharge relationships. This allows end of reach structures to function mathematically as boundary conditions and any bulk lateral outflow points to be incorporated. Six general types of structures are included in the model: (1) weir structures; (2) orifice structures; (3) transitions; (4) pumping plants; and (5) combination structures.

A thorough discussion on standard and non-standard, under and overflow, structures was presented in Chapter II. This section deals with special boundary conditions, a hydraulic jump downstream of a sluice gate, transitions, pumping plants, and combination structures.

Hydraulic jumps can occur in canal network where water flowing below critical depth enters a section in which the flow depth is above critical depth. Practical application of hydraulic jumps in irrigation canal network are: (1) dissipation of energy in water flowing through the control structure; (2) recovery of head on the downstream side of the jump; and (3) increase the discharge of a sluice gate by holding back tailwater thereby allowing free-flowing discharge (Chow, 1959). Hydraulic jumps occur where the rate of change of momentum is equal to the sum of the forces in the direction of flow. For small slopes, the sine of the slope is approximately zero and the cosine approximately equal to unity, the hydraulic jump equation becomes (Brater and King, 1976):

$$\frac{QV}{g} + A_1 h_{c_1} = \frac{QV_2}{g} + A_2 h_{c_2} \tag{15}$$

Where Q = discharge (L^3T^{-1}); V= mean velocity (LT^{-1}); A= flow cross-sectional area (L^2); h_c = depth to the center of gravity of the cross-sectional area (L); and the subscripts 1 and 2 demote the point of analysis.

Only the hydraulic jump below the regulating sluice gate in a mild channel is considered in this study for analyzing the transition from free to submerged flow conditions. Figure 16(a) shows the location of a hydraulic jump on a mild channel. The locaation, length, energy loss and sequent depth of the hydraulic jump are influenced by the tailwater regime, the supercritical condition, and the stilling basin. The location of the jump determines the prevailing conditions, free flow or submerged flow (Fig. 16(b)).

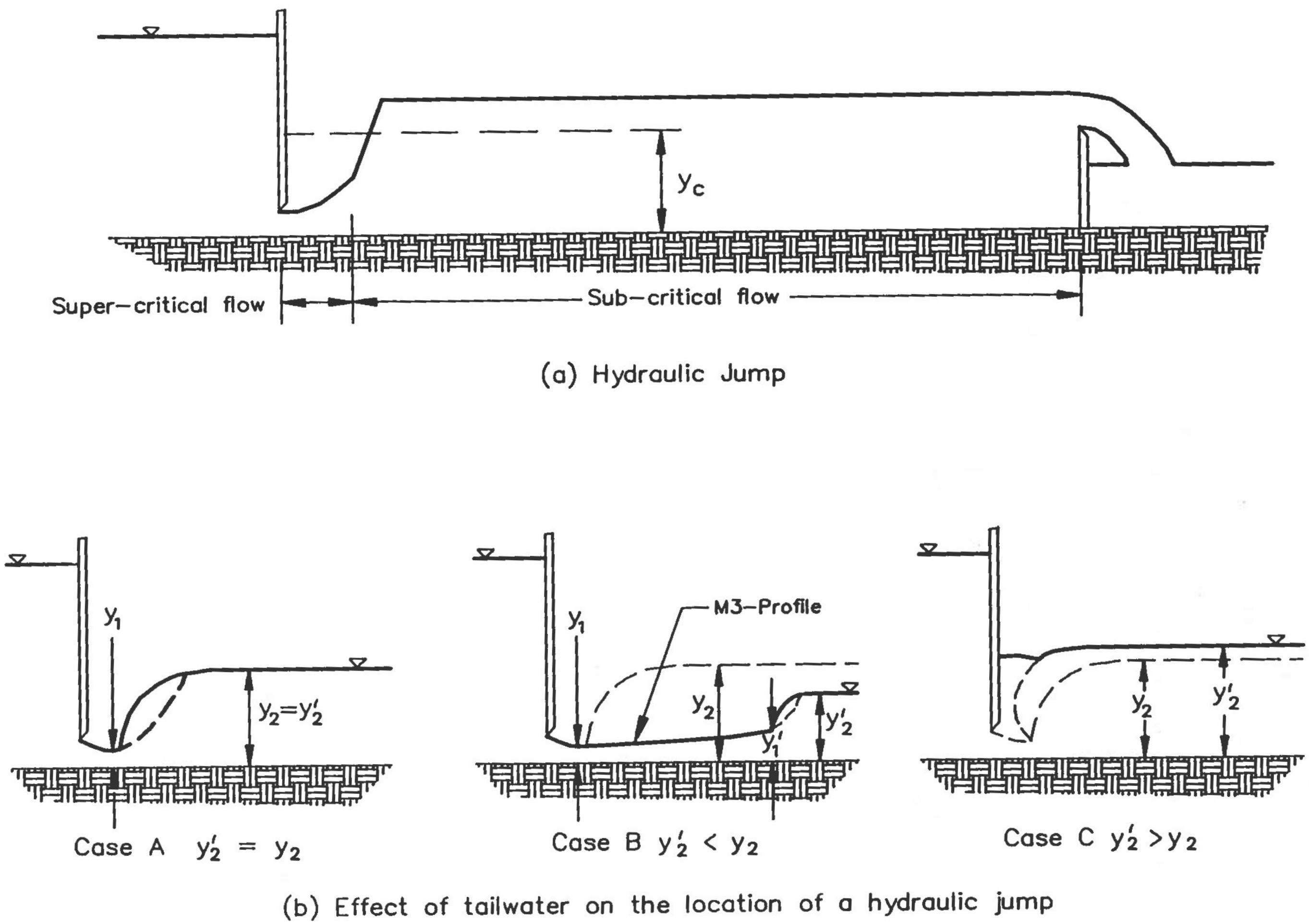

Figure 16. Hydraulic Jump Below a Sluice Gate (Chow, 1959)

In analyzing flow across a sluice gate, the flow depth y_1, which is a function of the gate opening and vena contracta is known and therefore its sequent depth y_2 can be calculated using Eq. 15. If the tailwater depth y_2 is equal to the conjugate depth y_2, a hydraulic jump occurs as shown on Fig. 16(b) and free flow conditions exist. When the tailwater depth is less than the conjugate depth, a hydraulic jump does not occur until the flow depth of the supercritical zone (M3 flow profile) equals to y_1, the conjugate depth of the downstream flow depth y_2. Submerged flow conditions exist when the tailwater flow depth y_2 is greater that the conjugate depth y_2 because the location of the jump is forced upstream to a point where it is drowned out at the source (Chow, 1959).

Where the canal cross-section changes, a transition is used to provided a smoother water flow and reduce energy loss by producing gradually accelerating velocities in inlet transitions and a gradually decelerating velocity in outlet transition. The equations governing flow across these structures are based on the principle of conservation of energy. In open channel flow, specific energy at any point can be computed using Eq. 16.

$$E = Y + \frac{Q^2}{2gA^2} \tag{16}$$

where E = specific energy (L) and Y = flow depth (L). By taking one point upstream and the other at the downstream end of the transition, the change in specific energy between the two points can expressed as:

$$h_l = E_1 - E_2 \tag{17}$$

in which h_l = head or friction loss (L) and the subscripts 1 and 2 identify parameters defined at the upstream and downstream points respectively. By assuming that the friction loss across the transition is five percent of the energy at the upstream point, Eq. 6 can be rewritten as:

$$0.95 \left[Y_1 + \frac{Q^2}{2g(A_1)^2}\right] = Y_2 + \frac{Q^2}{2g(A_2)^2} \tag{18}$$

solving for Q we obtain

$$Q = \sqrt{\frac{2g(Y_2 - 0.95Y_1)}{\frac{0.95}{(A_1)^2} - \frac{1.0}{(A_2)^2}}} \tag{19}$$

Pumping plants are used to withdraw water from a canal reach for the purpose of raising water to a higher elevation and may be located along the reach or at the end of the reach. The governing stage-discharge relationship becomes:

$$Q = \alpha(Y - H_s)^{\beta} \tag{20}$$

where h_s = the flow depth (L) below which the pump is turned off; and α and β are again fitting coefficients. When the pump has a constant discharge, β = 0.

When there is more than one structure controlling the water leaving the reach, it is necessary to define a stage-discharge equation that takes into account the existing combination of structures (Fig. 17). The equation of individual structures can be added together such as when a combination of a sluice gate and a weir exists.

$$Q = C_{d1}A[2g(h_u - h_d)]^{0.5} + C_{d2}(2g)^{0.5}W(H_2)^{1.5} \tag{21}$$

where the subscripts 1 and 2 identify parameters for the two structures.

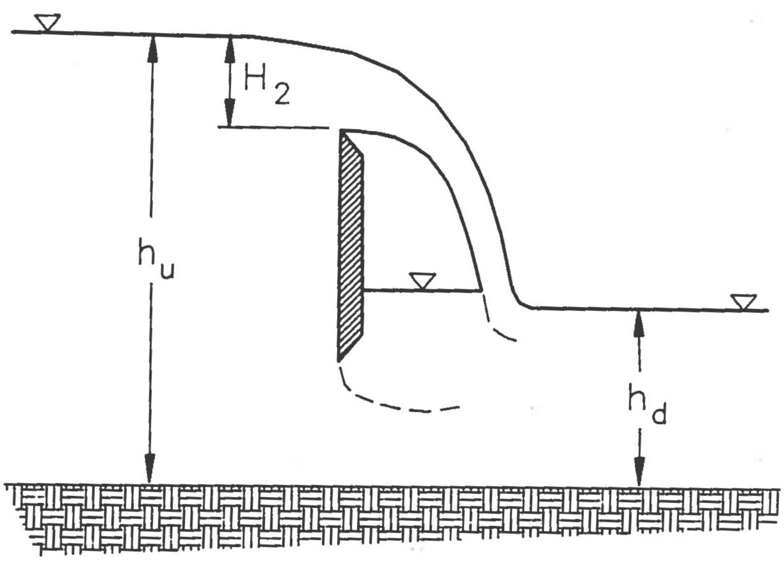

Figure 17. Combination of Flow Control Structures

Initial Conditions

The model has three initial conditions options, empty canal, previously computed hydraulic status, and user-specified steady state conditions.

Empty canal filling: When water is introduced into a dry canal, the advancing front analysis is equivalent to the advance phase of furrow irrigation although occurring more rapidly because of higher flow rate, less resistance and seepage loss. Simulation of this phase is, therefore, similar to that presented by (Walker and Skogerboe, 1987).

Previously computed hydraulic status: The model computed the flow profile on a five minute time step. Consequently, in a large canal network it is necessary to split the irrigation season into 12 hour time blocks. Also, canal operating staff are normally interested in current flow status and 12 hours before and after. The model saves the hydraulic status at the end of each simulation run which becomes the initial conditions for the next simulation period.

Steady State Start-up: It is sometimes necessary to start the simulation with a predetermined steady state condition. In this case the computations for the gradually varied flow profile proceed from the downstream end to the upstream boundary of each reach and from the tail end to the headwork. To determine the backwater flow profile, the flow conditions at the downstream boundary point should be specified.

Consider the case of a free-flowing orifice structure with a flow equation:

$$Q = C_d bW \sqrt{2g(y - \frac{b}{2})} \tag{22}$$

where C_d = coefficient of discharge; b = orifice opening (L); W = orifice width (L); and y = flow depth (L). The variables C_d and W are characteristics of the control structure and will therefore be in the configuration data file. The remaining variables, Q, y, and b are time dependent and will therefore be used to determine the initial steady state condition. A combination of any two of these time dependent variables is sufficient to specify the initial condition at the downstream boundary condition (Q and y, or Q and b, or y and b). The steady state start-up adapted for this model requires that Q and y be specified for the tail-end reach and only for the other reaches. The end of reach outflow is determined by evaluating the inflow into the downstream reach.

The equation governing the gradually varied flow in a prismatic channel and one which includes terms for flow entering or leaving the channel in the x direction is (Jeppson, 1986):

$$\frac{dy}{dx} = \frac{S_0 - S_f - \dfrac{Qq^*}{gA^2} - F_q}{1 - F^2} \tag{23}$$

in which q^* = the lateral flow per unit length (L^2T^{-1}); $F_q = 0$ for bulk lateral outflow; $F_q = Qq^*/(2gA^2)$ for seepage outflow; and $F_q = (v - u)q^*/(gA)$ for bulk inflow; v = velocity in the main channel (LT^{-1}); and u = component of velocity in the direction of the main channel flow (LT^{-1}). Other variables are as previously defined.

The method used to solve this equation depends on whether y or x is the unknown. In this case, the objective is to determine the flow depth at specified intervals taking into consideration the location of the turnouts. The solution begins at the downstream end where Q and y_0 are known and continues to the upstream end of the reach. The method used to solve the above first order ordinary differential equation is divided into two steps: (1) starting the solution, and (2) continuing the solution (Flammer, Jeppson and Keedy, 1982). The Euler's predictor and corrector method is used to start the solution. In this method, an estimate of Y_1 at a distance $x_0 + \Delta x$ is obtained from the equation,

$$y_1 = y_0 + \Delta x \left(\frac{dy}{dx}\right) \tag{24}$$

A better estimate of the slope (dy/dx) is the average of the slope evaluated at the two ends. Thus, the Euler corrector equation is:

$$y_1 = y_0 + \frac{\Delta x}{2}\left[\left(\frac{dy}{dx}\right)\Big|_{x=x_0} + \left(\frac{dy}{dx}\right)\Big|_{x=x_0+\Delta x}\right] \tag{25}$$

This equation is repeated until the change in consecutive values of y_1, is within an acceptable range. This procedure is repeated until there are sufficient known values of y for the continuing method. The Milne's method is used for continuing the solution. The predictor equation for this method is:

$$y_{i+1} = y_{i-3} + \frac{4}{3} \Delta x \left[2 \left.\frac{dy}{dx}\right|_{i-2} - \left.\frac{dy}{dx}\right|_{i-1} + 2 \left.\frac{dy}{dx}\right|_{i} \right] \tag{26}$$

in which the subscripts denote the computation node numbers where the term is to be evaluated. The corrector method's equation is,

$$y_{i+1} = y_{i-1} + \frac{\Delta x}{3} \left[\left.\frac{dy}{dx}\right|_{i-1} + 4 \left.\frac{dy}{dx}\right|_{i} + \left.\frac{dy}{dx}\right|_{i+1} \right] \tag{27}$$

Numerical Solution

When simulating canal filling, the solution begins when water is introduced into a dry reach, proceeds though the canal filling phase introducing a control volume for each time step and continues through the operation phase with a fixed number of control volumes. At any time j, the flow status variables Q and A are known at time j-1 and the infiltration is assumed to be a known function of the wetted perimeter and is therefore known for all computational nodes at all times. The upstream boundary condition is specified by the inflow hydrograph. For each time step, the unknowns are A at the upstream boundary, two unknowns (A, Q) for intermediate computational nodes, and at the downstream node are x (since A, and Q are equal to zero) during the filling phase or A and Q during the post-filling phase. The momentum and continuity equation (Eqs. 13 and 14) can be written for each control volume, providing 2N equations for a reach with N control volumes and 2N unknowns for the filling phase and 2N+1 unknowns for the post-filling phase. The downstream boundary condition provides an additional equation so as to balance the number of unknowns with the number of equations for an implicit solution technique (Strelkoff, 1970; Walker and Skogerboe, 1987).

By applying the Newton-Raphson method, the system of non-linear algebraic equations is solved by first transforming it to a linear system which can be written in a matrix notation such as $A\bar{x} = \bar{b}$. The linearized equations for each control volume take the form (for more details see Walker and Skogerboe, 1987):

$$A\delta A_l + B\delta Q_l + C\delta A_r + D\delta Q_r = - P \tag{28}$$

$$E\delta A_l + F\delta Q_l + G\delta A_r + H\delta Q_r = - R \tag{29}$$

Where P = residual of the continuity equation; R = residual of the momentum equation; δA and δQ are elements of the solution vector; the continuity equation derivatives are:

$$A = \frac{dP}{d\bar{A}_l}; \quad B = \frac{dP}{d\bar{Q}_l}; \quad C = \frac{dP}{d\bar{A}_r}; \quad D = \frac{dP}{d\bar{Q}_r}; \text{ and}$$

the momentum equation derivatives are:

$$E = \frac{dR}{d\bar{A}_l}; \quad F = \frac{dR}{d\bar{Q}_l}; \quad G = \frac{dR}{d\bar{A}_r}; \text{ and } \quad H = \frac{dR}{d\bar{Q}_r}$$

Because the resulting matrix is banded it can be efficiently solved using the Preissmann Double Sweep Algorithm. This algorithm is based on assuming a linear relationship between the discharge and area variables (Eq. 30).

$$\delta Q_{m-1} = S_m \delta A_{m-1} + T_m \qquad (1 \leq m \leq N) \tag{30}$$

where

$$S_{m+1} = \frac{U_m C_m - G_m}{H_m - U_m D_m} \tag{31}$$

$$T_{m+1} = \frac{R_m - F_m T_m - U_m (P_m - B_m T_m)}{H_m - U_m D_m} \tag{32}$$

$$U_m = \frac{E_m + F_m S_m}{A_m + B_m S_m} \tag{33}$$

The values of S and T are calculated using the above recursive equations in the first sweep which allows the computation of the solution vector in the second sweep using Eqs. 30 and 34.

$$\delta A_{m-1} = \frac{-P_m - B_m F_m - C_m \delta A_m - D_m \delta Q_m}{A_m + B_m S_m} \qquad (1 \leq m \leq N) \tag{34}$$

Canal Filling Phase

During this phase, the number of unknowns is equal to the number of equations since $Q_N = 0$, $A_N = 0$ and the unknown at the downstream node is the incremental advance distance δX_N. By letting $\delta\delta$ be the change in incremental distance and assuming a linear relationship between the incremental distance and the flow cross-sectional area of the advancing tip (A_N), we obtain $\delta\delta = T_{N+1}$ since δA_N equals zero. Consequently, the second sweep begins by the evaluation of δA_{N-1} using Eq. 35

$$A_{m-1} = \frac{-P_m - B_m F_m - C_m A_m - D_m}{A_m + B_m S_m} \qquad (m = N) \tag{35}$$

The system of equations is shown in Table 3 in the form of a matrix.

TABLE 3. Solution Matrix for Predetermined Control Structure.

$$\begin{bmatrix}
-S_1 & 1 & & & & & & & & \\
A_1 & B_1 & C_1 & D_1 & & & & & & \\
E_1 & F_1 & G_1 & H_1 & & & & & & \\
 & & A_2 & B_2 & C_2 & D_2 & & & & \\
 & & E_2 & F_2 & G_2 & H_2 & & & & \\
 & & & & & \ddots & & & & \\
 & & & & A_{N-1} & B_{N-1} & C_{N-1} & D_{N-1} & & \\
 & & & & E_{N-1} & F_{N-1} & G_{N-1} & H_{N-1} & & \\
 & & & & & & A_N & B_N & C_N & D_N \\
 & & & & & & E_N & F_N & G_N & H_N \\
 & & & & & & & & -S_{N+1} & 1
\end{bmatrix}
\begin{bmatrix}
\delta A_0 \\ \delta Q_0 \\ \delta A_1 \\ \delta Q_1 \\ \delta A_2 \\ \vdots \\ \delta Q_{N-2} \\ \delta A_{N-1} \\ \delta Q_{N-1} \\ \delta A_N \\ \delta\delta
\end{bmatrix}
=
\begin{bmatrix}
T_1 \\ -P_1 \\ -R_1 \\ -P_2 \\ -R_2 \\ \vdots \\ -P_{N-1} \\ -R_{N-1} \\ -P_N \\ -R_N \\ T_{N+1}
\end{bmatrix}$$

Post-Filling Phase

Predetermined Control Structure Setting. The post-filling phase downstream boundary condition is specified by the stage-discharge relationship of the hydraulic structure or structures existing there. The solution matrix is as shown in Table 4.

TABLE 4. Solution Matrix for Predetermined Control Structure Settings.

$$
\begin{bmatrix}
-S_1 & 1 & & & & & & & & & & & \\
A_1 & B_1 & C_1 & D_1 & & & & & & & & & \\
E_1 & F_1 & G_1 & H_1 & & & & & & & & & \\
 & & A_2 & B_2 & C_2 & D_2 & & & & & & & \\
 & & E_2 & F_2 & G_2 & H_2 & & & & & & & \\
 & & & & & & \ddots & & & & & & \\
 & & & & & & & A_{N-1} & B_{N-1} & C_{N-1} & D_{N-1} & & \\
 & & & & & & & E_{N-1} & F_{N-1} & G_{N-1} & H_{N-1} & & \\
 & & & & & & & & & A_N & B_N & C_N & D_N \\
 & & & & & & & & & E_N & F_N & G_N & H_N \\
 & & & & & & & & & & & -S_{N+1} & 1
\end{bmatrix}
\begin{bmatrix}
\delta A_0 \\ \delta Q_0 \\ \delta A_1 \\ \delta Q_1 \\ \delta A_2 \\ \vdots \\ \delta Q_{N-2} \\ \delta A_{N-1} \\ \delta Q_{N-1} \\ \delta A_N \\ \delta Q_N
\end{bmatrix}
=
\begin{bmatrix}
T_1 \\ -P_1 \\ -R_1 \\ -P_2 \\ -R_2 \\ \vdots \\ -P_{N-1} \\ -R_{N-1} \\ -P_N \\ -R_N \\ T_{N+1}
\end{bmatrix}
$$

The last row of the matrix calls for a relation between δQ and δA because as described above previously the Preissman Doublesweep Algorithm is based on assuming a linear relation between the discharge and the area. Since the hydraulic structure equations are normally nonlinear stage-discharge relationships, there is a need to satisfy the requirements of the double sweep algorithm with a linearized discharge-area relation:

$$\frac{dQ}{dA} = \frac{dQ}{dY}\frac{dY}{dA} \tag{36}$$

where dQ/dY depends on the structure type and geometry and dY/dA depends on the canal geometry. As an example, consider the case of an overflow structure with the stage discharge relation described by Eq. 1. Differentiating with respect to Y yields:

$$\frac{dQ}{dY} = 1.5C_dW[2g(Y - h_s)]^{0.5} \tag{37}$$

where Y = flow depth (L); and h_s = sill height (L).

The cross-section of the canal can generally be fitted to a simple power functions of the type $Y = \alpha A^{\beta}$ which can be differentiated as follows:

$$\frac{dY}{dA} = \alpha\beta A^{\beta-1} \tag{38}$$

Thus, combining Eqs. 37 and 38 into the relation given by Eq. 36 gives

$$\frac{dQ}{dY} = 1.5C_dW[2g(Y - h_s)]^{0.5}\ [\alpha\beta A_N{}^{\beta-1}] \tag{39}$$

From the Preissman Doublesweep algorithm, $\delta Q_N = S_{N+\delta}1A_N + T_{N+1}$, dividing this equation by δA_N, yields:

$$\frac{\delta Q_N}{\delta A_N} = S_{N+1} + \frac{T_{N+1}}{\delta A_N} \tag{40}$$

in which $\delta Q_N/\delta A_N$ can be approximated by Eq. 36. The Preissman forward sweep calculates the values of T and S. Thus, Eq. 40 can be solved for δA_N as follows:

$$\delta A_N = \frac{T_{N+1}}{\frac{dQ}{dA} - S_{N+1}} \tag{41}$$

for the first step of the backwards sweep. Other structures and combinations thereof can be analyzed in a similar manner.

<u>Predetermined upstream flow depth</u>. The solution matrix for the predetermined upstream flow depth is the same as the one above, except for the last row of the matrix because C_N, G_N, and δA_N are all zero because A_N is known. Therefore, $\delta Q_N = T_{N+1}$ provides the starting values for the Preissman backwards sweep.

Predetermined downstream discharge. Because Q_N is known, $D_N = H_N = \delta Q_N = 0$, and because D_N and H_N are zero the matrix should be rearranged to avoid division by zero in the Preissman solution technique. The only rearrangement necessary is the last two columns and two rows of the coefficient matrix. This is accomplished by setting $D_N = C_N$ and $H_N = G_N$. Hence $\delta A_N = T_{N+1}$. The matrix solution is shown in Table 5.

TABLE 5. Solution Matrix for Predetermined Downstream Discharge.

$$
\begin{bmatrix}
-S_1 & 1 & & & & & & & \\
A_1 & B_1 & C_1 & D_1 & & & & & \\
E_1 & F_1 & G_1 & H_1 & & & & & \\
 & & A_2 & B_2 & C_2 & D_2 & & & \\
 & & E_2 & F_2 & G_2 & H_2 & & & \\
 & & & & \ddots & & & & \\
 & & & & A_{N-1} & B_{N-1} & C_{N-1} & D_{N-1} & \\
 & & & & E_{N-1} & F_{N-1} & G_{N-1} & H_{N-1} & \\
 & & & & & A_N & B_N & C_N & D_N \\
 & & & & & E_N & F_N & G_N & H_N \\
 & & & & & & & -S_{N+1} & 1
\end{bmatrix}
\begin{bmatrix}
\delta A_0 \\ \delta Q_0 \\ \delta A_1 \\ \delta Q_1 \\ \delta A_2 \\ \vdots \\ \delta Q_{N-2} \\ \delta A_{N-1} \\ \delta Q_{N-1} \\ \delta Q_N \\ \delta A_N
\end{bmatrix}
=
\begin{bmatrix}
T_1 \\ -P_1 \\ -R_1 \\ -P_2 \\ -R_2 \\ \vdots \\ -P_{N-1} \\ -R_{N-1} \\ -P_N \\ -R_N \\ T_{N+1}
\end{bmatrix}
$$

The solution technique described above begins with an approximation of the unknown variables (A_l, Q_l, A_r, and Q_r) for each time step. During the time step iterations, values of δA and δQ as well as δX_N (when applicable) are used to improve the solution as follows:

$$A^{n+1} = A^n + \delta A;$$

$$Q^{n+1} = Q^n + \delta Q; \text{ and}$$

$$\delta X^{n+1} = \delta X^n + \delta\delta \tag{42}$$

The iterative search continues until the preselected convergence criteria are met. The convergence criteria used for this study are:

$$\text{Abs}(\delta A) = 0.05A; \text{ and}$$

$$\text{Abs}(\delta Q) = 0.05Q \tag{43}$$

Inflow Determination

The objectives of the inflow determination routine are:

1. Determine the system's inflow and flow into each branch and reach;
2. Maintain flow rates to within non-erosive velocity limits; and
3. Ensure minimum canal water level fluctuations.

The factors taken into consideration in determining the inflow rates are: (1) channel properties; (2) flow depths; (3) seepage rates; and (4) turnout discharges.

During the initial filling, it is important to ensure that the flow velocity is less than the erosive velocity in the channel. This is determined by computing the advance velocity dx/dt and comparing it with the allowable velocity for the channel being simulated. If the flow velocity is higher than the allowable velocity, the inflow rate is adjusted by a factor based on the ratio of the two velocities.

$$f = Va/V \tag{44}$$

where f = adjusting factor; Va = allowable velocity (LT^{-1}); and V is the actual velocity (LT^{-1}).

In determining the desirable flow rate throughout the system, one starts with the downstream end as illustrated in the algorithm below. If the branch is still in the filling phase, the accumulation starts with the reach that is in its initial filling phase.

The reach inflow is determined by the Eq. 45

$$Q^t_{ij} = Q^t_{ij+1} + \alpha\sum_{k=1}^{N} Qd^t_{ijk} + Ql^t_{ij} + Qs^t_{ij} \tag{45}$$

where t = time step superscript; i = Branch subscript; j = reach subscript; k = turnout subscript; N = number of turnout; and

Q^t_{ij} = Reach inflow required to satisfy reach outflows and the change in reach storage;

Q^t_{ij+1} = Reach outflow;

Qd^t_{ijk} = Turnout demands;

Ql^t_{ij} = Channel losses (seepage and evaporation);

Qs^t_{ij} = Flow required to refill wedge storage or decrease in flow to allow for flow level to return to required equilbrium;

$$\alpha = \frac{\sum_{k=1}^{N} Qd^{t+1}_{ijk}}{\sum_{k=1}^{N} Qd^{t}_{ijk}}$$

a forecasting factor that incorporates next time step demand. The value of this factor ranges from 0.8 to 1.2

An adjusting factor is finally applied to the calculated reach inflow to dampen the fluctuations when there is a large change in demand. The required reach inflow reduces to:

$$Q^t_{ij} = Q^t_{ij+1} + (Q^t_{ij} - Q^t_{ij+1})/3 \qquad (46)$$

Control Structure Setting Determination

When evaluating the optimal control structure setting, the rate and direction of the adjustment is determined by the deviation from the target of the controlling parameter. The controlling parameter is depth when simulating upstream flow depth control and discharge while simulating downstream discharge control option. Figure 18 shows the variation of upstream flow depth with time and the corresponding control structure setting.

The model allows a deadband zone in which the water level can fluctuate before control structure adjustment is initiated, and two zones above and below the deadband. The speed of control structure movement in zone two is taken to be twice that of zone one because small deviations require small adjustments. After the flow depth has started returning to the target depth the control structure is held constant. This "anti-hunt" operation prevents over- and under-correction and subsequent instability.

Assuming a control structure speed of b_i when the flow depth is in zone i, the incremental control structure setting Db becomes:

$$\Delta b = \delta b_i \; \delta t \qquad (47)$$

where δt = time step (T).

The control structure setting (b) is evaluated as follows:

$$b^t = b^{t-1} \pm \Delta b \tag{48}$$

An adjusting factor is finally applied to the calculated control structure setting to dampen the fluctuations which might result when large values of Δb are determined. The control structure setting to be reduces to:

$$b^t = b^{t-1} + (b^t - b^{t-1})/3 \tag{49}$$

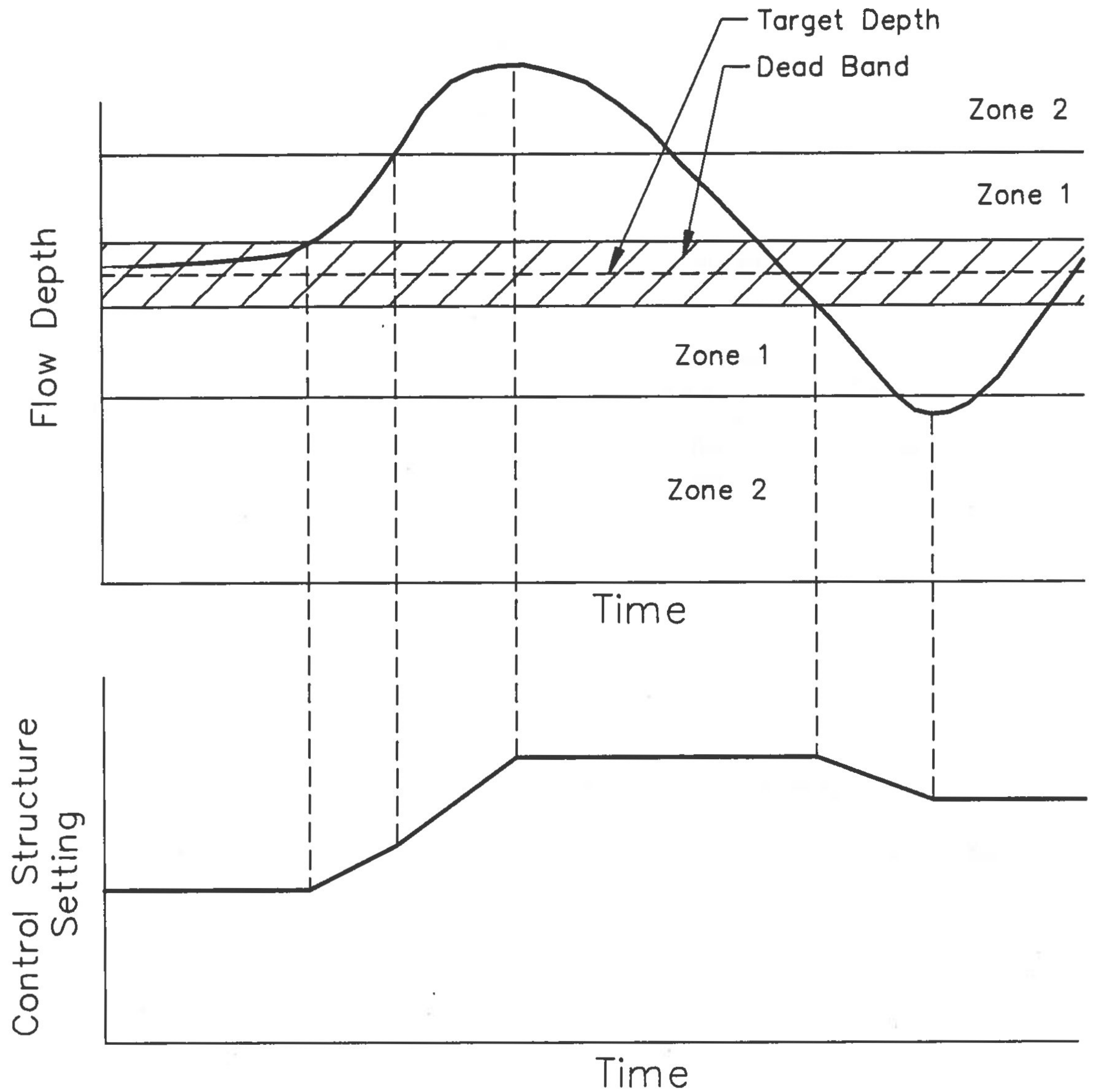

Figure 18. Flow Depth Fluctuation and Control Structure Setting

Program Coding

After the formulation of the model, the next phase was the program coding. In coding the program, the primary objective was to meet the potential users' hardware and software limitations. In the initial phase of the model development, the author conducted interviews with potential users' and participated in brainstorming sessions organized by the Water Management Synthesis Project. These discussions produced general guidelines on preliminary program capabilities and user-interface structure.

Figure 19 shows the conceptual flow chart of the CAHSM model's interactive simulation mode. After starting the program, the first thing is to setup the model by specifying the model parameters, and read various data files that the program requires. The second step involves reading the configuration data file for the project to be simulated. The next step is specification of the operational plan. This involves the selection of the operation mode and entry of the appropriate data. After the configuration and operational data are entered, they are tested for compatibility and integrity to eliminate any inconsistency. If the data are found to be compatible, simulation starts.

For each simulation time step, the flow characteristics at all the computational nodes in the network are calculated and those of selected canal reaches are displayed on the screen. The user then monitors the simulation and evaluates whether the simulation closely approximates his/her predetermined objectives. If there is a significant deviation, the user can interrupt simulation and modify the operational plan so as to minimize the deviation. This process continues until the end of the simulation. At this point the user assess the overall objective and repeats the simulation if the performance is not acceptable.

The author designed and implemented the technical algorithm and input/output routines based on the flow chart presented in Figure 19. This was followed by the extensive testing of the technical algorithms, read/write routines, and utility routines for numerical accuracy, numerical stability, and easy of use. The program is written in FORTRAN 77. It is modular in structure and consists of 120 subroutines most of them devoted to improved user interface.

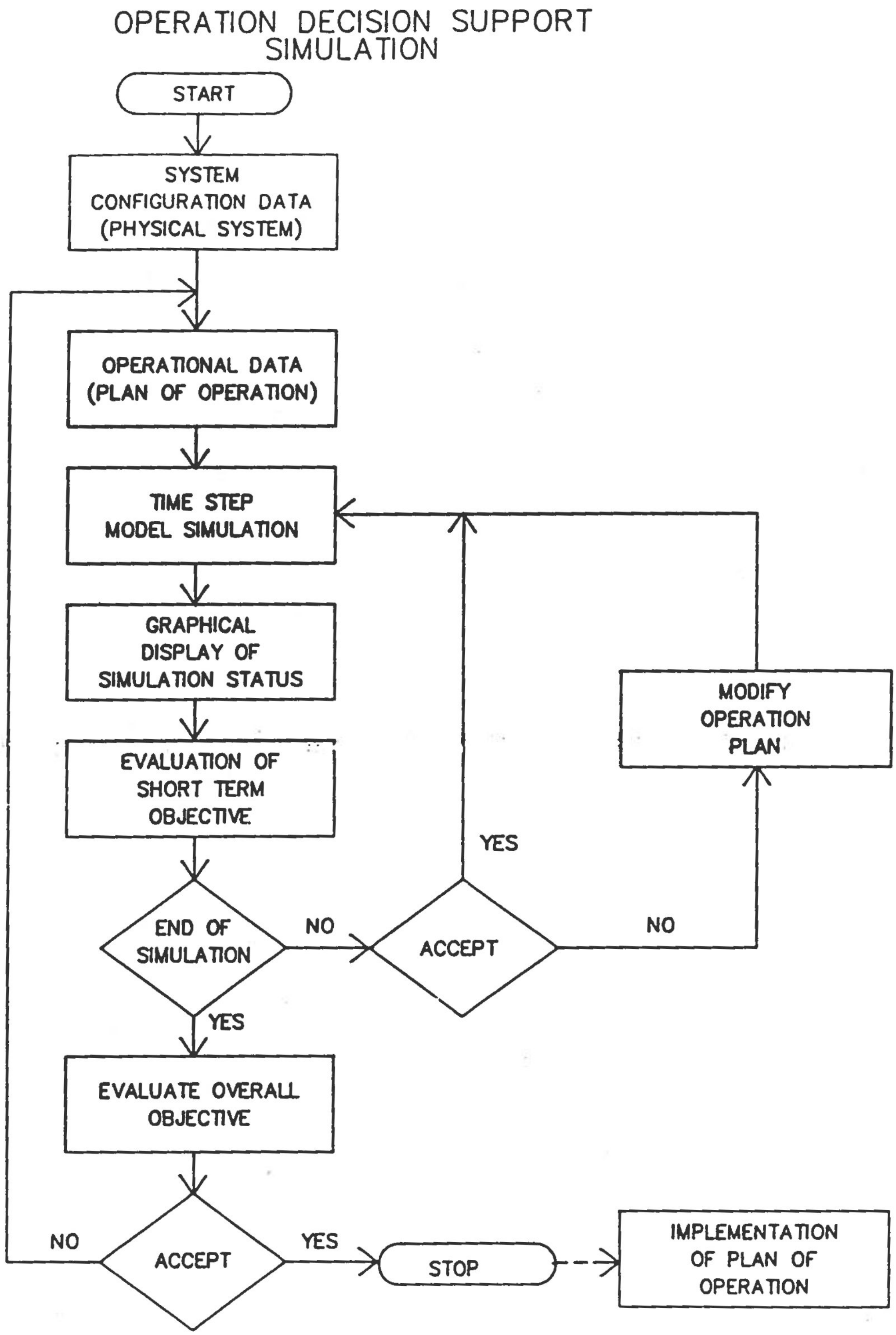

Figure 19. The Conceptual Flow Chart of CAHSM Model's Interactive Simulation Mode.

Simulation Process

The main program serves as the control center. It is from here that the control is transferred to the appropriate modules depending on the user's choice. The major parts of the simulation process are as follows:

1. Read configuration and operational data files;
2. Initialize the program constants;
3. Calculate the flow geometry parameters;
4. Select and execute initial conditions with three options: (a) canal completely empty, (b) steady state start up condition, and (c) resume simulation using previously stored simulation status;

5. Computation of the flow characteristics

```
Time loop
     If program is in scheduling mode compute required flows
     at the end of each reach and the branch inflow

     Branch loop
          Reach loop
            Compute flow profile
          Next reach
     Next branch
Next time step
```

Management of Simulation Activities

The Subroutine MANAGER manages simulation activities which are grouped into six categories:

1. Display output of present simulation status;
2. Display elapsed time;
3. Check for user interrupt;
4. Check end of data;
5. Calculate headwork inflow if the depth or discharge control options are selected; and
6. Branch loop
    a) determine branch inflow
    b) update previous flow status
    c) calculate flow profile reach by reach
    d) display output.

Subroutine MANAGER starts off by establishing the program constants and calculating reach geometry parameters and design discharge. For a first time simulation, the model simulates the initial tip cell before starting on the time loop. Simulation progress through time at 5 minute intervals for 12 hours unless simulation is interrupted.

Computation of Steady State Start-up Condition

The first step in computing the steady state start-up condition is to layout the computational grid. In doing so the choice of the length of the control volume is constrained by the location of the turnouts and by the maximum number of computational nodes in each reach (in this case it is limited to 20). This is accomplished by Subroutine SetGRID. A detail discussion of the procedure is presented under the grid management section. The next step is to determine the flow rate at each computational node. This process starts at the downstream end of the system and works upwards. The flow at each computational node is equal to the sum of all the seepage and turnout discharges below it. This is accomplished by two subroutines. Subroutine FlowRATE computes the flow rate at the downstream end of each reach and at the branch inlet, while Subroutine NodeFLOW computes the flow at the computational node. The computation of the gradually varied flow profile using Eq. 23 is accomplished by subroutines GVFlowP and DERVdydx.

Flow Profile Computation

The initial control volume of each reach is obtained by solving for the flow cross-sectional area at the inlet and the length of the control volume using Subroutine HYDTIP. Subroutine INFILT computes infiltrated volume per unit length based on a wetted perimeter dependent channel loss equation.

Subroutine HYDROD is the heart of the flow profile computation and handles the advance phase after the initial tip cell is calculated and the post-advance phase. Computation of the flow profile is accomplished after several iterations. The actual number depends on the changes that occurred during the time step. During the advance phase Subroutine TIP2 computes the coefficient for the solution matrix for the tip cell control volume while Subroutine COEINC computes the coefficient for the other control volumes. Computation of the coefficients of the solution matrix require computation of pressure and drag terms and their derivatives with respect to discharge and flow cross-sectional area using the subroutines PRESSUR and DRAG, respectively. After the coefficient matrix and the right hand side vector are evaluated, the subroutine DBLSWEP is called upon to solve the system of equations. For each iteration, Subroutine CONTROL calculates the flow leaving the reach at the end of the downstream end and via the turnouts and Subroutine NEWTON is the called upon to: (1) add the correction terms to the variables; and (2) evaluate whether the convergence criteria has been met. If the criteria is met, the flow profile computation is completed for the reach.

Subroutines GEOMET, HYDTIP, HYDROD, TIP2, COEINC, PRESSUR, DRAG, and DBLSWEEP are copyrighted sub-programs by the Utah State University Foundation and have been modified and used by permission.

The transition between the advance and post-advance phase is handled by Subroutine ENDCELL. This subroutine is called at each time step during the advance phase to incorporate the new control volume if the incremental advance distance is greater than 1 meters. If the advance distance exceeds the length of the reach, the subroutine adjusts the control volume by superimposing the volume of water in the control volume below the structure on the last control volume of the reach. The adjustment depends on the flow conditions.

<u>Blocked downstream conditions</u>: This conditions occurs when the flow depth at the control structure is such that no flow leaves the reach (for example, when the gate is closed or the flow depth is less that the sill height). Two steps are followed in adjusting the control volume as follows: First a triangular shaped control volume is added to the flow profile to take into consideration the effect of the control structure in place. The psuedocode for this operation is:

$$A_{N-2} = A_{N-2} + (\delta X_{N-1}+\delta X_N)*A_N/Xo \quad \text{if } (\delta X_{N-1} + \delta X_N) < Xo \text{ ;}$$

$$A_{N-1} = A_{N-1} + \delta X_{N-1}*A_N/Xo \quad \text{if } \delta X_{N-1} < Xo \text{ ; and}$$

$$A_N = (1 + 0.8)*A_N \tag{50}$$

The second step is to adjust the length of the control volume upstream of the structure such that $X_{N-1} = X_N$ to avoid having a very small cell at the downstream end.

<u>Weir conditions</u>: The algorithm for adjusting the flow cross-sectional area for this case is the same as that described for blocked end conditions except for the value of f. In this case f is defined as:

$$f = 0.8 * A_w/A_n \tag{51}$$

where Aw is the flow cross-sectional area for a flow depth equal to the sill height, and dx_{N+1} is the length of the control volume for the downstream reach is it exists.

<u>Open gate boundary condition</u>: In this case, there are three possible flow depths conditions: (1) the downstream flow depth can be above the gate opening; (2) the flow depth can be below the maximum gate opening; and (3) the flow depth can be below the sill height on the gate.

In case 1, f is computed as follows

$$f = 0.8 * (A_n - G_a)/A_n \tag{52}$$

where G_a = gate opening area (L^2) and A_n = flow cross-sectional area (L^2). In case 2, f is computed in the same way as the weir boundary condition. Case 3 is the same as the blocked end boundary condition.

Improving Initial Solutions

The program's numerical solution is based on the Newton-Raphson technique which is very sensitive to initial solution for such a system of equations. Thus, good initial solutions are required for continued convergence to the final solution. This is well taken care of for normal canal operation, but there are problems when the user specifies radical changes in flow rate or control structure settings. In these cases the programmed initial solutions may not be sufficiently close to the final solution and the program may fail to converge. To alleviate this problem, two subroutines have been incorporated to provide improved starting points. Subroutine TRY1AGAIN is called when the program fails to converge to a solution during the advance phase. The major cause of this failure is when the incremental advance distance is too high and the inflow reduces drastically. To alleviate this problem, the following steps are taken:

1. Simulation is divided into two substeps of one-half the normal time step; and
2. An improved initial solution attempted.

Subroutine TRY2AGAIN is called when the program fails to converge to a solution during the post-advance phase. The major cause of this failure is when there is a radical change in control or turnout structure setting or in the inflow. These problems can also be alleviated by introducing intermediate computational time steps as well as gradually adjusting the control and turnout structure settings.

Grid Management

To reduce computational time and the program memory requirements to store flow parameters at each computational node, a grid management procedure is adapted. The initial node locations are defined at the successive points where water advances to during the filling process. If the inflow into the reach during the advance phase is low and/or the length of the reach is long, the water advance may be so slow that many computational nodes are introduced in a canal reach. This slows the computation process with no appreciable improvements in numerical stability or accuracy. By reducing the computational nodes, the simulation time can be reduced. The Subroutine THINNER reduces the number of computational nodes when there are more than 10 during the advance phase and the flow is a substantial distance from the end of the reach. Note that the last two computational nodes are not adjusted but are left intact, while changes are made in the others where the flow conditions are more stable.

A second grid management scenario incorporates the turnout control volumes within the reach. To minimize the number of control volumes, three strategies are applied.

1. Relocation of turnouts when they are located too close to the upstream or downstream end of the reach to avoid having control volumes that are too small;
2. Readjusting the length of the control volume when the turnout is located too close to its upper or lower computational node; and
3. A turnout control volume is incorporated between two computational nodes.

The third strategy introduces two additional control volumes for each turnout, whereas the other strategies introduce only one.

CHAPTER IV

THE COMPUTER MODEL OVERVIEW

The resulting model, Canal Hydraulics Simulation Model (CAHSM), provides the user with sufficient flexibility in the application of the algorithm to simulate the physical and operational characteristics of an irrigation conveyance and distribution system. The mathematical model can be used to determine flow rates and flow cross-sectional areas at all points in the canal network that result from a given physical structure and operational scenario.

Model Data Requirements

While it can be argued that highly sophisticated models require more data to run. This is not the case with the hydraulic model developed in this study. It has about the same data requirements as the "volume balance" modeling approach developed for the Sri Lanka Water Management Project (Dearth, 1985). The model however, requires a higher precision for the data so as to take advantage of the comprehensive treatment of the problem. Walker and Skogerboe (1986) noted that, "Some of this information can be collected at the project site as part of operation and maintenance programs and then monitored periodically." They concluded that computerization streamlines and redirects field data by gathering only the relevant data and reducing the time required for data analysis. Thus, less effort is needed not more. The model input data are generally that which the system operators need for efficient operation and therefore represents no added burden.

Model Setup Data

The model setup data are required to control the simulation process. The setup data base is divided into two levels. Level one consists of general simulation control data and level two contains the setup data that are required during the calibration phase of the model. All of this information is stored in a file with an extension ".SET". The outline of the setup data file structure is as follows:

1. Data directory specification
2. Project name

3 Simulation period identifier
    Beginning period date
    Number of simulation periods
4 Simulation status
    Initial conditions
    Canal operation mode
    Simulation mode
5 Model parameters
    Model choice
    Time step
    Time averaging coefficient
    Distance averaging coefficient
6 Units (metric or english)

Canal Network Configuration Data

The configuration data describe the physical facilities of the canal network to be studied. As alluded to earlier, the canal network is divided into branches and reaches for the purposes of simulation. The uppermost branch becomes Branch 1 and others are numbered consecutively to the most downstream branch. The branches are divided into reaches. The end of a reach is determined by the presence of a cross-canal structure, any change in canal cross-section, slope or roughness, or any other structure that affects the canal water surface elevation on the upstream side. This information can be extracted from a project layout map (see Fig. 14). The configuration data required are listed below and are stored in a file with an extension ".CFG". The following is an outline of the configuration data file structure.

Project description
Number of branches

Branch information (Repeats for each branch in the project)
    Branch that supplies water to this branch
    Reach that supplies water to this branch
    Turnout that supplies water to this branch
    Number of reaches in this branch

Reach information (Repeats for each reach in the branch)
    Number of turnouts
    Maximum canal depth
    Design flow depth
    Side slope
    Bottom width
    Seepage rate
    Manning's n
    Longitudinal slope
    Length of the reach
    Distance between reaches
    Change in canal invert elevation between reaches

Reach water control structure information
- Type
- Sill height
- Discharge coefficient
- Gate opening
- Width of structure

Reach turnout information (Repeat for each turnout in the reach)
- Type
- Sill height
- Discharge coefficient
- Gate opening
- Width of structure
- Location (distance from upstream end of the reach

Most of the data required by the model can be obtained from the irrigation system design documents. However for old systems, significant deterioration and modification will have occurred in the system and, therefore, field data warrants collection.

<u>Cross-sectional data</u>. The model assumes that the canal is prismatic. The cross sectional data required includes the maximum flow depth (Ymax) in m. (ft), design or normal flow depth (Yn) in m. (ft), bottom width (B) in m. (ft), and side slope (z). If the design drawings are not available these data can be obtained by field measurements. If there is a lined canal, the cross-sectional data are readily obtained. For irregular cross-sections, it is necessary to obtain an equivalent bottom width and side slope that results in an equivalent depth-area relationship.

<u>Longitudinal data</u>. The longitudinal data describe the reach longitudinal profile. The pertinent data in this category include: (1) channel losses in cm/day (inches/day), (2) longitudinal slope in m/1000m (ft/1000ft), (3) the channel roughness coefficient, (4) length of the reach in m. (ft), and (5) the characteristics of the transition between reaches.

Channel losses are a function of channel properties and flow rates. These in turn depend on (1) size of the command areas; (2) cropping patterns; (3) irrigation methods; and (4) the season of the year. Channel losses include seepage, leaks, evaporation, and canal bank vegetation transpiration losses as well as any undetected unauthorized use of water. Channel losses are the most difficult variables to evaluate with reasonable accuracy due to the high spatial variability in seepage. The methods are available for evaluating channel losses include: (a) inflow-outflow; (b) ponding; and (c) seepage meter measurement. The model user is encouraged to obtain more details on this subject in order to more effectively evaluate the channel losses. The model input is expressed in cm/day (inches/day).

Where data are available in the form of conveyance efficiency, loss in m^3/s (cfs), etc., attempts should be made to convert it to cm/day (inches/day) over the wetted perimeter. If there is significant evaporation and leaking structure loss, these values should be evaluated and incorporated in the channel loss equation.

Longitudinal slope obtained from design data should not be relied on completely due to construction deficiencies in meeting design specifications and to channel bottom deterioration as a result of scouring and silting. An average slope value should be obtained by profile surveying at several locations along the reach.

The roughness coefficient is an indicator of the channel resistance to flow. This is one of the most elusive model parameters due to its variability in space and time. The model uses the Manning's formula to establish a discharge-area relationship. Thus, the Manning's n is the parameter of interest. Most hydraulic text books present a guide to proper selection of Manning's n and a summary has been included in the on-line help system in the model.

The transition between reaches is delineated by the structure (eg. siphon, drop chute, etc.) that separates the reaches. Two parameters that describe the transition are required: (1) distance between the control structure and beginning of next reach (especially in case of an inverted siphon or drop); and (2) the change in canal invert elevation.

Control and turnout structures. The control and turnout data (sill height, structure width, maximum gate opening) can be obtained by taking direct measurement of the structure or from design drawings. Other information required to describe the structures include:

1. Downstream control structure data describing the type of structure, its stage-discharge relationship; and
2. Bulk lateral outflow structure description noting distance from the upstream end of reach to the structure itself, the type of structure, and stage discharge relationship.

Wherever bulk outflow occurs in a particular reach (at turnouts or end of a reach), a stage-discharge relation is required to determine the flow rate leaving the reach. For many standard flow control structures, a stage-discharge relationship is readily available. However with time, the structure deteriorates and a new calibration is required to determine the prevailing relationship. This is also necessary for any constrictions in the open channel and for non-standard structures.

Operational Data

The operational input represents the dynamic inputs of the system which include and depend on the mode of operation. These data are stored in a file with an extension .C01 or .D01. The two digits depict the simulation time period. The available modes of operation are as follows.

Operator specified control: Operator specified control is a option used to simulate the system under full manual control of an operator. All decisions on the flow rates and control structure settings are made by the model user. The input data are therefore:

1. An inflow hydrograph into the system; and
2. Regulating structure settings for all in-line and turnout structures for the entire simulation period.

Upstream depth control: Upstream depth control options allow the computer to simulate the regulating structure settings required to maintain a predetermined flow depth upstream of the structure and at the same time meet the demands below and from turnouts. The input data are the demand hydrographs for all turnouts in the system.

Downstream discharge control: Downstream discharge control option allows the computer to determine the flow rate and control structure settings required to satisfy a given demand to the lower reaches. The input data are the same as above.

Suspended Simulations

This set of data is used to store the simulation status at the end of a time period and provides the initial condition for the continuation of the simulation at a latter time. It is stored in a file with an extension ".STS". Below is an outline of the suspended simulation status data file structure.

Project description
Number of branches

Branch information (Repeats for each branch in the project)
- Branch that supplies water to this branch
- Reach that supplies water to this branch
- Turnout that supplies water to this branch
- Number of reaches in this branch

Reach information (Repeats for each reach in the branch)
- Number of turnouts
- Maximum canal depth
- Design flow depth
- Side slope

Bottom width
Seepage rate
Manning's n
Longitudinal slope
Length of the reach
Distance between reaches
Change in canal invert elevation between reaches

Reach water control structure information
Type
Sill height
Discharge coefficient
Gate opening
Width of structure

Reach simulation status information
Left node number
Right node number
Number of turnouts incorporated into the solution
Operation mode control status
Control structure setting
Reach inflow
Reach outflow

Reach turnout information (Repeat for each turnout in the reach)
Type
Sill height
Discharge coefficient
Gate opening
Width of structure
Location (distance from upstream end of the reach)

Reach simulation status information
Computational node to the right of this turnout
Turnout discharge
Turnout setting

Flow profile information
Flow cross-sectional area
Discharge rate
Distance of node from upstream end of the reach
Length of the computational node

Running the Program

User Interface

The resulting software is intended for use by people with minimal computer and hydraulics background. Thus, a great deal of effort was devoted to improving the user interface. Detailed evaluations and comments of a number of people are incorporated in the final work. Special attention was given to data entry, graphical displays, and interactive simulation. When using a program for the first time, data entry is extremely critical if the user is to get a favorable first impression. Therefore, no effort has been spared in developing user friendly data entry routines. The following requirements for user-friendliness were taken into consideration:

1. Understandable prompts for all communications to the user;
2. Following logical flow of data entry interaction;
3. Use of menu driven commands;
4. Provision of consistent program instructions displayed uniformly throughout the program to avoid confusion; and
5. Scanning all keyed-in information for acceptability to avoid program failure due to non-numeric data entry in place of numeric data or value range error in the model parameters.

To obviate simulation failure due to the user entering non-numeric data when numeric data are called for, data entry routines that analyze each keystroke and discard any non-numeric keystroke were developed. Another possible data input error is entering a value that is out of an acceptable range. For each data item, a minimum and maximum value are displayed and only data in this range are accepted. In an effort to make data entry, editing, and display easier and efficient, graphics have been used extensively. For each system configuration data entry prompt, a pictorial display of the requested information is presented. A plot of the time-dependent operational data (control structure setting and flow rates) is used for operational data entry. The simulation results are also displayed graphically.

The program is completely menu-driven and very simple to use. Figure 20 shows the sequence of menu pages incorporated in the program. The bottom of the screen is devoted to providing the user with operation instructions such as: (1) currently available options; (2) how to move around or enter data; and (3) display error messages.

Data Entry

System configuration data. The user is guided through menu-driven commands in the selection of data entry options. For a first time entry of system data, the user is required to identify the size of the project to be simulated in terms of the number of branches and number of reaches in each branch.

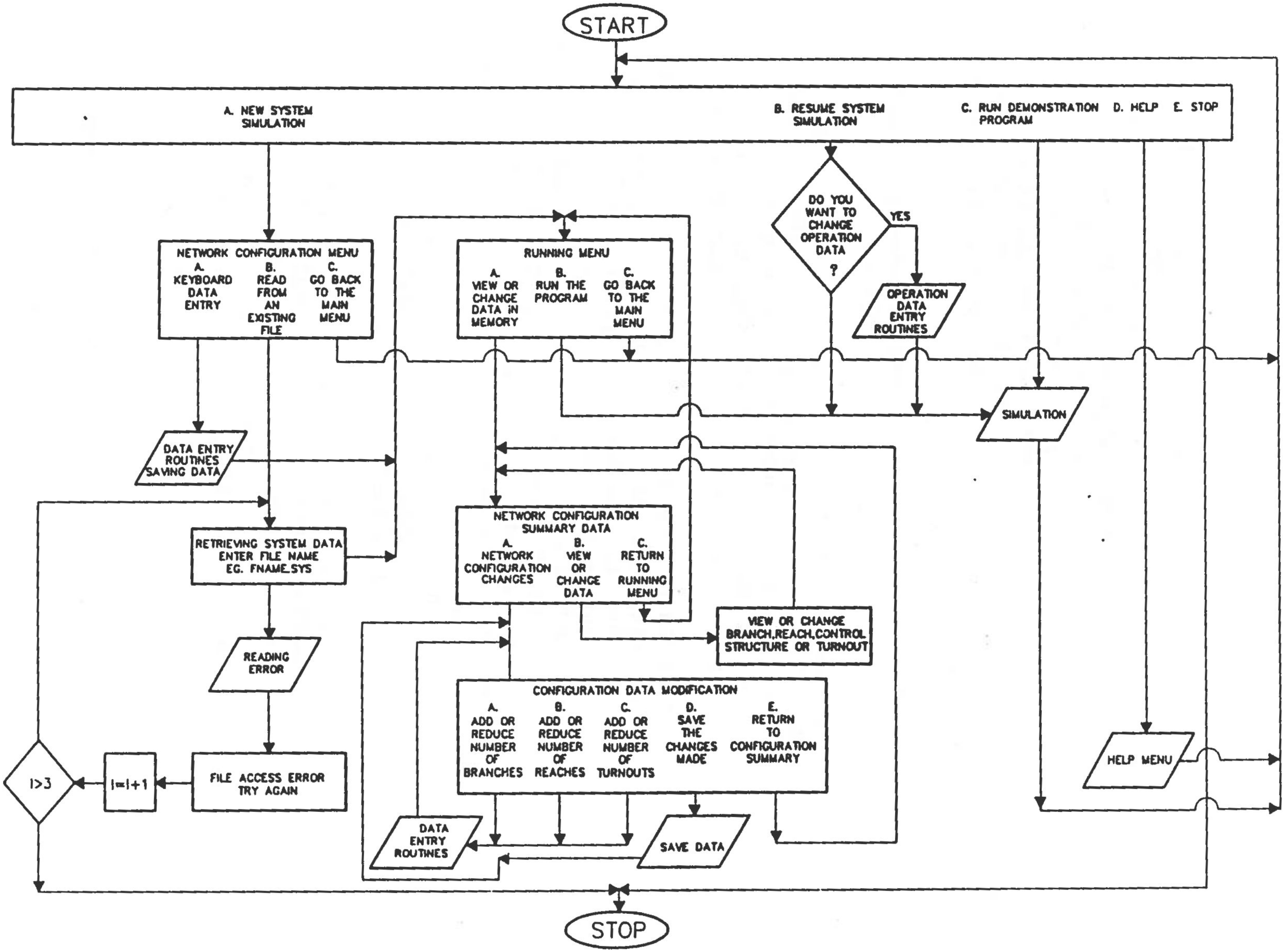

Figure 20. Sequence of Menu Pages for Canal Hydraulic Simulation Model.

It is recommended that the user start with a one branch-one reach unit and build it up unit wise, saving the data after completion of each additional reach data entry. Configuration data consist of branch and reach information that describe the physical facilities. Branch information identifies the number of branches, the source of its inflow, and the number of reaches in each branch. Reach data are divided into cross-section, longitudinal, control structure, and turnout information. Turnouts are identified by the branch and reach on which they are located and are numbered in a consecutive order from the upstream end of the reach. The location of the turnout is identified by specifying the distance from the upstream end of the reach to the center of the turnout. When two turnouts are located at the same location (left and right sides of the canal), the model requires that they be at least 3 meters apart.

For the graphical data input option, the screen is divided into two windows (Fig. 21). The upper window displays the branch, reach, and turnout to which the data pertains, the data prompt statement, the maximum and minimum values of the variable, and the value the user enters. The lower portion displays a pictorial representation of the data being entered. At the end of each class of data, the program asks the user to confirm whether the data entered are correct thus far.

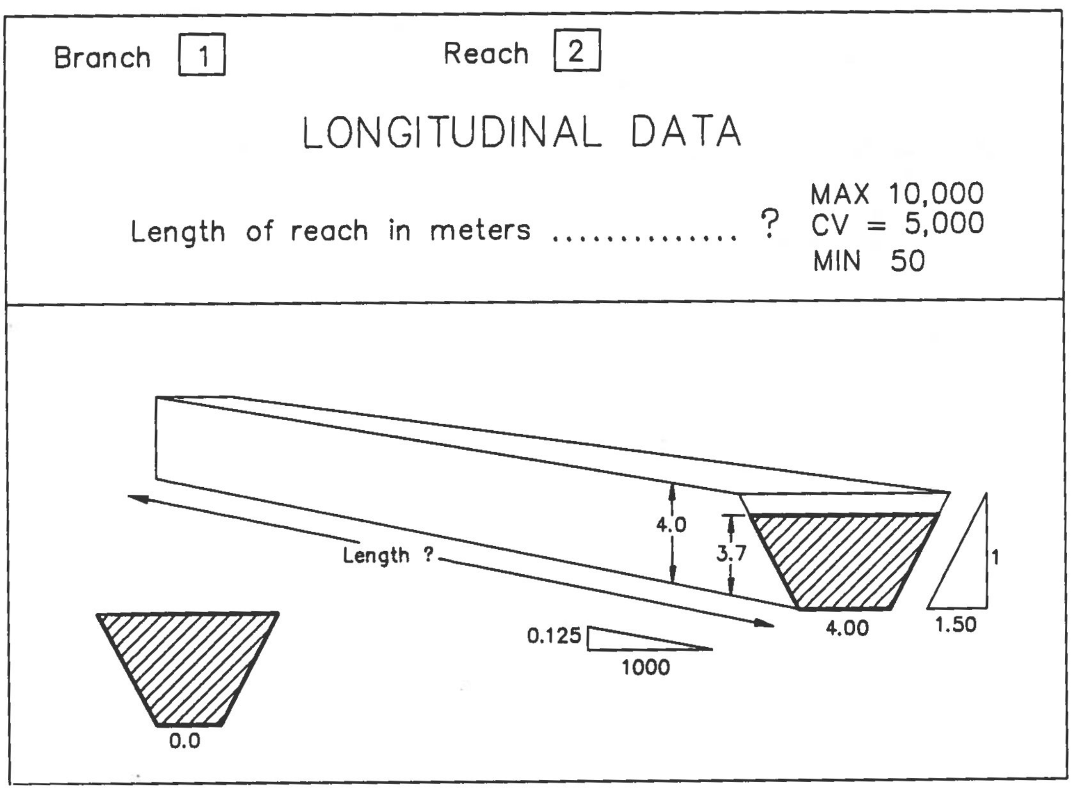

Figure 21. Typical Configuration Data Entry Screen.

If the answer is no (N), the program takes the user to the beginning of the class data entry point. If any data entry error is detected after that class of data has been confirmed to be correct, the available options are: (1) stop and start all over again; or (2) continue to the end, save the data, and then later modify the data using the VIEW OR CHANGE DATA IN MEMORY option. Users already familiar with the terminology used in canal hydraulics, can display and edit data as shown in Table 6.

TABLE 6. Tabular Data Display and Editing Screen.

REACH INFORMATION

BRANCH NUMBER	1
REACH NUMBER	1
Number of Turnouts	4
CROSSECTIONAL DATA	
Max Flow Depth in meters	2.80
Normal Flow Depth in meters	2.40
Side Slope	1.50
Bottom Width in meters	4.50
LONGITUDINAL DATA	
Channel losses in cm/day	0.00
Manning's N	0.014
Longitudinal slope in m/1000m	0.125
Length of the Reach	10000.00
Length of Structure between reaches .	0.00
Change in canal bottom elevation	0.00

CONTROL STRUCTURE INFORMATION

BRANCH NUMBER	1
REACH NUMBER	1
TYPE (Adjustable Sill Weir)	2
Minimum sill height	0.4
Discharge coefficient	1.86
Width of the structure	4.00

Operational Data. The operational data define the temporal variation in flows and structure settings as well as the management activities depending on the mode of water control selected. When operator decision control is selected, the user has to specify the inflow into the system and the control structure settings required to implement the desired objective. The flow depth and downstream discharge control options have the same data requirements and output. In this option, the operational data required are the demand hydrographs at all the off-take points. Here again the screen is divided into two windows.

The upper window displays the branch, reach and turnout whose demand or structure setting is being prompted, and the maximum, minimum and data being keyed in. The user enters the data and the program plots a graph of the input as a function of time in the lower window (Fig. 22).

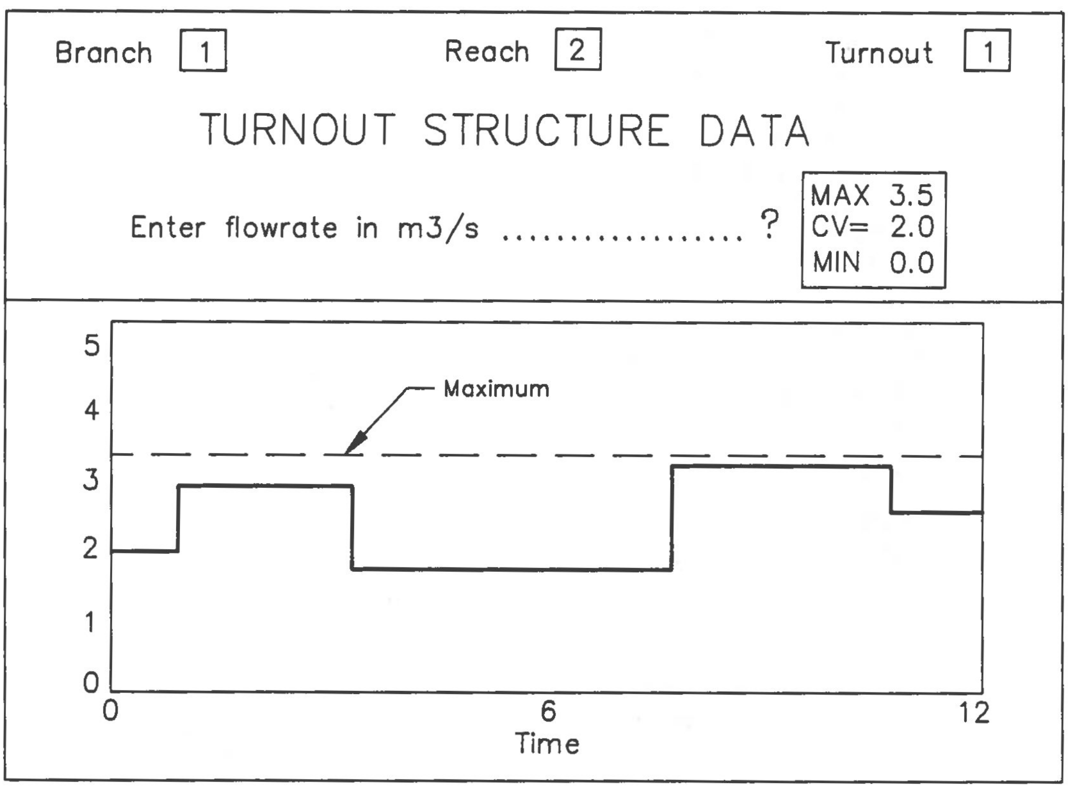

Figure 22. Operational Data Entry Screen.

Additional information that is displayed includes the elapsed time since the beginning of the simulation. It ranges between 0-12 hours for the 1st half of the day and 12-24 hours for the second half. The available commands are :

1. F1 function key - invokes the help menu;
2. Esc key - stops the program;
3. F9 function key - Exit operational data entry section and return to the calling subroutine;
4. Enter key - enter same value for the next five minutes; and
5. F10 - repeats entry of the same value until the end of the 12 hours or any key is pressed.

In entering a variable flow hydrograph or control structure setting, the user can either use the enter key for each time step data entry or press F10 key and wait until the elapse time equals the time to change the setting and then enter a new value and press F10 key again.

Output Displays

Computers normally generate output at a faster rate than a human mind can assimilate. Therefore, when output is to be displayed on the computer screen, it should be presented in a manner that optimizes the communication between the user and the computer. In this program, the output is arranged in the most logical sequence so that information needed for understanding any point is provided in advance insofar as possible. Scrolling screen output is avoided because it differs from the natural reading technique in that the material moves upwards instead of the eye moving downward across a steady display. Tabular data are displayed in logically related groups which fill the screen.

The model output consists of flow depths, flow rates and structure settings for in-line or turnout structures. This information is displayed on a graphical screen as shown on Fig. 23. The top window displays the flow profile of all of the reaches in the branch. This plot is updated at the end of each time step. The middle left window displays branch inflow and reach outflows while the lower one displays in-line structure settings. The middle right window displays turnout discharges while the lower one displays the turnout settings. These displays provides past and current values of the flows and structure settings.

The second option of output display is tabular. The tabular output consists of a fill-out form display in which only the numeric data are updated on a stable screen. Only one branch output can be displayed at a time. Thus, when simulating a system with more than one branch, the user selects the branch of interest and toggles to it from the current branch by simply keying in the number corresponding to the branch. The program redraws the background and past simulation status for the branch before proceeding on to display the current status. Pressing zero will display the schematic layout of the system with the branch being monitored highlighted. This option displays only the current status. Pressing F6 will change display from graphical to tabular. To switch back to graphics simply press F8. At the end of the simulation, summary plots of the flow rates and control structure settings are displayed for each branch. An option for printing the final results is also provided.

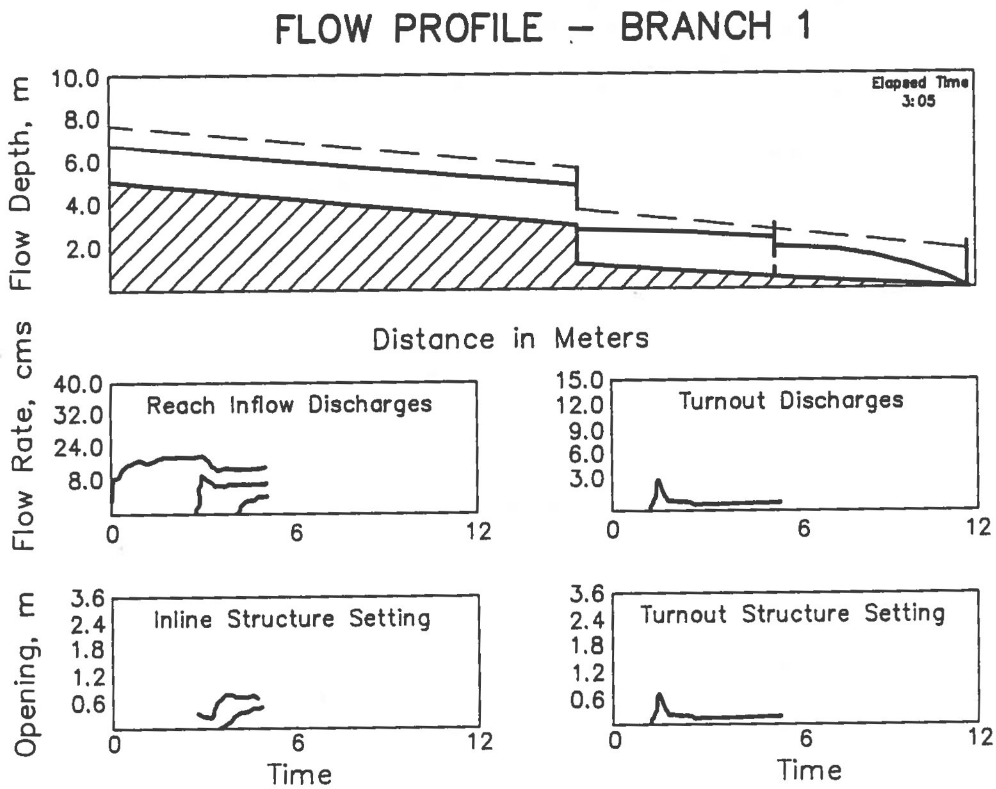

Figure 23. Graphical Output Display Screen.

User Interrupt

When running the program in an interactive mode, the user's interest in monitoring the simulation and intermediate output becomes exceedingly important and simulation interrupt capabilities vital. In simulating the effects of alternative future actions in a changing environment, the displays of both current and past results are crucial. This enables the user to observe the results, stop to critically examine any phase of the simulation, modify future actions, or go back in time and change model parameters, and then continue the simulation.

At the beginning of each time step loop, the program scans the keyboard buffer to determine whether any key had been pressed and reacts accordingly. The following interrupt options are provided:

1. **Number Key 1-5** - The program switches the branch being displayed to the one corresponding to the key that was pressed;
2. **F1 key** - The program temporally halts simulation and pulls out the on-line help menu;
3. **F5 key** - The program pauses simulation, giving the user time to critically examine the output display;
4. **F6 key** - The program switches from graphical output display to tabular form;
5. **F8 key** - The program switches from tabular to graphical display;
6. **Esc key** - The program aborts simulation; and
7. **Any other key** - The program displays the user interrupt menu.

When the canal overflows at a certain reach, the program displays the following overflow message.

```
WARNING - OVERFLOW

OVERFLOW AT :-
     BRANCH NO.  3
     REACH NO.  2

ABORT SIMULATION (Y/N)
```

The warning messages draw the users' attention to the deviations in the anticipated results. If the answer is no (N), the program switches the branch display to the one currently experiencing an overflow so that the user can see clearly what is happening and overlays the user interrupt menu shown below. This gives the user a chance to correct the operational mistakes and continue simulating.

```
     USER INTERRUPT MENU

A. CHANGE SYSTEM INFLOW
B. CHANGE TURNOUT DEMAND
C. CHANGE CONTROL STRUCTURE SETTING
D. CHANGE TURNOUT DATA DISPLAY
E. CONTINUE  S. SAVE AND STOP
```

On-line help

Due to the complexity of this program, an on-line help is incorporated to save the user the time of intensive reading of the user's manual or simply the expense of giving up in frustration. The user should be able to get information about program management, model parameters, etc. simply by pressing the F1 key. Help is available on the following topics:

1. Definition of terms used;
2. System configuration;
3. Operational data;
4. Data files;
5. Interactive simulation; and
6. Graphical display.

CHAPTER V

MODELING VERIFICATION

> All models are inevitably distortions of reality and no matter how elegant the conception, synthesis, and coding of a model, its abstract artificiality should never be forgotten. (Anderson J.R. and Dent. J.B. 1971. p. 388)

The above quote is aimed at stressing the importance of calibration and verification of mathematical models, and not at spurring further model skepticism. The model use dictates the amount of effort and time put into model verification and calibration. If the model is to be used in the operation of an irrigation project, the consequences of prediction error will be more important than if it is to be used as a training tool. The approach adopted in this study, not only compares final model results with those of the real system but also examines the intermediate output to ensure the localized divergence of the solution (which could occur and yet have a minimal effect on the overall system output) does not occur. A multi-stage approach, in which the validation proceeded concurrently with the model formulation and coding, was used. The fidelity of the model was tested during the development phase by verifying the model formulation, testing the coding by manually calculating the intermediate results, and thoroughly testing the individual subroutines for most conceivable cases before merging it to the rest of the program.

Verification Studies

The model was verified using field data collected on the first reach of South Gila Canal near Yuma, Arizona and on the Abraham Canal near Delta, Utah. Data on South Gila Canal were initially used to verify a mathematical model, Gate Stroking Model (GSM), developed by U.S. Bureau of Reclamation to study the performance of the prototype EL-FLO plus RESET controller and the response characteristics of an operating canal system as the controller responds to changes in turnout demands (Buyalski and Serfozo, 1979). The model was also verified by comparing it with the U.S. Bureau of Reclamation's model.

The Abraham canal conveys water over a relatively flat terrain from the Gunnison Bend Reservoir to service approximately 3600 hectares in Central Utah. During the summer of 1987, Tzou and Rodriquez (1987) collected data on the operation of the conveyance system. The data mainly consisted of upstream and downstream flow depths at both ends of previously calibrated structures and were collected at irregular interval. Flow rates were evaluated from the depth readings. These data were used to verify the simulation of the following conditions:

1. Flow into a dead-pool; and
2. Sudden opening of a sluice gate.

Single Reach Simulation

The field data on the South Gila Canal were collected on September 19, 1973. The data collected consisted of water levels upstream of the inline structures and the actual gate openings using electronic and mechanical chart recorders. Configuration data are presented in Fig. 24 and Table 7 shows the steady state initial condition. The gate setting data were extracted from Fig. 25 of the USBR report entitled "Electronic Filter Level Offset (EL-FLO) plus Reset Equipment for Automation Control of Canals," (Buyalski and Serfözo, 1979).

TABLE 7. Steady State Data for South Gila Canal.

Headwork inflow	1.16 m^3/s
Turnout MP0.5 discharge	0.0 m^3/s
Turnout MP1.19 discharge	0.15 m^3/s
Reach Outflow	1.01 m^3/s
Check structure opening	0.317 m.
Downstream flow depth	1.265 m.

The inflow hydrograph and check and turnout physical data were not available. The inflow hydrograph was generated based on the assumption that the operating head in the supply canal was constant throughout the study, thereby yielding a linear relationship between the headgate opening and the discharge. The check and turnout structure configuration (physical) data were obtained by varying the data until the steady-state condition yielded the flow depth and flow rate equal to the field data readings. The canal operation for the 13 hours study was as follows:

Time (hr)	Description of Event
0.00	Beginning the test with the steady state conditions described above.
0.25	Downstream gate opening increased from 0.317 m to 0.378 m.
3.75	Downstream gate opening reduce to 0.344 m.
4.36	Turnout locate at MP0.5 was turned. Turnout discharge increased from 0.0 to 0.35 m^3/s.
9.28	Downstream gate opening reduced to 0.256 m.
13.00	End of the test.

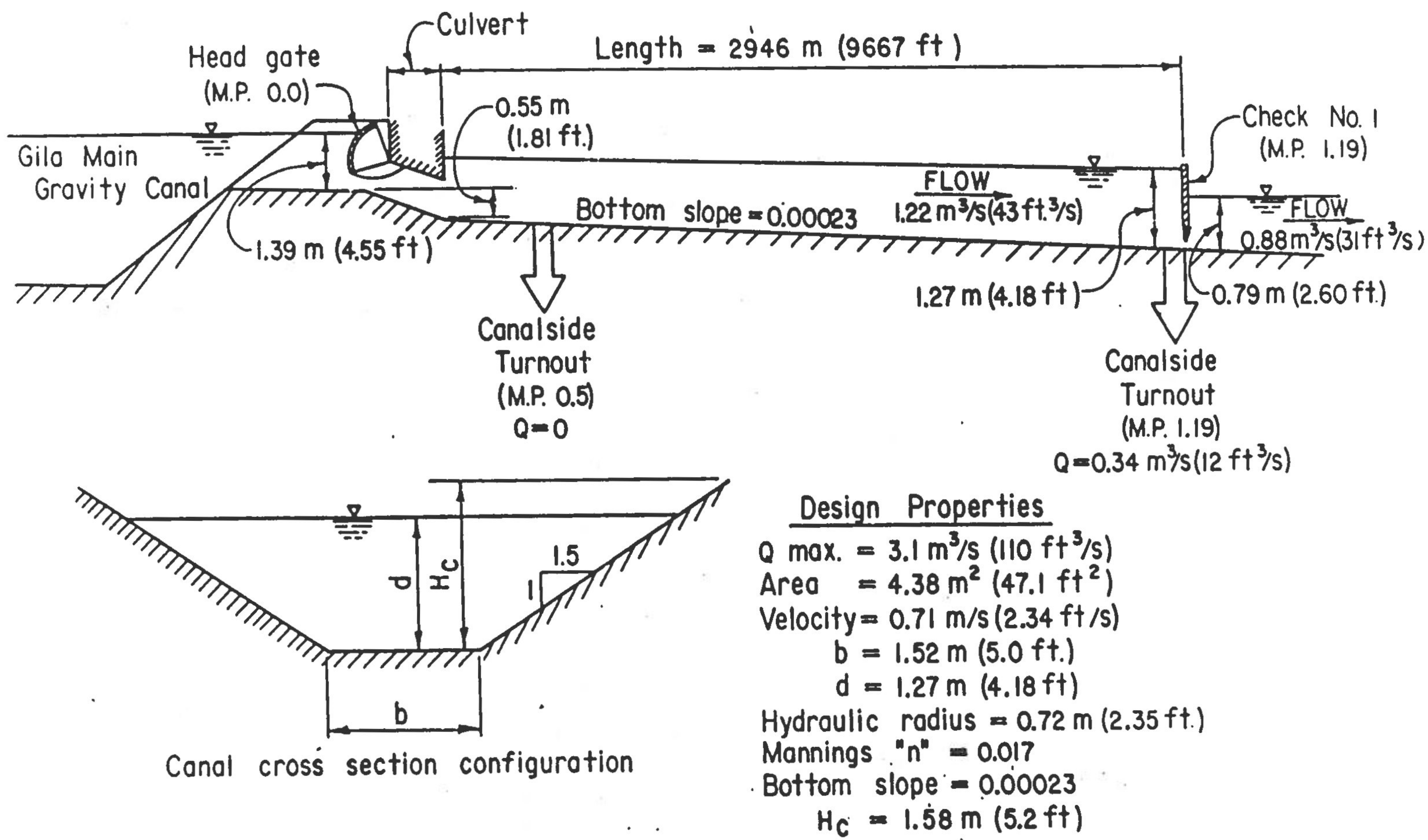

Figure 24. Schematic of South Gila Canal Configuration. (Buyalski and Serfozo, 1979)

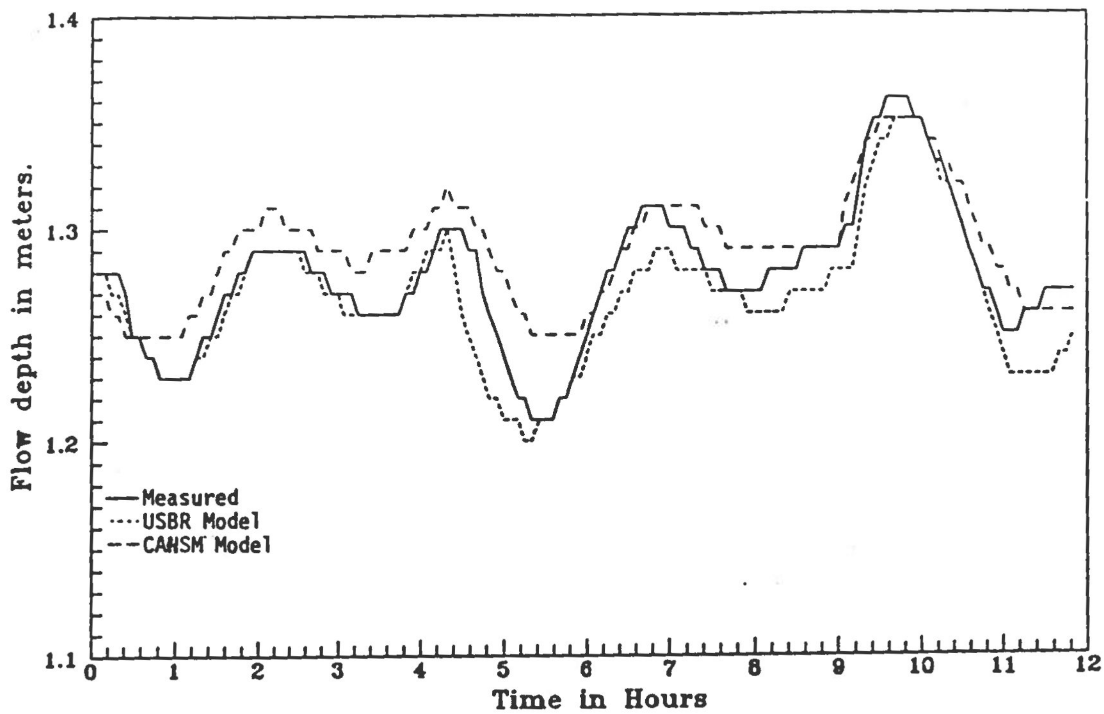

Figure 25. Measured Versus Predicted Flow Depth for South Gila Canal.

Figure 25 shows that the mathematical model simulated the flow depth response with a reasonable accuracy. Although there is no point to point agreement between actual and simulated water levels, there is a distinct similarity in flow depth and the largest deviation is only six centimeters. The CAHSM model consistently over-estimated the flow depth except for the peak flow depth that occurred between hours 9-10.

The major causes of the deviation between the mathematical model and the recorded data are:

1. The mathematical model uses a five minute time step which explains why the model did relatively poorly in predicting the peaks and valleys. The field data were collected continuously;
2. Errors in extracting flow rate, flow depths and gate openings from the figures;
3. Errors in approximating the initial steady state conditions;
4. Failure to take into account the variation in turnout flow rates in field data collection; and
5. The model was not fully calibrated due to lack of a complete set of data describing the steady-state flow conditions.

Flow into a Dead-pool

The canal reach used to study the flow of water into a dead-pool has the configuration and initial condition data shown in Table 8.

TABLE 8. Abraham Canal Configuration Data.

REACH INFORMATION	REACH 1	REACH 2
CROSS-SECTIONAL DATA		
Max Flow Depth in meters	1.50	1.50
Normal Flow Depth in meters	1.20	1.20
Side Slope	4.00	4.50
Bottom Width in meters	4.50	3.00
LONGITUDINAL DATA		
Channel losses in cm/day	0.00	0.00
Manning's N	0.035	0.035
Longitudinal slope in m/1000m	0.210	0.140
Length of the Reach	3377.00	2323.00
Length of Structure between reaches .	0.00	0.00
Change in canal bottom elevation	0.00	0.00
CONTROL STRUCTURE INFORMATION		
TYPE	Sluice	Sluice
Minimum sill height	0.0	0.0
Discharge coefficient	0.55	0.70
Width of the structure	0.91	0.97
INITIAL CONDITIONS		
Reach inflow (m^3/s)	0.0	0.0
Reach outflow (m^3/s)	0.0	0.0
Upstream flow depth (m)		
Downstream flow depth (m)	0.8	1.1
Gate opening (m)	0.0	0.0

The inflow into reach #1 was zero at the beginning of the study and it was suddenly increased to 4.5 m^3/s. Figure 26 shows the inflow hydrograph used. Because data were collected at irregular intervals, the five minute time step values required by the model were obtained by linear interpolation. The sluice gate #1 remained closed for 3 hours and then opened by 0.65 m whereas gate #2 was opened by 0.14 m after 4.5 hours.

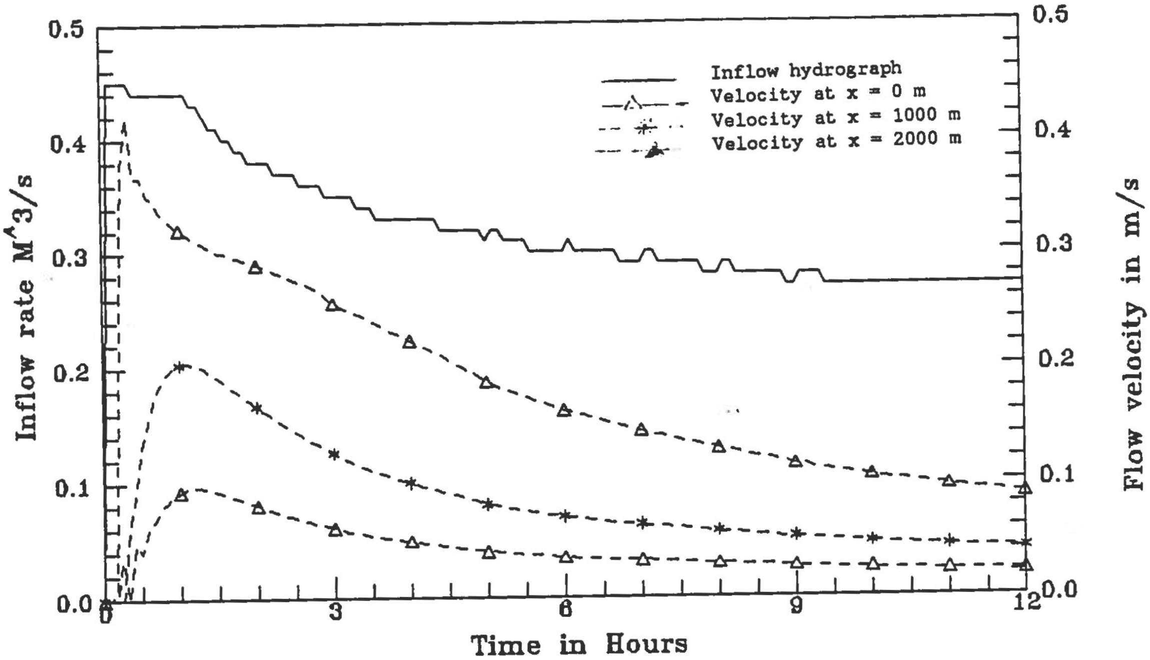

Figure 26 Inflow Rate and Flow Velocity at Various Locations Reach #1

Figure 26 also shows flow velocities at various locations. As one might expect, the velocity was highest at the upstream end and gradually decreased to zero at the downstream end because the gate was close. The model produced slight instability at the upstream end when the water was introduced into the still pool but handled the transient flow well (no field data on the movement of the wave were collected). The gradual decline in flow velocity is consistent with the change in inflow hydrograph.

Sudden Opening of a Sluice Gate

Two reaches of the Abraham canal were used to verify the simulation of sudden opening of a sluice gate. The configuration data and initial condition are as shown in Tables 8 and 9. The sluice gate #1 remained closed for 3 hours and then opened by 0.65 m. whereas gate #2 was opened by 0.14 m. after 4.5 hours. Figure 27 shows observed and simulated reach outflow hydrographs. Upon opening gate #1, a positive and a negative wave were initiated. As the positive wave travelled downstream, it increased the flow depth in reach #2 (see Figs. 28-30). Conversely, the negative wave travelling upstream in reach #1 reduced the flow depth. The increase in flow depth downstream of gate #1 and the reduction in depth upstream gradually reduced the operating head and the discharge across the structure. Figure 30 shows the relationship between upstream and downstream flow depth, gate opening, and discharge across gate #1.

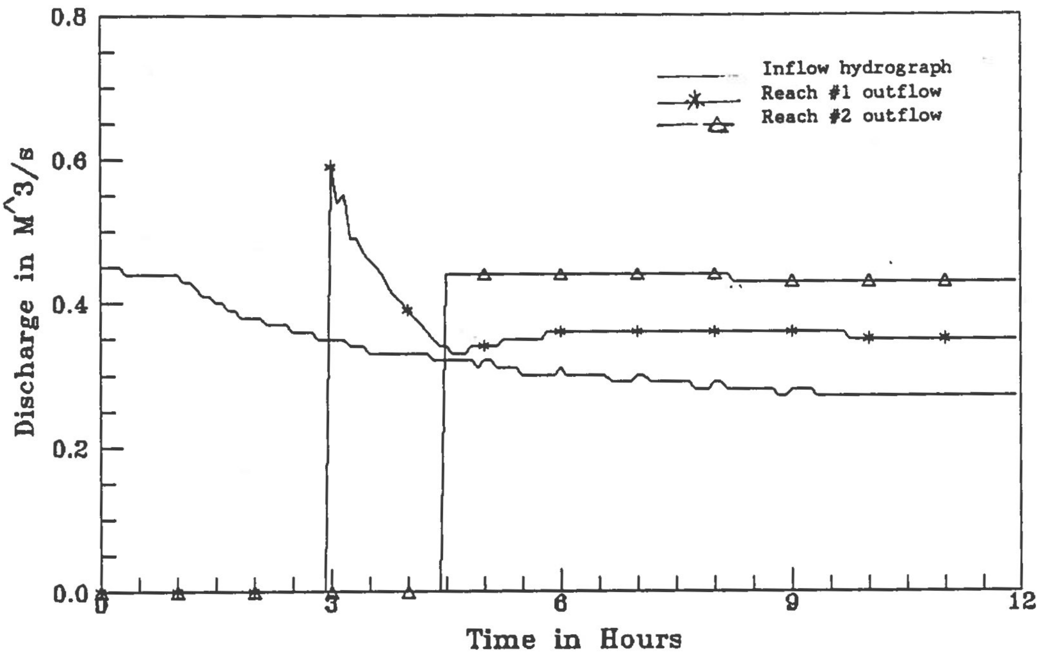

Figure 27. Inflow and Outflow Hydrographs for Reach #1 and #2

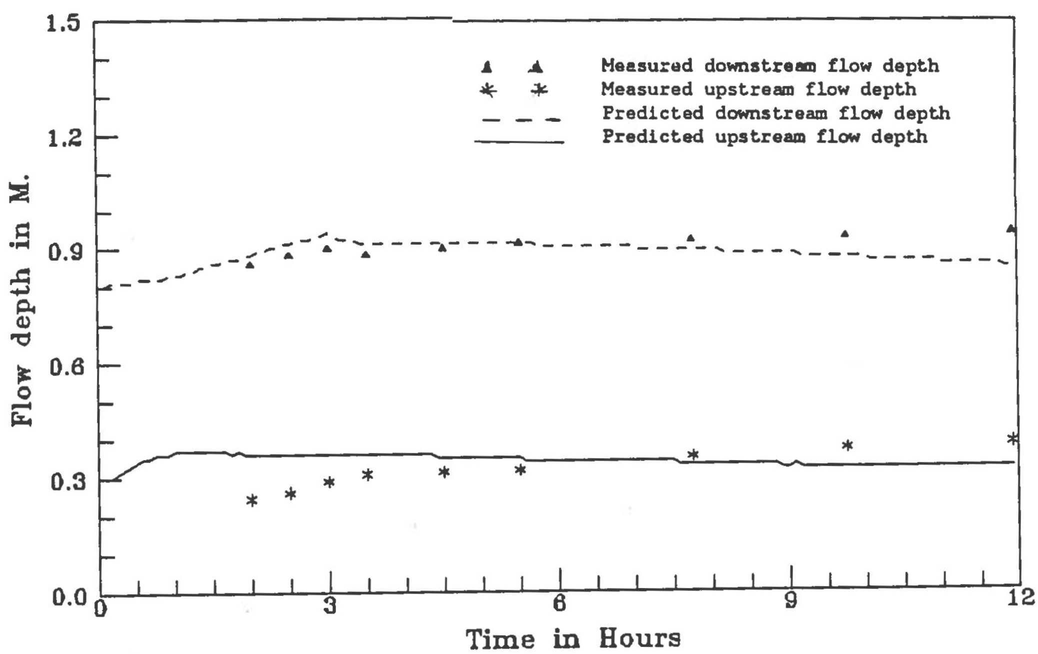

Figure 28. Upstream and Downstream Flow Depth Reach #1

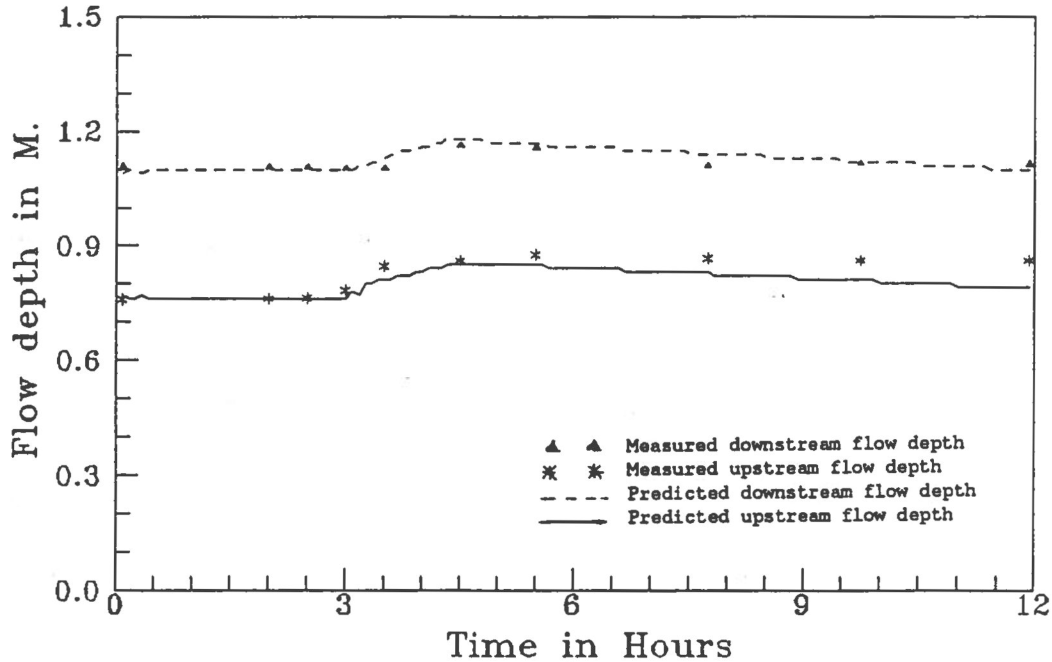

Figure 29. Upstream and Downstream Flow Depth Reach #2

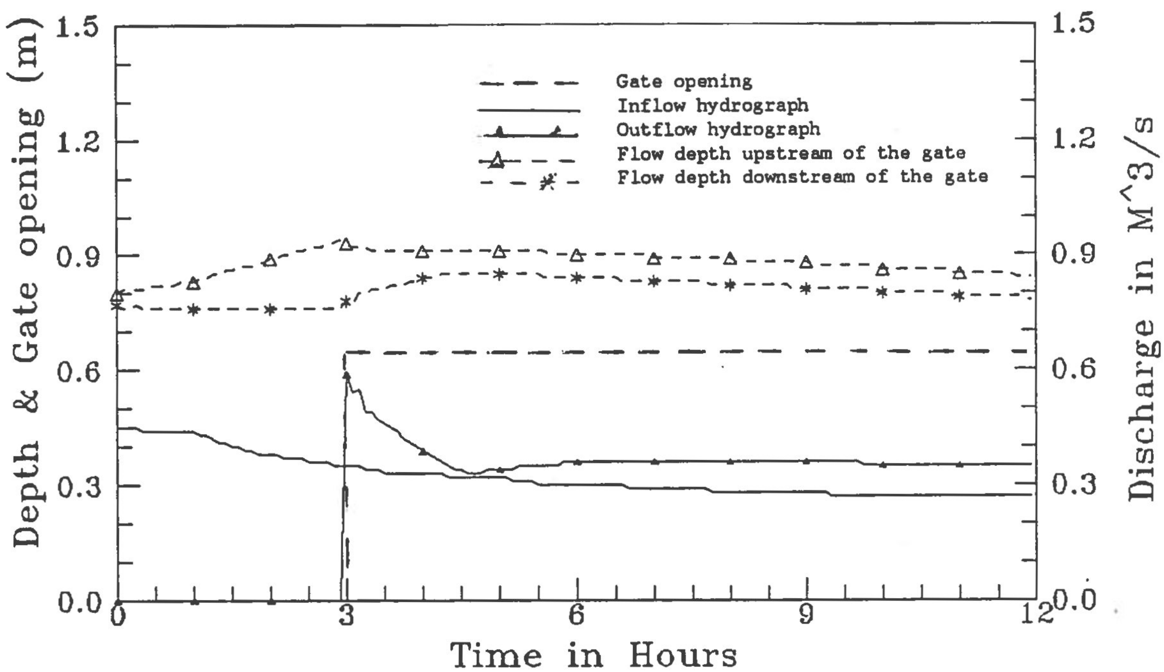

Figure 30. Flow Conditions at Sluice Gate #1

The sudden opening of gate #2 did not have a similar effect because it was free flowing and the fact that the gate opening was small and the operating head high (see discussion on the choice of structures in Chapter II).

Submerged Flow Conditions

Because irrigation canals aim at serving as large a command area as possible, the slopes are generally low and submerged flow conditions common. The case presented above illustrates the capability of the model to simulate submerged flow conditions for a two reach canal section. This section compares the model calibration with that of the US Bureau of Reclamation's (GSM) model on a five-reach system. The configuration and initial condition data are presented in Table 10. The data for this study is presented in english units.

TABLE 10 Corning Canal Configuration Data.

REACH INFORMATION	REACH 1	REACH 2	REACH 3	REACH 4	REACH 5
CROSSECTIONAL DATA					
Max Flow Depth in ft	10	10	10	10	10
Normal Flow Depth in ft...	8	8	8	8	8
Side Slope	2	2	2	2	2
Bottom Width in ft........	22	22	22	20	20
LONGITUDINAL DATA					
Channel losses in ft......	0	0	0	0	0
Manning's N	0.0225	0.025	0.0225	0.0225	0.0225
Longitudinal slope ft/1000ft.	0.098	0.181	0.118	0.1	0.14
Length of the Reach	23889	7167	10750	13910	12917
Length between reaches .	0	0	0	0	0
Change in canal	0	0	0	0	0
CONTROL STRUCTURE INFORMATION					
TYPE	Sluice	Sluice	Sluice	Sluice	Sluice
Minimum sill height	0	0	0	0	0
Discharge coefficient	0.8	0.8	0.8	0.7	0.5
Width of the structure ft....	10	10	10	13	13
INITIAL CONDITIONS					
Reach inflow (cfs)	144.69	143.18	128.06	119.98	109.9
Reach outflow (cfs)	143.18	128.06	119.98	109.9	79.65
Upstream depth (ft)					
Downstream depth (ft)	7.17	7	6.70	7.09	6.39
Gate opening	1.701	1.572	1.6	0.661	0.348

The GSM model simulation results, control structure setting and inflow hydrograph provided the input data for CAHSM user-specified input operation run. Figure 31 presents the model inflow hydrograph. The first wave was created by increasing flow rate from 144.69 to 161.5 cfs in 20 minutes. The second increased the flow from 146 to 147 in 20 minutes. After one and a half hours the flow rate remained constant for the remaining simulation period. The flow rate increase did not increase the flow depths as would be expected due to the control structure settings that were implemented to maintain a nearly constant water surface profile despite the change in flow rate.

Plots of simulated flow depths for both GSM and CAHSM are presented in Figs 32-36. The results show a close match between the two simulation models for all the reaches. In reach #1 CAHSM model over-estimated the downstream flow depth and under-estimated the upstream flow depth. The differences are however not significant (see Fig. 32). The CAHSM model over-estimated flow depth in reaches two and three because of an increase in flow rate to this reach resulting from an over-estimation of downstream flow depth in reach #1 (see Figs. 32-34). The volume passed from reach one was stored in the upstream end of reach #4 which shows an over-estimation of the upstream flow depth (see Fig. 34).

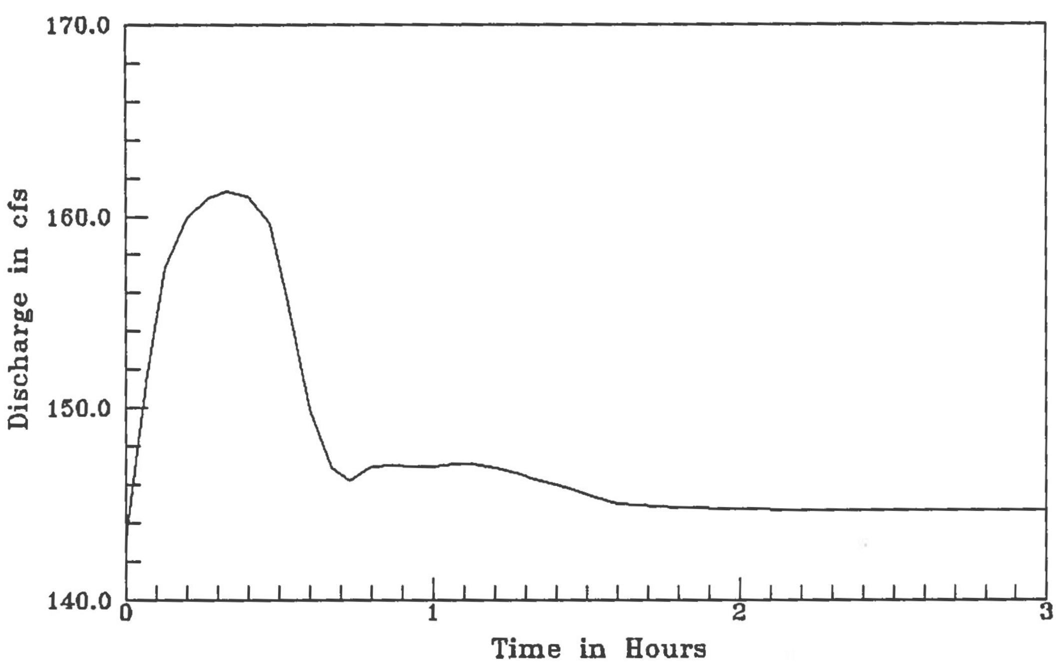

Figure 31. Inflow Hydrograph

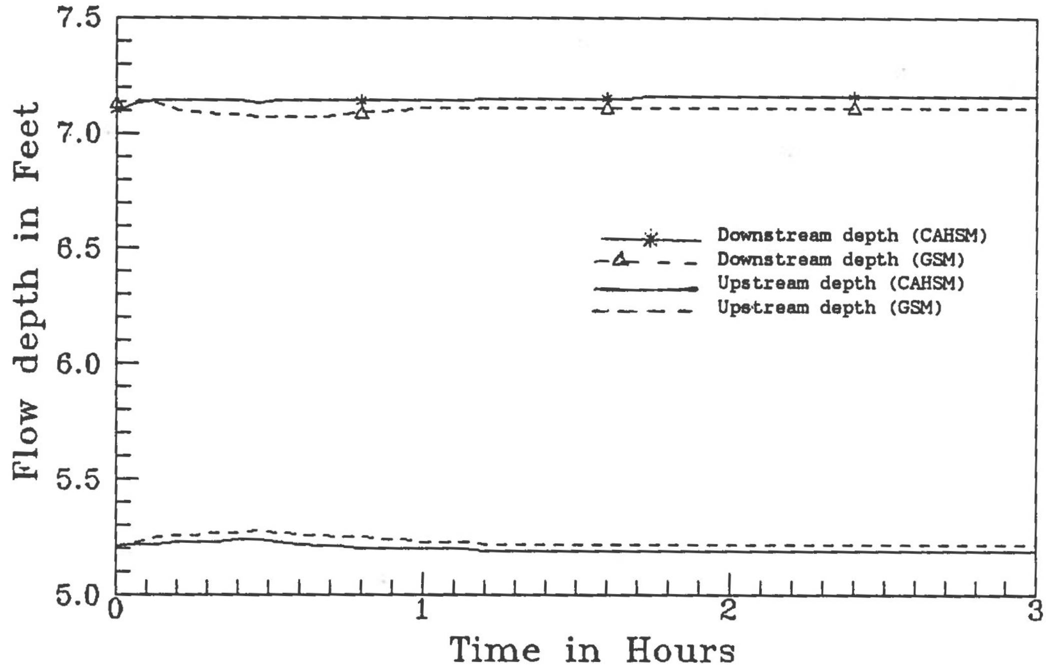

Figure 32. Upstream and Downstream Flow Depth Reach #1

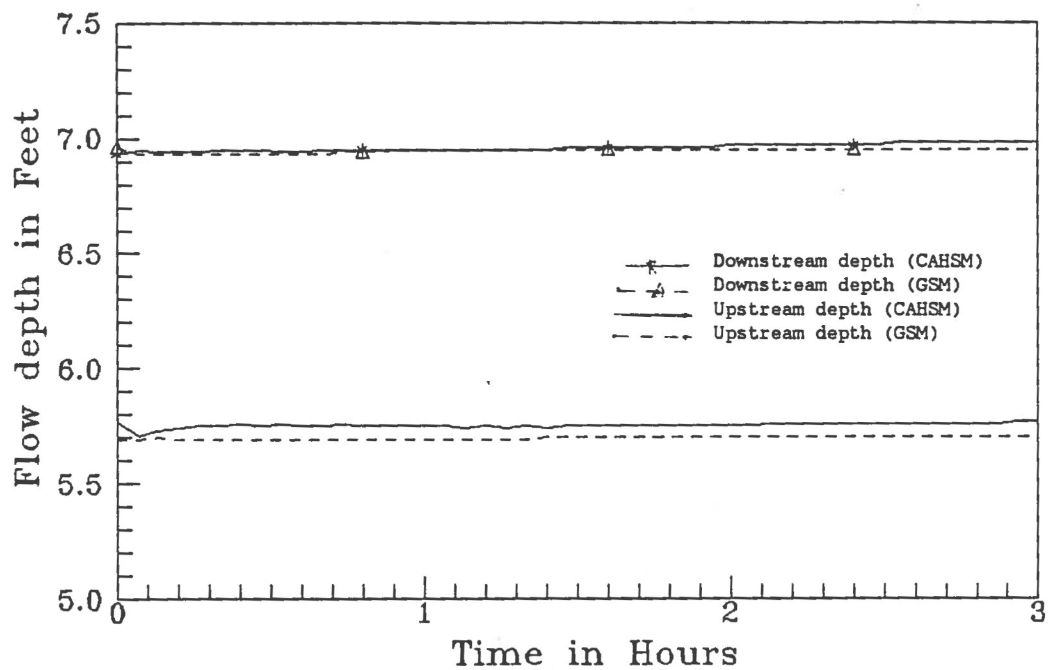

Figure 33. Upstream and Downstream Flow Depth Reach #2

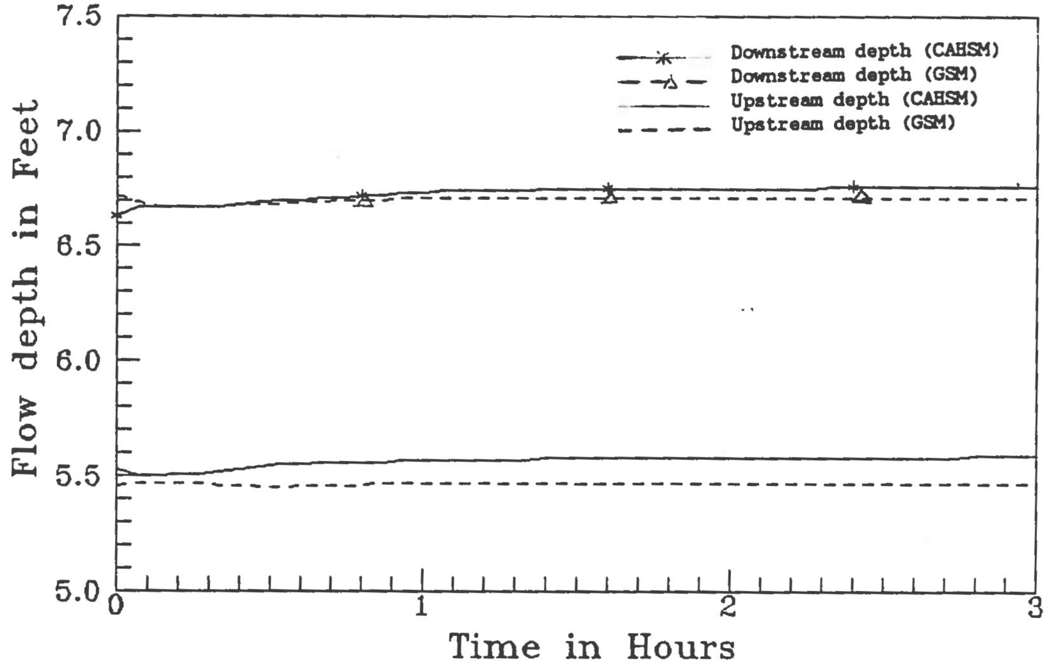

Figure 34. Upstream and Downstream Flow Depth Reach #3

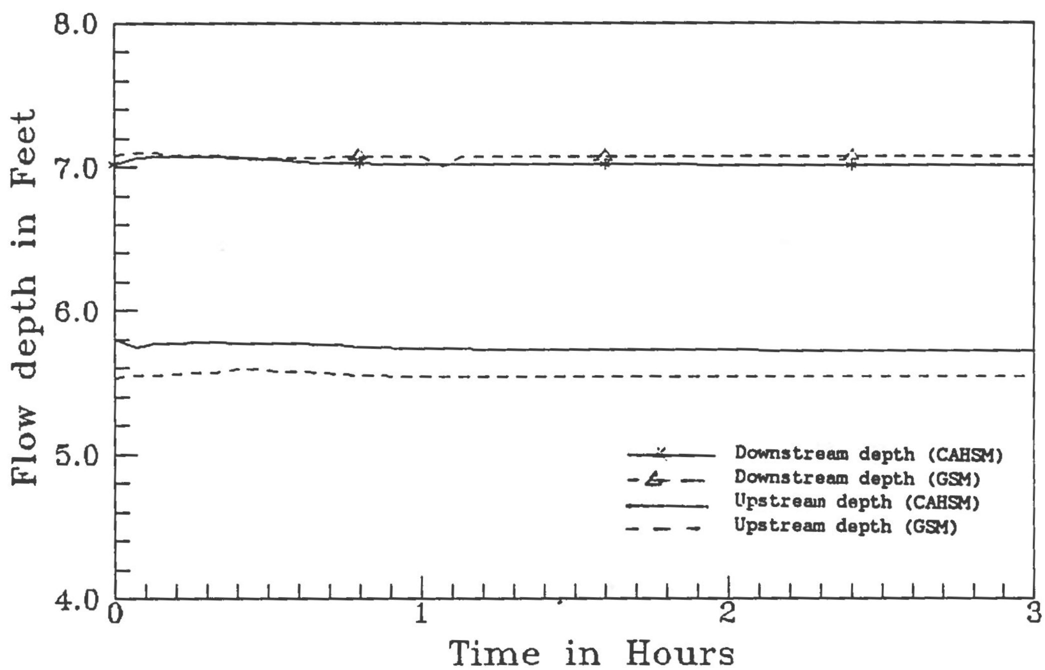

Figure 35. Upstream and Downstream Flow Depth Reach #4

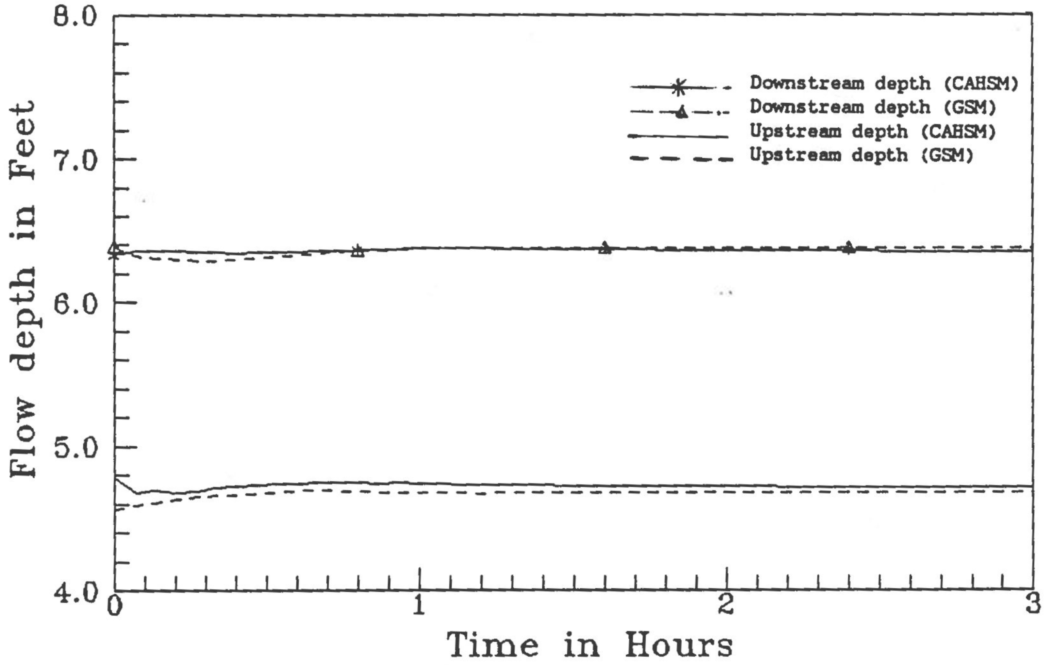

Figure 36. Upstream and Downstream Flow Depth Reach #5

Conclusions

Although lacking complete information, this verification demonstrates that the mathematical model predicted the overall hydraulic response with reasonable accuracy. Use of the model to investigate hydraulic responses of canal systems for various physical and operating conditions can now be made with a higher degree of confidence.

CHAPTER VI

APPLICATION OF THE MODEL

The general consensus is that irrigation performance is constrained by our ability to make the right decisions and that the provision of the necessary tools and methodologies will go a long way in improving the performance. Walker and Skogerboe (1986) observed that:

> Analysis of an irrigation system must be prepared to deal with a multitude of important linkages between the watershed, storage facilities (if present), main system and the individual command areas. These linkages are not generally considered in sufficient details during the design, operational or rehabilitation phases of an irrigation project. As a consequence, a large variety of operational weakness have developed (p. 2).

In searching for system improvements, there is a need to predict the system's behavior for various operating scenarios and to determine the appropriate procedure to achieve the desired objective. The advent of computers and the development of numerical and systems analysis technology is luring researchers and facilitating the development of multi-disciplinary methodologies. Advances in computer technology have resulted in the introduction of rugged, affordable micro-computers with ample computing power and memory. The increased availability of this cost-effective technology has stimulated the development of improved analytical tools and methodologies for evaluating alternatives so that the best possible decision can be made. This is making the synthesis and analysis of alternative future designs and operational policies more readily available to engineers, planners, and system operators throughout the world. It is, however, important to stress the fact that: (1) the micro-computers will only produce technically feasible solutions, and therefore, financial, socio-economic, and political acceptance of these solutions should be reviewed before arriving at the final operating procedure; and (2) the use of models should be complementary to the study of the real system and therefore any model solution must be ultimately proven in a real system setting.

Uses of the Model

The CAHSM model provides irrigation professionals and amateurs with a vehicle to address issues related to the interaction between design and operation of a canal network as one pursues a more dynamic operation and management of the conveyance network. It can be used to: (1) evaluate the hydraulic response of the conveyance network to changes in physical and operational features of the system to determine the best combination of controllable parameters to achieve pre-decided system performance goals; (2) establish opportunities and constraints of the system; (3) determine points in the system that are sensitive to managerial interference; (4) predict system behavior; and (5) provide guidelines on possible improvements on water control over the entire system. Some of the specific uses of the model can be grouped into three categories: (1) design issues; (2) evaluation of required interventions; and (3) operational and training issues.

Design issues include:

1. Investigating the effect of different control structures on canal hydraulics;
2. Determining the optimal type, number, and location of control structures;
3. Investigating the need for intermediate reservoirs, their location, and their capacity in order to reduce the spills and reduce system lag time; and
4. Subjecting the conveyance and distribution system design to various operating scenarios to identify operational bottlenecks before the system is constructed.

Evaluation of possible interventions include:

1. Determining the hydraulic response (lag time, water level and discharge fluctuations) associated with varying levels of maintenance;
2. Determining operational schedules and policies that are most appropriate for good management; and
3. Evaluating the performance of an existing system to determine the need for rehabilitation.

Finally, operational and training issues include:

1. Determining the optimal control structure settings required to minimize the discrepancies between demand and supply;
2. Determining the optimal inflow hydrographs for a given command area's demands;
3. Determining the effects of the command area's rejected demand on the canal hydraulics and the appropriate control structure settings to minimize the spills;
4. Determining the optimal filling and emptying time of the channels in order to: (a) minimize losses; (b) minimize delayed deliveries; and (c) prevent rapid filling or draining of the channels;
5. Determining the optimal hydrograph and control structure settings that will minimize water level fluctuations and/or limit rapid filling or drawdawn; and
6. Training operational staff on how best to operate and manage the system.

Case Studies

Experimenting with analytical models has received considerable attention over the last few years due to its distinct advantages over physical experimentation in that the modeler has control over the variability of model parameters. When using simulation techniques, the experimenter can achieve perfect homogeneity of the experimental medium, allowing treatment to be compared under identical conditions. The experimentation with simulation models facilitates:

1. Estimation of the response of the system to changes in the levels of a single input;
2. Exploration of the response surface, generated for different combinations of input levels;
3. Comparison of alternative courses of action; and
4. Estimation of the input combinations required for an optimal or near optimal level of output.

The case studies presented here illustrate how this model can be used to access the reliability of water delivery and also to generate guidelines on how to improve the reliability of water delivery. As previously noted, the reliability of water supply is influenced by both physical and human factors. Although, this study is biased on the physical factors, it acknowledges the importance of the human factors in implementing the proposed plan.

Objective

The objectives of these case studies are: (1) to determine how the hydraulic structures affect the water delivery; and the identification of effective and responsive canal operations that would increase the manageability of the system. The specific questions to be answered are:

1. What is the impact of non-regulation?
2. How should the system be operated to minimize delivery flow rate fluctuations and the discrepancy between supply and demand? and
3. What effect does over or under estimation of model input values have on the deliveries?

Description of the Physical System

A single reach with four turnouts and a downstream demand is considered. Two types of free-flowing turnouts structures, weir and orifice are used. The test reach configuration is shown in Table 11.

The delivery objective was to supply 1.2, 1.25, 1.09, and 0.77 m^3/s to turnouts 1 to 4 respectively and 7.69 m^3/s to the downstream users for the first 13 hours of the operation, then increase it to

13.69 m^3/s for 12 hours and finally reduce it to 7.69 m^3/s for 11 hours. The reach turnout demands remain constant throughout the 36 hours of simulation period. Hence, the reach inflow was 12 m^3/s for the initial 13 hours, then increased to 18 m^3/s for 12 hours and finally reduced to 12 m^3/s for the last 11 hours.

TABLE 11. Configuration Data.

CROSSECTIONAL DATA	
Max Flow Depth in meters	3.00
Normal Flow Depth in meters	2.70
Side Slope	1.50
Bottom Width in meters	5.00
LONGITUDINAL DATA	
Channel losses in cm/day	0.00
Manning's N	0.014
Longitudinal slope in m/1000m	0.125
Length of the Reach	10000.00
Length of Structure between reaches .	0.00
Change in canal bottom elevation	0.00
CONTROL STRUCTURE INFORMATION	
TYPE (Adjustable Sill Weir)	2
Minimum sill height	0.4
Discharge coefficient	1.86
Width of the structure	4.00
TURNOUT STRUCTURE INFORMATION	
TYPE (Circular gated structure)	4
LOCATION (distance from upstream end)	3000.0
Minimum sill height	0.4
Discharge coefficient	0.6
Diameter in meters	1.00
TYPE (Adjustable Sill Weir)	2
LOCATION (distance from upstream end)	3015.0
Minimum sill height	0.4
Discharge coefficient	1.86
Width of the structure	4.00
TYPE (Circular gated structure)	4
LOCATION (distance from upstream end)	7000.0
Minimum sill height	0.4
Discharge coefficient	0.6
Diameter in meters	1.00
TYPE (Adjustable Sill Weir)	2
LOCATION (distance from upstream end)	7015.0
Minimum sill height	0.4
Discharge coefficient	1.86
Width of the structure	4.00

The Impact of Non-regulation

In studying the effects of non-regulation of a canal network, six fixed control structure settings are to be assumed. As noted above, the downstream demand is changed and the headwork inflow adjusted accordingly but all control and turnout structures settings are held constant throughout the test (see Table 11). The objective is to determine the deviation between supply and demand for the aforementioned settings.

The results in Table 12 show that as the structure setting (sill height) is increased, the reach storage and turnout discharges increase at the expense of reach outflow (downstream supply). Figure 37 shows how the downstream supply varied. For sill heights less or equal to 0.6 m., the flow stabilized with a drawdown flow profile. Consequently, turnouts three and four were under-supplied and the downstream supply was greater than the demand. Increasing the sill height influenced the flow depth over most of the reach.

TABLE 12. Percent Distribution of Inflow Volume for Different Inline Structure Sill Height.

	Sill Height in Meters					
	0.4	0.6	0.8	1.0	1.2	1.4
Reach Storage	6.16	6.67	7.08	7.56	7.97	8.52
Reach outflow	66.67	64.76	62.40	59.49	56.19	52.51
Turnout 1	8.18	8.29	8.39	8.55	8.74	8.96
Turnout 2	9.00	9.17	9.45	9.85	10.37	11.00
Turnout 3	6.71	6.98	7.31	7.64	8.00	8.33
Turnout 4	3.28	4.13	5.37	6.91	8.73	10.68

The results on Table 12 also, show that the weir turnouts (turnouts 2 and 4) experienced greater increases in discharge than did the orifice type turnout. This is due to the fact that the discharge over the weir varies with the operating head raised to the 3/2 power, as opposed to 1/2 power for the orifice turnout. Also, as the sill height increased, the turnout structures at the lower end (3 & 4) experienced a higher increases in delivery than the upper turnouts (1 & 2).

Figure 38 shows how delivery to turnout number one, located at a distance of approximately 7000 meters from the control structure, increased as the sill height was increased. Due to lack of regulation, changes in inflow causes changes in the turnout discharge, too. The magnitude of the changes depend on the control structure settings (high control structure setting results in high deviation between the demand and supply especially during the period of inflow change). Thus, lack of regulation leads to very serious translocation of water.

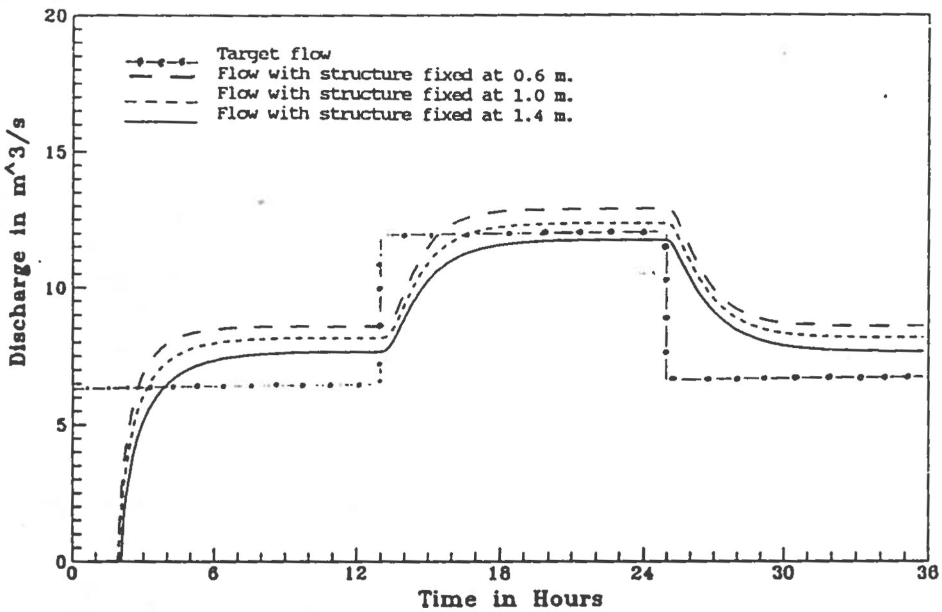

Figure 37. Downstream Supply Hydrograph as a Function of the Inline Structure's Sill Height.

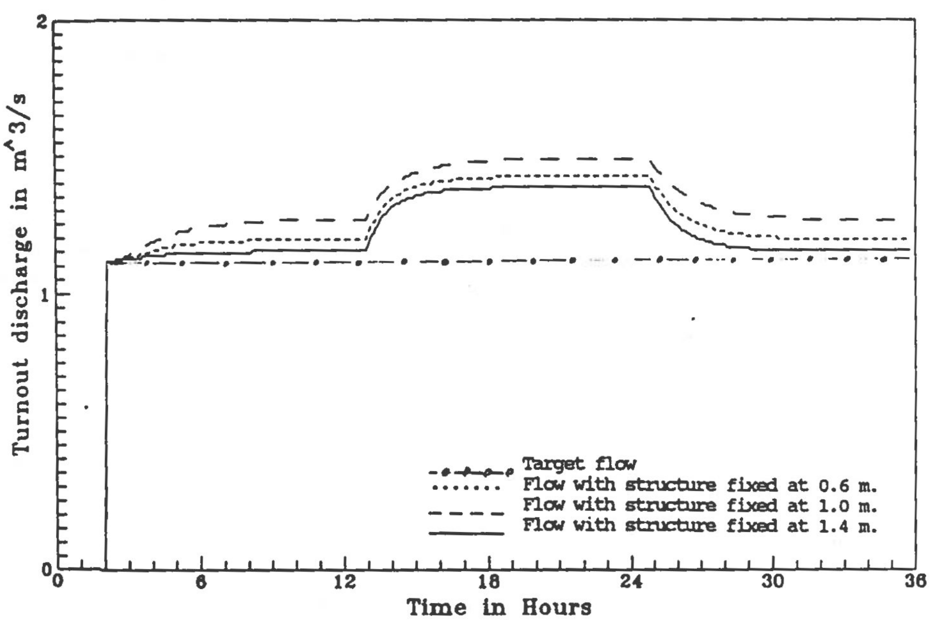

Figure 38. Turnout #1 Supply Hydrograph as a Function of the Inline Structure's Sill Height.

This analysis clearly indicates the profound effect that the downstream control structure has on the reliability of water supply, and stress the need for built in flexibility in the operation of the structures to cater to the changing demands experienced in day to day operation of a conveyance system.

Control Structure Scheduling

The major accomplishment of this study is the development of a model that will determine the optimal control structure operations that are required to ensure a reliable supply. This section is devoted to applying the model to generate an operational scheme that is required to achieve the predetermined objective. In this case, the requirement is to deliver water as closely as possible to the demand. Figure 39 shows the inflow and downstream supply hydrograph. Because the reach turnout demands were constant, the downstream supply hydrograph parallels the inflow hydrograph very closely. Figure 40 shows the turnout discharges which are fairly constant. This operational schedule ensured that the water destined for downstream users was not taken by the upstream users.

Figures 41 and 42 show the control structure schedule required to achieve the predetermined goal. Although it is fairly constant for the orifice structures, it is highly variable for the weir type of control structures. Such a control structure schedule would require more labor or the use of automatic gates. Whereas the fixed structure setting operation (with minimum structure and labor investment), resulted in preferential treatment for the head-enders, this option provides a high level of water control but requires substantial investment in structures and/or operation labor.

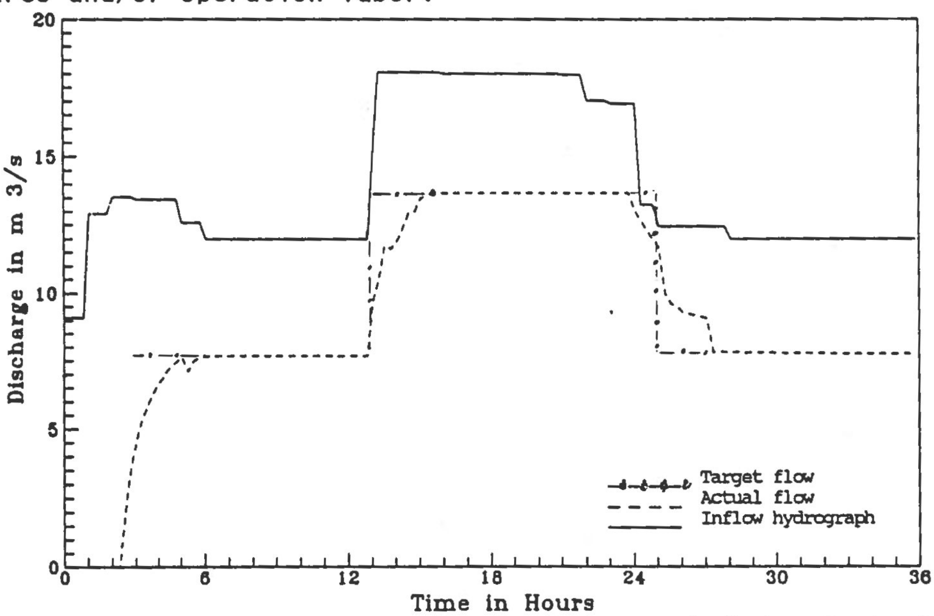

Figure 39. Inflow and Downstream Supply Hydrograph Under Control Structure Scheduling Operation.

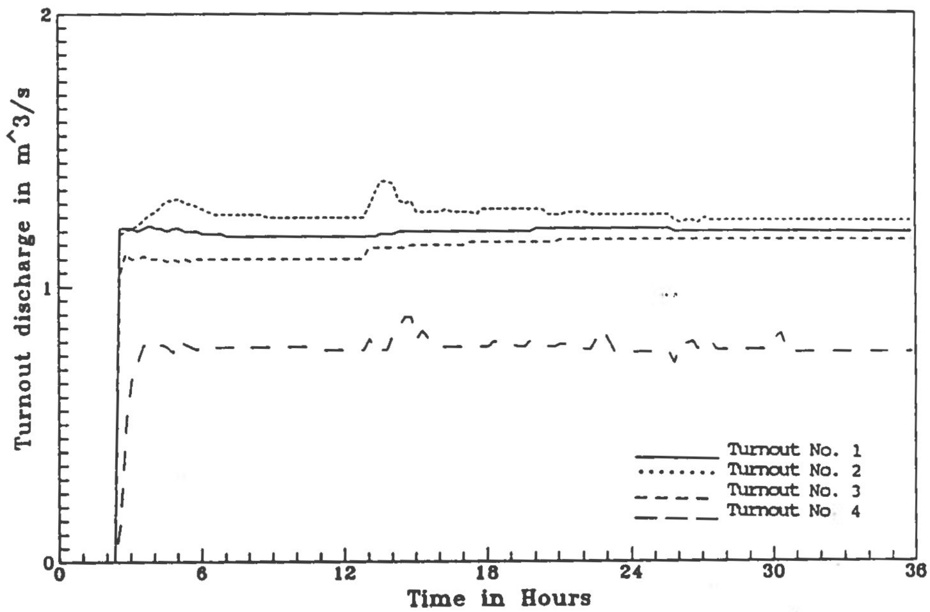

Figure 40. Turnout Supply Hydrographs Under Control Structure Scheduling Operation.

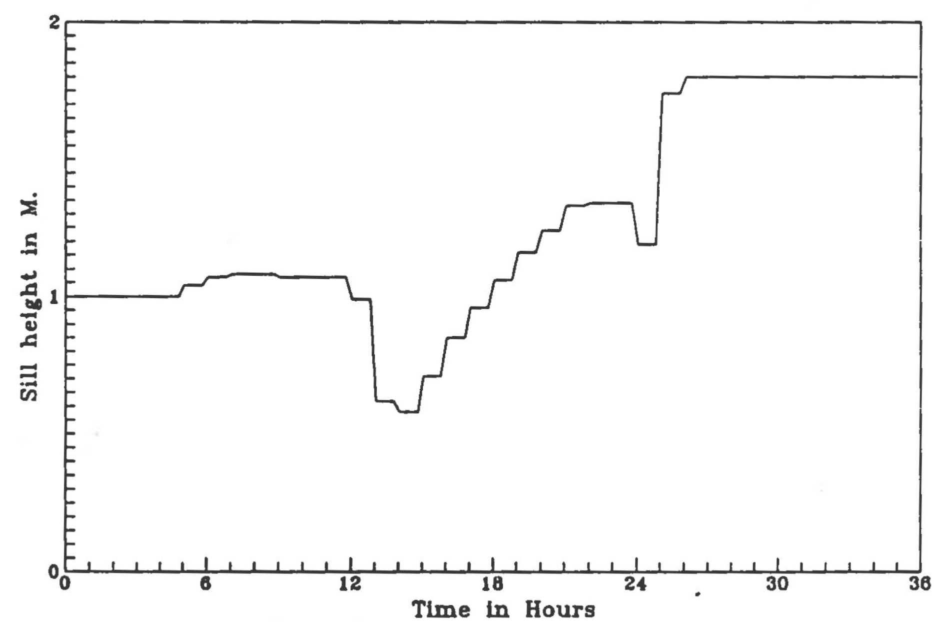

Figure 41. Inline Control Structure Setting Under Control Structure Scheduling Operation.

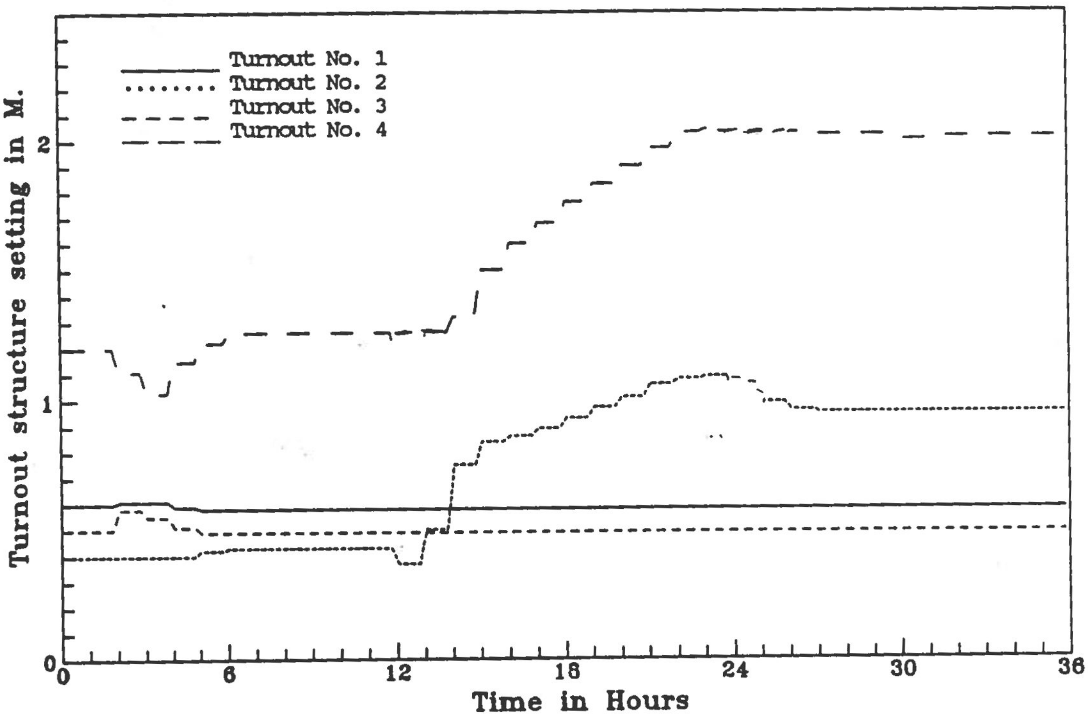

Figure 42. Turnout Control Structure Setting Under Control Structure Scheduling Operation.

<u>Sensitivity to Errors in Model Inputs</u>

The simulated flow conditions: flow cross-sectional area; flow rates; and flow depths are dependent on the model inputs, some of which (slope, Manning's n, coefficient of discharge, seepage rate, etc.) maybe difficult to determine accurately. This section explores the effect of over- or under-estimation of the model input on the simulation results. Over estimating some parameters and under-estimating others may have a compounding or cancelling effect. However, in the following analyses, only one parameter is varied at a time in order to illustrate the magnitude each variable has on the results.

<u>Longitudinal slope</u>. The slope was varied from 0.125 m/1000m to 0.225 m/1000m by a step of 0.05. Figures 43 and 44 show the increase in reach outflow and flow depth at the downstream end as the longitudinal slope was increased. Table 13 indicates that although the flow depth at the downstream end of the reach increased, the reach storage decreased as the depth upstream end decreased due to higher velocities. The flow from upstream were also reduced by an increased slope as shown in Fig. 45. Here again the weir type turnout structures experienced the greatest decrease. It appears from the results that errors in defining the slope would have to be substantial before large errors would be noticed in flows and flow depths. One would not expect erosion and siltation process to create as large errors as used in this illustration.

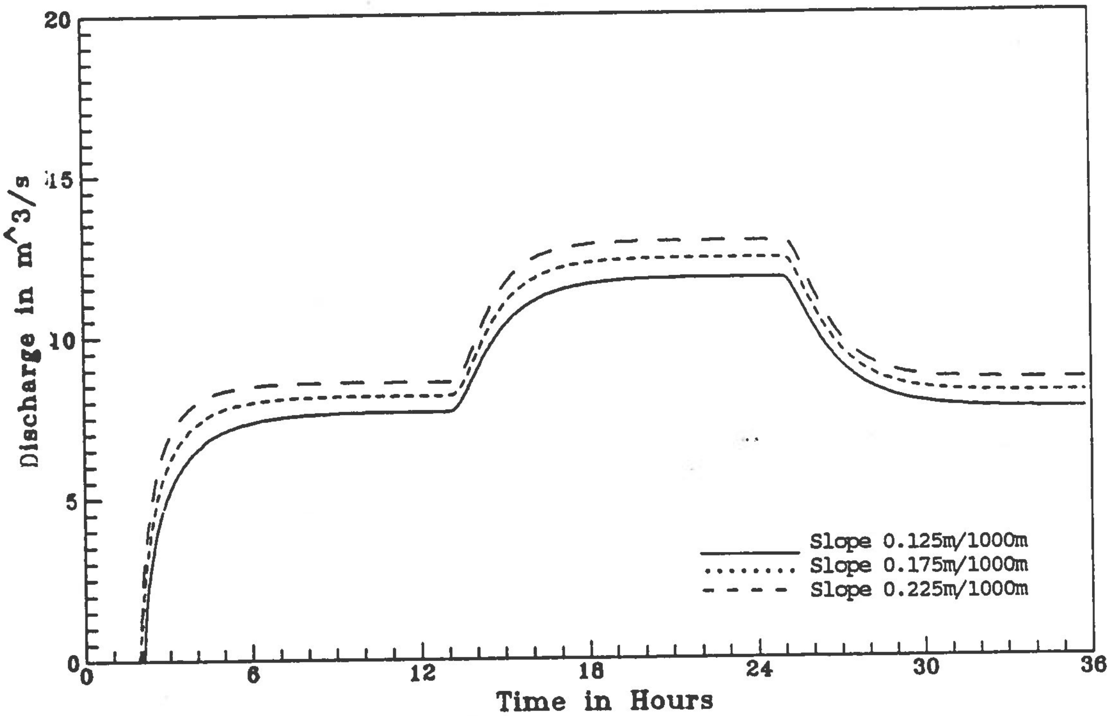

Figure 43. Downstream Supply Hydrograph as a Function of Longitudinal Slope.

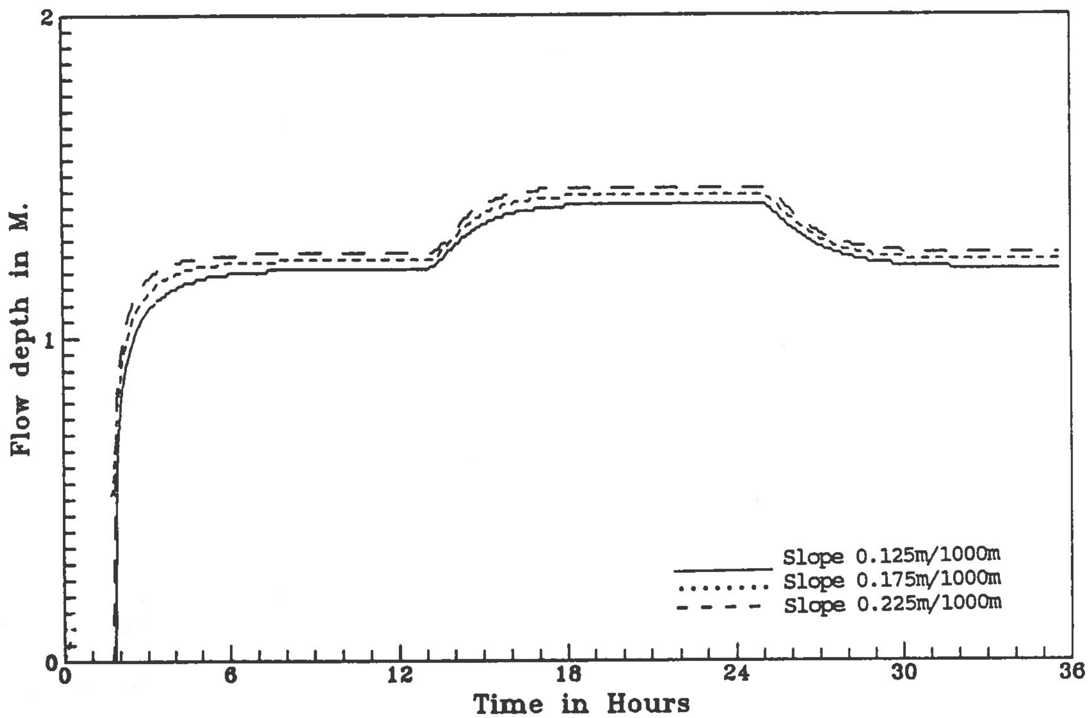

Figure 44. Downstream Flow Depth as a Function of Longitudinal Slope.

TABLE 13. Percent Distribution of Inflow Volume for Different Longitudinal Slope.

	Slope in m/1000m		
	0.125	0.175	0.225
Reach Storage	7.56	7.02	6.68
Reach outflow	59.49	63.42	66.71
Turnout 1	8.55	7.99	7.52
Turnout 2	9.85	8.43	7.36
Turnout 3	7.64	7.40	7.12
Turnout 4	6.91	5.74	4.61

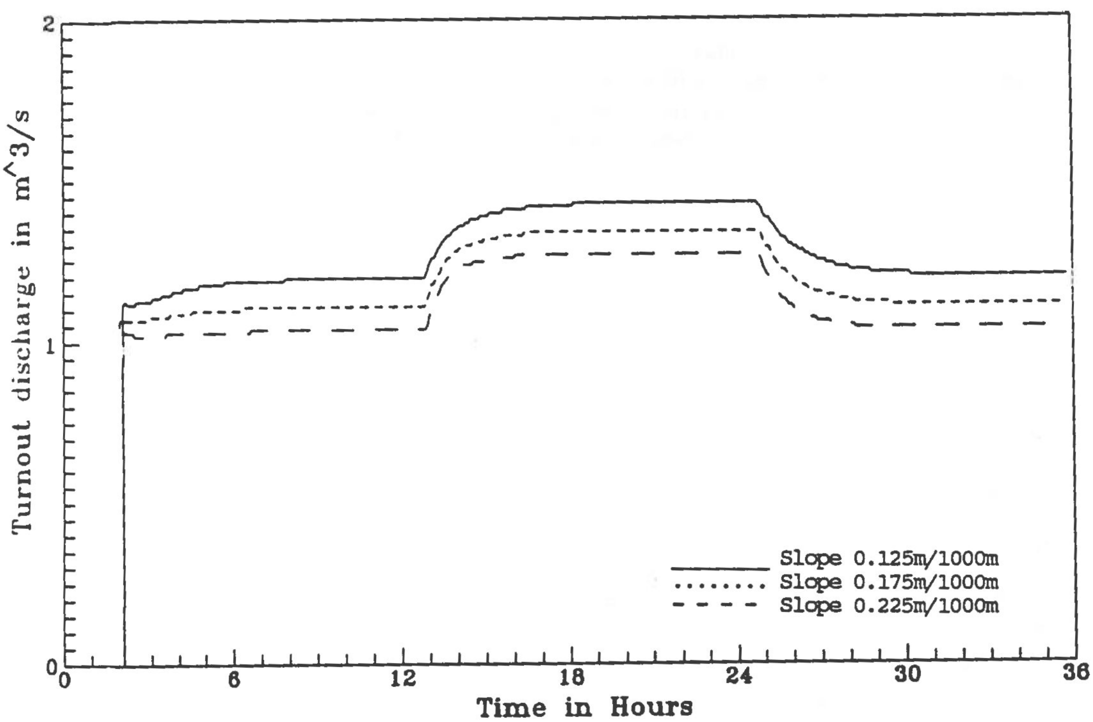

Figure 45. Turnout #1 Supply Hydrograph as a Function of Longitudinal Slope.

Manning's n. Manning's roughness coefficient was varied from 0.007 to 0.021 by a step of 0.007. The results in Table 14 indicate that although the reach storage increased indicating an increase in flow depth, the reach outflow decreased due to the reduction in flow velocity attributed to a high Manning's n (Fig. 46). The flow depth increase was mainly at the upstream end. The lower end experienced a decrease in flow depth as shown on Fig. 47. The increase in flow depth benefited the turnouts in the reach, where flow delivery increased (see Fig. 48).

Channel roughness coefficient is affected by many interdependent factors, the most salient ones being (Chow, 1950):

1. Surface roughness;
2. Height, density, distribution and type of vegetation on the channel;
3. Irregulaties in cross section (size and shapes) along the reach length;
4. Channel alignment;
5. Extent and nature of silting and scouring of the reach;
6. Nature, shape, size, number and distribution of physical obstructions;
7. Stage and discharge; and
8. Seasonal change.

All these factors makes it very difficult to make an accurate determine of its value. However, as illustrated in the discussion above a large error in estimation of this parameter can significantly affect the validity of the model results.

TABLE 14. Percent Distribution of Inflow Volume for Different Manning's n.

		Manning's n		
		0.007	0.014	0.021
Reach Storage		7.16	7.56	8.59
Reach outflow		64.27	59.49	54.21
Turnout	1	7.46	8.55	9.32
Turnout	2	7.13	9.85	12.41
Turnout	3	7.63	7.64	7.70
Turnout	4	6.35	6.91	7.77

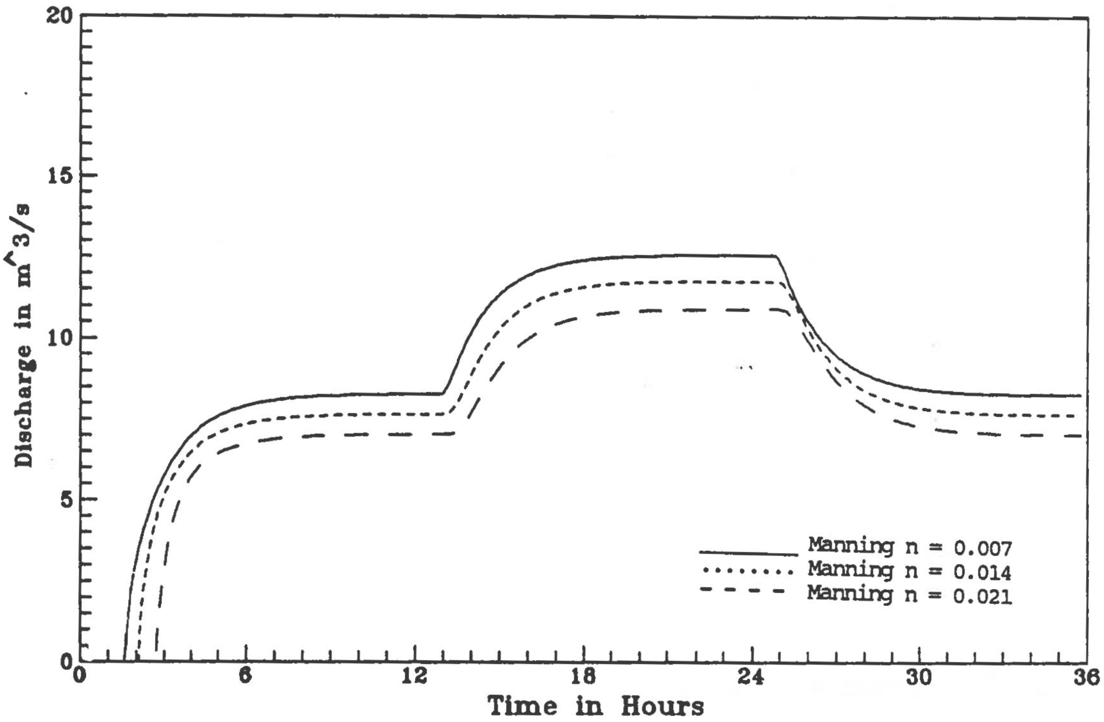

Figure 46. Downstream Supply Hydrograph as a Function of Manning's n.

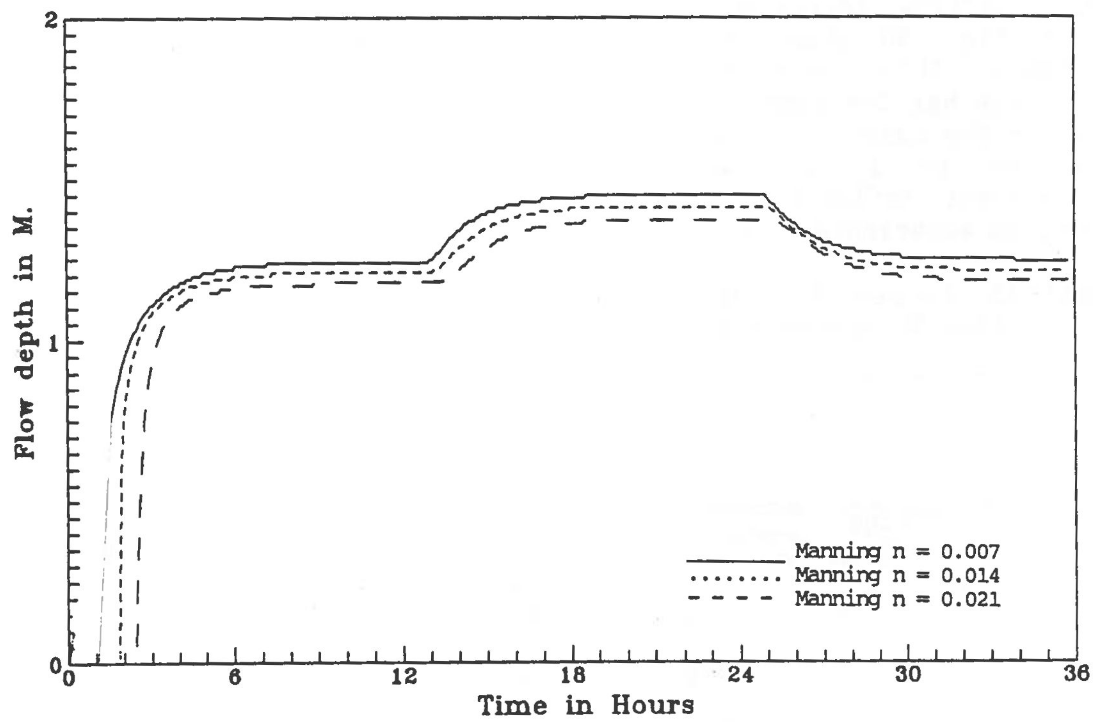

Figure 47. Downstream Flow Depth as a Function of Manning's n

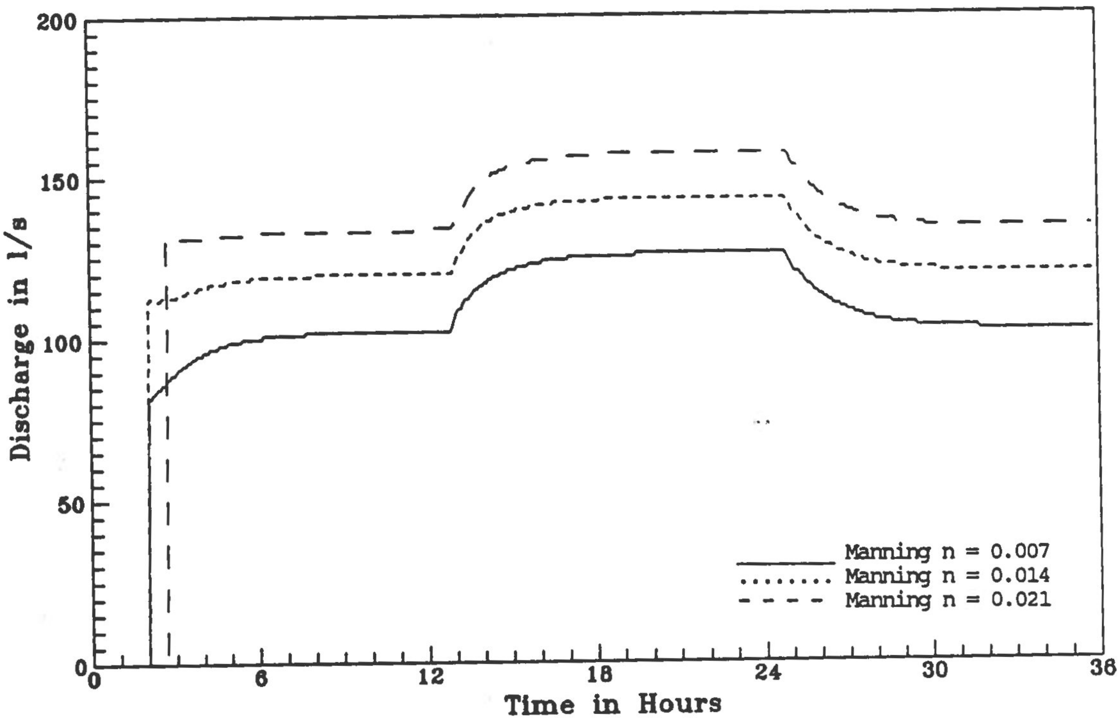

Figure 48. Turnout #1 Supply Hydrograph as a Function of Manning's n.

Discharge coefficient of inline control structure. The discharge coefficients used were 1.5, 1.86, and 2.10. Figure 49 shows that the reach outflow increased as the coefficient of discharge increased, while Fig. 50 shows a decrease in flow depth at the downstream boundary. This indicates that the increase in the coefficient of discharge has the same effect as lowering the sill height, because just like in the case of a lower sill height, high coefficients of discharge resulted in a drawdown flow profile. Increasing the discharge coefficient influenced the flow profile upstream such that all the turnouts experienced a reduction in flow delivered (see Table 15).

TABLE 15. Percent Distribution of Inflow Volume as a Function of In-line Structure's Discharge Coefficient.

		Coefficient of Discharge		
		1.500	1.860	2.100
Reach Storage		7.96	7.56	7.34
Reach outflow		56.67	59.49	60.84
Turnout	1	8.71	8.55	8.48
Turnout	2	10.29	9.85	9.66
Turnout	3	7.94	7.64	7.49
Turnout	4	8.44	6.91	6.18

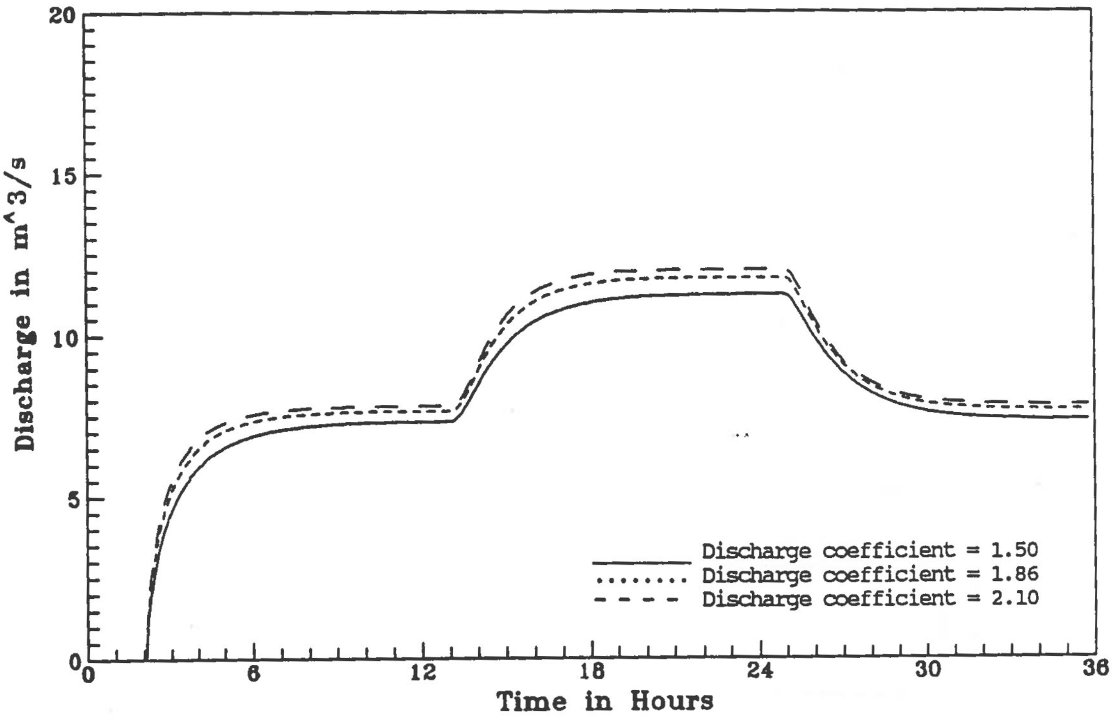

Figure 49. Downstream Supply Hydrograph as a Function of Inline Structure's Discharge Coefficient.

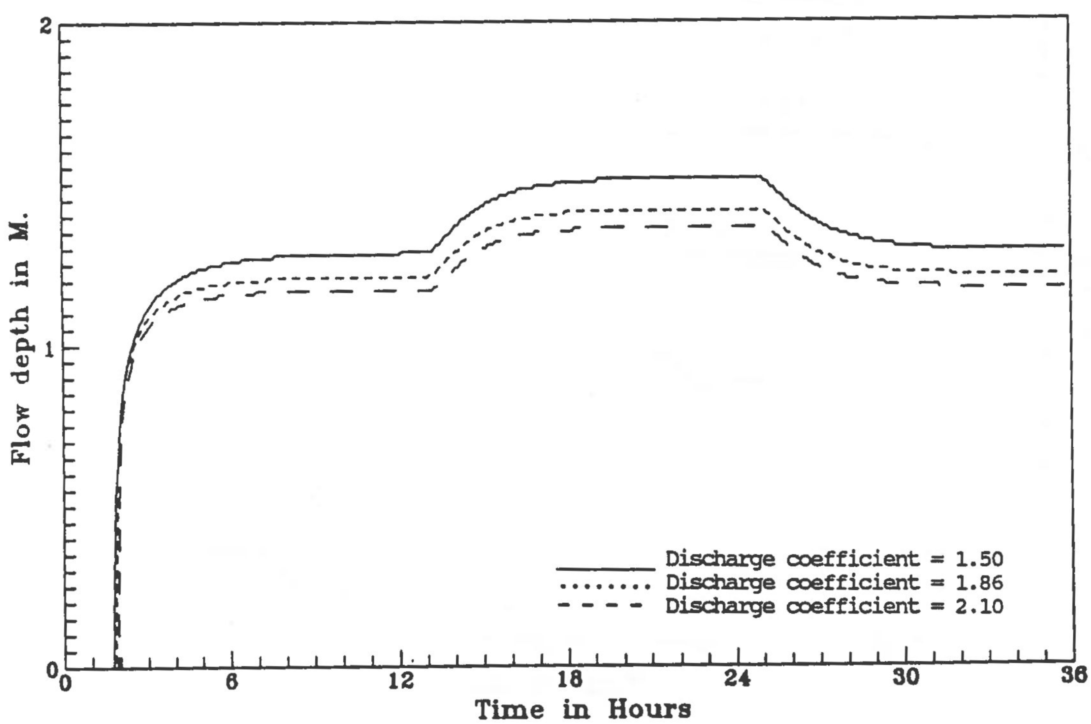

Figure 50. Downstream Flow Depth as a Function of Inline Structure's Discharge Coefficient.

Discharge coefficient of a turnout structure . The discharge coefficient used for the orifice turnout structure (turnout #1), were 0.4, 0.6, and 0.8. Figure 51 shows that the discharge changed significantly as the coefficient of discharge is altered. Taking 0.6 as the standard value, the results on Table 16, indicate that under or over estimating by 33 percent results in approximately 32 percent reduction or increase in delivery.

TABLE 16 Turnout #1 Flow rate as a Function of the Discharge Coefficient.

Coefficient of discharge	Discharge in l/s	
	Qin = 12 M^3/s	Qin = 18 m^3/s
0.4	81.0	96.0
0.6	120.0	143.0
0.8	157	189.0

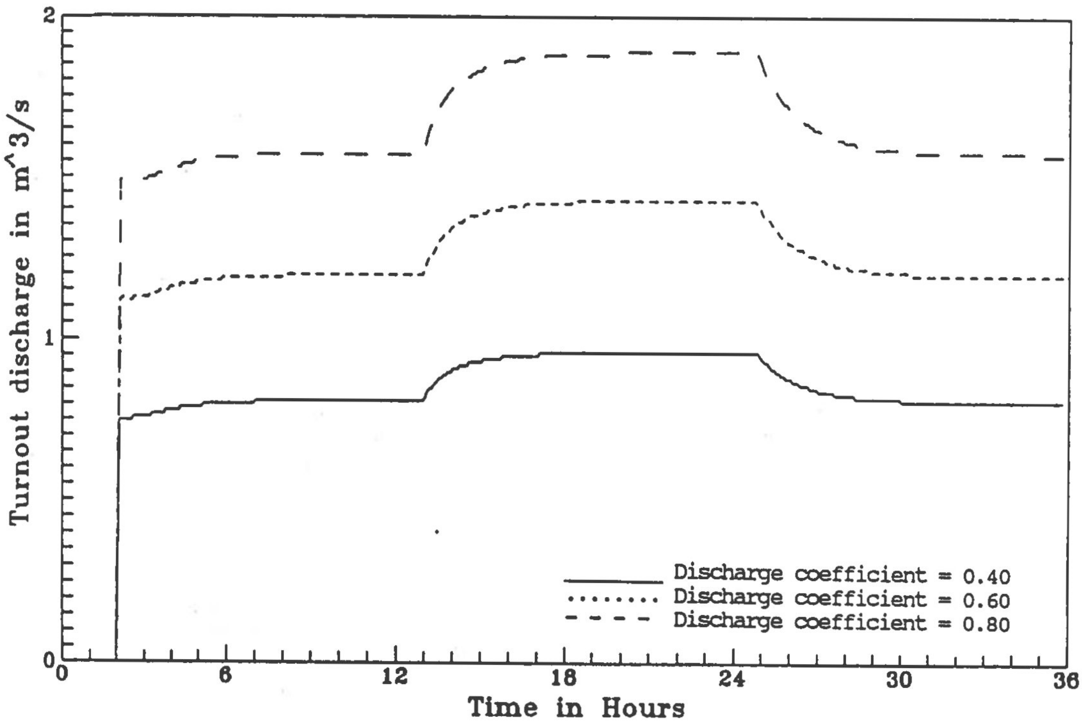

Figure 51. Turnout #1 Supply Hydrograph as a Function of the Coefficient of Discharge.

Use of CAHSM in Real Time Management

As was discussed in Chapter II, the conveyance system is a complex setup that is influenced by the natural (physical and biological) and human factors. Before embarking on the use of the model in the improvement of the system performance. The following questions should be addressed:

1. What are the existing procedures, and could better results be achieved by using other procedures which are more appropriate to the local needs?
2. What procedures are socially, economically, financially, and technically feasible? and
3. What resistance to changes should we expect?

The CAHSM model can be used for real time management of irrigation canal networks that experience frequent canal transients. The benefits of using the model stem from the models capability to:

1. Process the overall data and generate a comprehensive control schedule;
2. Incorporate farmers' demands making it possible to deliver water on demand without excessive seepage and operational losses;
3. Use stored past simulation or generate a steady state start-up condition and compute flow propagation in order to detect possible disturbances, rapidly and accurately, and devise strategies to alleviate the disturbances; and
4. Generate water control strategies for specific contexts, such as: (a) the beginning and end of season, (b) shortage periods, (c) reach closure, (d) and control structure failure, etc.

To take full advantage of the model's capabilities, the working environment must be conductive to sustained computerized irrigation management (Deart G., 1985). Control of this dynamic system requires:

1. Physical structures that facilitate conveyance, flow measurement, and that are easy to operate;
2. Data acquisition systems that collect accurate and timely data on the system's operation and users' demands (ditch riders, extensions agents, etc.);
3. Data processing systems, consisting of engineers, computers, and calculators, that will quickly and accurately analyze incoming data and issue appropriate operating instructions;
4. A system to implement the operating instructions accurately and without bias (ditch riders); and
5. A fast and accurate communication link between the data acquisition, data processing, and the execution of the operating instructions.

Specific items of concern to the viability of computerized conveyance system management are:

1. Will the hardware and software work reliably in the environment;
2. What contingency plans should be made in case of hardware failure;
3. Will there be qualified computer operators and programmers available to make modifications to suit the changing physical and operating settings;
4. Is sufficient and accurate data available, and if not, could missing data be obtained at a reasonable cost;
5. Can a reliable communication between model operators and field personnel be established and sustained; and
6. Would the results of the model be used regularly or would model application be abandoned in favor of the old and easy operation and management routine?

In applying the model, the following activities are proposed:

1. Periodic and careful re-calibration of the model to take care of any changes in the physical system;
2. Collection of adequate and accurate data;
3. Daily simulation to develop water delivery procedures and plans that reflect current water users' demand and climatic conditions;
4. Comparison of the planned system operation with actual system operation so that appropriate initial conditions are used at the beginning of each simulation period; and
5. Monitoring deviations between simulated and actual performance so that the necessary improvements in model calibration and/or system operation can be identified and corrected.

CHAPTER VII

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

This study provide a tool that can be used to evaluate the hydraulic response of the conveyance network to changes in physical and operational features of the system and determine the best combination of controllable parameters to achieve pre-decided system performance goals. The mathematical model developed is based on solving the integrated form of Saint Venant equations which describe steady or unsteady, uniform or non-uniform flow regimes of sub-critical flow in open-channels. This approach makes it possible to simulate canal filling and draining phases and handle bulk lateral outflow or inflow into the section being modelled. The analysis is done for each reach in the system. The outflow of a reach becomes the inflow of the reach downstream, thereby making simulation of a branching canal network possible.

The model has three water control options. The first is "Operator Decision Control" in which the model user decides on what inflows and control structure settings should be implemented. In the other two options, the program generates the control structure settings that are required to achieve (1) upstream flow depth control and (2) downstream discharge control. In the "Upstream Flow Depth Control", the objective is to minimize flow depth fluctuations in the reach thereby maintaining a constant turnout discharge for a particular structure setting. "Downstream Discharge Control" gives preference to the tail end users. In this case the analysis starts at the downstream end, releasing into last reach a flow rate equal to the sum of all its outflows and systematically working upstream until the system inflow at the headwork is determined. This is done for every time step.

To make the model readily usable in different canal systems, a wide range of water control structures options have been included. Free and submerged flow conditions are considered. For unique structures found in specific projects but not included in the original software, directions on how to write a FORTRAN subroutine and link it with the existing routines will be provided.

A lot of effort has been devoted to the improvement of the man-machine interface. The program is menu driven, it has a build in "help" explanation that can be called at any point in the simulation process, traps data entry errors and has a graphical display of input and simulation status at any point. Interactive simulation is provided to enable the user to pause simulation and critically examine the simulation status. Adverse conditions that may develop during simulation can therefore be evaluated and modified just like a canal operator would react in case of a real emergency.

Conclusions

The model developed represents a unique set of integrated modules that can be used to better assess the reality in dealing with flow conditions prevailing in canals with the aim to identify constraints and opportunities to increase manageability of the system. CAHSM model highlights are:

1. The model simulates closely the behavior of existing canal networks making it acceptable by operating staff.
2. The model input, output, and operation meets the needs of different categories of users.
3. The computer program optimizes the computation and memory allocation giving the software the highest possible level of simulation performance on microcomputers
4. The computer program has state-of-the-art algorithms and modules that prevent hydraulic simulation errors, numerical instability, and divergence of the solution.

The model's application to: (1) establishing and locating basic improvements to the system; (2) determining points in the system that are sensitive to managerial interference; (3) predicting system behavior; and (4) establishing procedures for better water control are demonstrated in chapter IV. However, successful application of the model requires an integration of hydraulic science and practical skills and knowledge gained during the operation and management to better address broader issues in design, operation, and management of the system.

Recommendations

Model development and refinement is a continuing process. This model, therefore, serves as a platform upon which enhancements can be made. Specific areas requiring additional research are:

1. Further testing and model refinement to adopt the model to an even wider range of water conveyance operational scenarios;
2. Application of the model to non-prismatic canals and/or spatially dependent Manning's friction coefficient;
3. Incorporate the speed of gate movement in the gate stroking algorithms;
4. Incorporate an algorithm that explicitly handles inverted siphons, culverts, and constrictions;
5. Application of the model to computerized control of the conveyance system;

6. Additional sensitivity analyses to examine the impact of under and/or overestimation of model parameters on the reliability of water supply; and
7. Application of the model in evaluating the performance of the conveyance system.

REFERENCES

Aisenbrey, A.J. 1983. "General requirements and design considerations," Chapter 1. Design of small canal structures, U.S. Department of the Interior, Bureau of Reclamation, Denver, Co.

Amorocho. J., and Strelkoff, T. 1965. "Hydraulic transients in the California aqueduct," Water Science and Engineering Paper, 1008, Department of Water Science and Engineering, University of California, Davis, California, June.

Anderson, J.R. and Dent J. B. 1971 "Agricultural systems analysis: Retrospects and prospects," Chapter 17. Systems analysis in agricultural management, John Wiley and Sons, New York.

Bachmat, Y., Andrews, B., Holtz, D., and Sebastian, S. 1978. "Utilization of numerical groundwater models for water resource management," U.S. Environmental Protection Agency, Robert S. Kerr Environmental Research Laboratory, Ada, OK, June, pp. 178.

Biswas A. K. 1975. "Systems approach to water management," ICID Bulletin, Central Electric Press, New Dehli, January, pp. 12-19.

Bottrall, A. 1981. "Improving canal management: The role of evaluation and action research," Water Supply and Management, Vol. 5, No. 1, pp. 67-79.

Brater E.F. and King H.W. 1972. "Handbook of hydraulics for the solution of engineering problems," McGraw-Hill Book Company, New York.

Bredehoeft J. 1984. "Physical limitation of water resources," Chapter 1, Water Scarcity: Impacts on Western Agriculture, E. A. Englelbert and A. F. Scheuring eds., Berkeley, University of California Press.

Burt, C.M. 1983. "Regulation of sloping canals by automatic downstream controls," dissertation presented to Utah State University, Logan, Utah, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Burt, C. M. 1984. "Canal automation for rapid demand deliveries," Proceedings, Specialty Conference of Irrigation and Drainage Division, American Society of Civil Engineers, Flagstaff, Arizona, pp. 502-509.

Burt, C. M. 1987. "Overview of canal control concepts," Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems, Specialty conference of irrigation and Drainage Division, ASCE, Portland, Oregon.

Buyalski C.P. and Serfozo E.A. 1979. Electronic filter level offset (EL-FLO) plus reset equipment for automatic downstream control of canals, USBR publication No. REC-ERC-79-3.

Chaudhry Y.M., and Contractor D.N. 1973. "Application of the implicit method to surges in open channels,' Water Resources Research, Vol. 9, No. 6, December.

Chevereau G. and Schwartz-Benezeth, S. 1987. "BIVAl system for downstream control," Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems, Specialty conference of irrigation and Drainage Division, ASCE, Portland, Oregon.

Chow, Ven Te. 1959. Open channel hydraulics. McGraw-Hill Book Company, Inc., New York.

Clemmens, A.J. 1983. "Fluctuations in irrigation water deliveries," Proceedings, ASCE Irrigation and drainage specialty conference, Jackson, WY, July 20-22, pp. 546-553.

Corriga, G., Patta, F., Sanna. S., and Usai, G. 1979. "A mathematical model for open-channel networks", Applied Mathematic Modelling, Vol. 3.

Deart, G. 1985. "Irrigation water management modeling in Sri Lanka," Report on use of computer models for water scheduling, in Gal Oya water management project.

Dewey H.G., and Madsen, W.R. 1976. "Flow control and transients in the California aqueduct", Journal of irrigation and drainage division, ASCE, Vol. 102, IR3, September.

Eisele H. 1988. "Modeling main system water delivery under various allocation procedures," dissertation presented to Utah State University, at Logan, Utah, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Falvey, H.T., and Luning P.C. 1979. "Gate Stroking," United States Bureau of Reclamation NO. REC-ERC-79-7.

Flammer G.H., Jeppson, R.W. and Keedy H.F. 1983. Fundamental pronciples and applications of fluid mechanics, Utah State University, Logan, UT

Framji, K. K. 1984. "Past and likely future developments (By 2000 AD) in irrigation, drainage, and flood control measures in developing countries," ICID Bulletin, Central Electric Press, New Dehli, July, pp. 1-58.

Fredeiksen H.D. 1969. "Design of California aqueduct", Journal of Irrigation and Drainage division, ASCE, Vol. 95, No. IR2, June.

Gientke, F. J. 1974. "Transient control in Lower Sacramento River," Journal of the Hydraulics Division, ASCE, Vol. 100 No. HY3.

Hamilton D. L., and DeVries J.J. 1986. "Microcomputer simulation of canal operation," Journal of Irrigation and Drainage Division, ASCE, Vol. 112, No. 3, August, pp. 264-273.

Horst L. 1987. "Choice of irrigation structures: The paradox of operational flexibility," paper presented at the irrigation design for management regional symposium, Topaz Hotel, Kandy, Sri Lanka.

Jeppson, R.W. 1986. "Class Notes CEE 645," Utah State University, Logan, Utah.

Joliffe I.B. 1984. "Computation of dynamic waves in channel networks," " Journal of the Hydraulics Division, ASCE, Vol. 110, No. 10, October, pp. 1358-1369.

Keller, A. A. 1987. "Modeling unit command area demand and response to water delivered by the main system," dissertation presented to Utah State University, at Logan, Utah, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Keller J. 1986. "Irrigation system management," Chapter 11, Irrigation management in developing countries, K. C. Nobe and R. K. Sampath, eds. Boulder, Colorado: Westview Press.

Kraatz A., and Mahajan D. 1975. "Small hydraulic structures," FAO Irrigation and Drainage Paper No. 26 Vol 1, Food and Agriculture Organization of the United Nations, Rome.

Manz, D. H. 1985. "Systems analysis of irrigation conveyance systems," dissertation presented to the University of Alberta, at Edmonton, Alberta, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Merriam, J. L. 1977. "Level-top canals for semi-automation of on-farm irrigation and supply systems," Proceedings of the irrigation and drainage division specialty conference, American Society of Civil Engineers, Reno, Nevada, pp. 217-224.

Mozayeny, B. and Song, C.S. 1969. "Propagation of flood waves in open channels," Journal of the Hydraulic Division, ASCE, Vol. 95, No. HY3, May, pp. 877-892.

O'Loughlin E.M. 1972. "Application of unsteady flow analysis to operation decisions in long aqueducts," Eigth congress of ICID.

Ponce V.M., Indlekofer H., and Simons D.B. 1978. "Convergence of four-point implicit water wave models," Journal of Hydraulic Division, ASCE, HY7, July.

Price R.K. 1974. "A comparison of four numerical methods for flood routing," Journal of the Hydraulic Division, ASCE, Vol. 100, No. HY7, July, pp. 879-899.

Rangeley, W. R. 1983. "Irrigation--Current trends and a future perspective," Seminar at World Bank, February.

Replogle J.A. et al., 1980. "Farm water delivery systems," Chapter 9 of ASAE monograph Design and operation of irrigation systems, p. 317-343.

Replogle, J. A. 1984. "Some environmental, engineering, and social impacts of irrigation water delivery schedules," Proceedings of the 12th congress of ICID, Question 36, Report 61, pp. 965-978.

Replogle, J. A. 1986. "Some tools and concepts for better irrigation water use," Chapter 5, Irrigation management in developing countries, K. C. Nobe and R. K. Sampath, eds. Boulder, Colorado: Westview Press.

Rogier D. Coeuret, C. and Bremond J., 1987. "Dynamic regulation of Canal de Provence," Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems, Specialty conference of irrigation and Drainage Division, ASCE, Portland, Oregon.

Skogerboe, G.V. 1977. "Water measurement and water management relationships," Proceedings of the ASCE irrigation and drainage specialty conference, Reno, Nevada, July 20-22, pp. 295-304.

Skogerboe, T., Shafique M.S., and Gandarillas C. 1986. "Field calibration of irrigation structures for discharge measurement," International Irrigation Center, Utah Stated University, Logan, Utah.

Streeter V.L., 1967. "Valve stroking for complex piping systems," Journal of Hydraulics Division, ASCE, Vol. 93. No. HY3.

Strelkoff, T. 1969. "The one dimensional equations of open channel flow," Journal of the Hydraulics Division, ASCE, Vol. 95, No. HY3, May, pp. 861-876.

Strelkoff, T. 1970. "Numerical solution of the Saint-Venant equations," Journal of the Hydraulics Division, ASCE, Vol. 96, No. HY1, January, pp. 223-252.

Tzou C. snd Rodriguez L. 1988. "Field data for verification and calibration of the USU hydraulic model," A field report presented to Utah State University, International Irrigation Centre, at

Logan, Utah.

U.S. Department of the Interior, Bureau of Reclamation and Bureau of Indian Affairs. 1978. "Report on the water conservation opportunities study," Bureau of Reclamation, Denver, CO.

Walker R.E. 1987. "Long crested weirs," Planning, Operation, Rehabilitation and Automation of Irrigation Water Delivery Systems, Specialty conference of irrigation and Drainage Division, ASCE, Portland, Oregon.

Walker, W.R., and Skogerboe, G.V. 1987. The theory and practice of surface irrigation, Prentice-Hall, Englewood Cliffs, NJ.

Walker, W.R., and Skogerboe, G.V. 1986. "Improving provincial irrigation departments' water management practices through the implementation of computer-based operations and data processing," Seminar on computerized irrigation system management," Presented in Lahore, Pakistan, July 9th.

Walker, W. R. 1986. "Class Notes AIE 760," Utah State University, Logan, Utah.

Wylie E.B. 1969 "Control of transient-free surface flow," Journal of Hydraulics Division, ASCE, Vol. 95, No. HY1, January, pp. 347-361.

Zimbelman, D.D. 1981. "Computerized control of an open-channel water distribution system," dissertation presented to Arizona State University, at Tempe, Arizona, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Zimbelman, D.D, and Bedworth D.D. 1983. "Computer control for irrigation canal system," Journal of Irrigation and Drainage Division, ASCE, Vol. 109, No. 1, March.

WATER MANAGEMENT SYNTHESIS PROJECT REPORTS

WMS 1 Irrigation Projects Document Review

Executive Summary
Appendix A: The Indian Subcontinent
Appendix B: East Asia
Appendix C: Near East and Africa
Appendix D: Central and South America

WMS 2 Nepal/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 3 Bangladesh/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 4 Pakistan/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 5 Thailand/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 6 India/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 7 General Asian Overview

WMS 8 Command Area Development Authorities for Improved Water Management

WMS 9 Senegal/USAID: Project Review for Bakel Small Irrigated Perimeters Project No. 685-0208

WMS 10 Sri Lanka/USAID: Evaluation Review of the Water Management Project No. 383-0057

WMS 11 Sri Lanka/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 12 Ecuador/USAID: Irrigation Sector Review

WMS 13 Maintenance Plan for the Lam Nam Oon Irrigation System in Northeast Thailand

WMS 14 Peru/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 15 Diagnostic Analysis of Five Deep Tubewell Irrigation Systems in Joydebpur, Bangladesh

WMS 16 System H of the Mahaweli Development Project, Sri Lanka: 1980 Diagnostic Analysis

WMS 17 Diagnostic Analysis of Farm Irrigation Systems on the Gambhiri Irrigation Project, Rajasthan, India: Volumes I-V

WMS 18 Diagnostic Analysis of Farm Irrigation in the Mahi-Kadana Irrigation Project, Gujarat, India

WMS 19 The Rajangana Irrigation Scheme, Sri Lanka: 1982 Diagnostic Analysis

WMS 20 System H of the Mahaweli Development Project, Sri Lanka: 1983 Diagnostic Analysis

WMS 21 Haiti/USAID: Evaluation of the Irrigation Component of the Integrated Agricultural Development Project No. 521-0078

WMS 22 Synthesis of Lessons Learned for Rapid Appraisal of Irrigation Strategies

WMS 23 Tanzania/USAID: Rapid Mini Appraisal of Irrigation Development Options and Investment Strategies

WMS 24 Tanzania/USAID: Assessment of Rift Valley Pilot Rice Project and Recommendations for Follow-On Activities

WMS 25 Interdisciplinary Diagnostic Analysis of a Work Plan for the Dahod Tank Irrigation Project, Madhya Pradesh, India

WMS 26 Prospects for Small-Scale Irrigation Development in the Sahel

WMS 27 Improving Policies and Programs for the Development of Small-Scale Irrigation Systems

WMS 28 Selected Alternatives for Irrigated Agricultural Development in Azua Valley, Dominican Republic

WMS 29 Evaluation of Project No. 519-0184, USAID/El Salvador, Office of Small-Scale Irrigation - Small Farm Irrigation Systems Project

WMS 30 Review of Irrigation Facilities, Operation and Maintenance for Jordan Valley Authority

WMS 31 Training Consultancy Report: Irrigation Management and Training Program

WMS 32 Small-Scale Development: Indonesia/USAID

WMS 33 Irrigation Systems Management Project Design Report: Sri Lanka

WMS 34 Community Participation and Local Organization for Small-Scale Irrigation

WMS 35 Irrigation Sector Strategy Review: USAID/India; with Appendices, Volumes I and II (3 volumes)

WMS 36 Irrigation Sector Assessment: USAID/Haiti

WMS 37 African Irrigation Overview: Summary; Main Report; An Annotated Bibliography (3 volumes)

WMS 38 Diagnostic Analysis of Sirsia Irrigation System, Nepal

WMS 39 Small-Scale Irrigation: Design Issues and Government Assisted Systems

WMS 40 Watering the Shamba: Current Public and Private Sector Activities for Small-Scale Irrigation Development --

WMS 41 Strategies for Irrigation Development: Chad/USAID

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