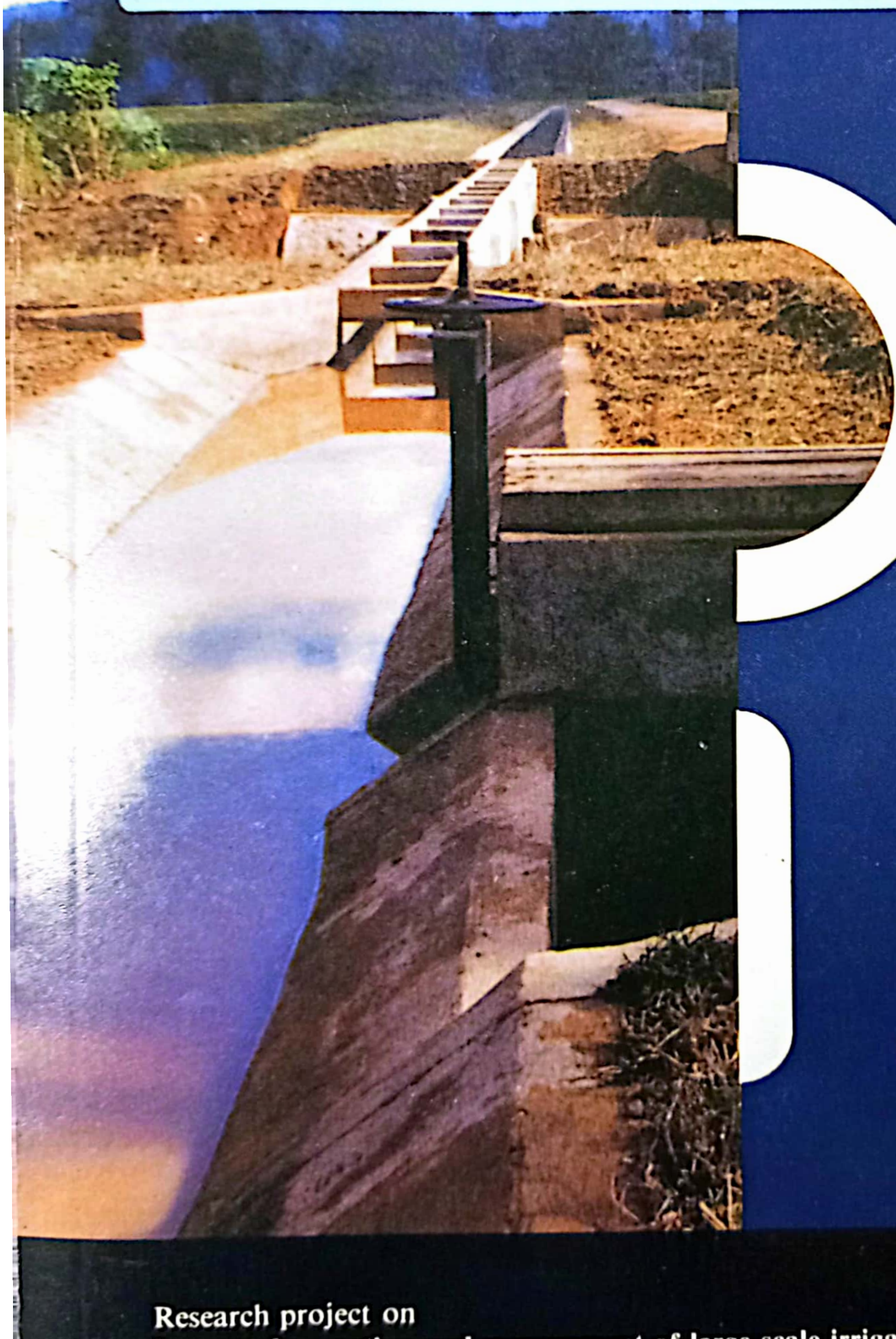
MATHEMATICAL MODELING OF IRRIGATION CANAL SYSTEMS

STATE OF THE ART



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Research project on
Improved operation and management of large scale irrigation systems
Monograph No. 2
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Improved operation and management of large scale irrigation systems

1. Improvement of irrigation system operation
2. Mathematical modeling of irrigation canal systems: State of the art

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Loof, R., Paudyal, G.N. and Manguerra, H.B., 1991, Mathematical modeling of irrigation canal systems: State of the art, Improved operation and management of large scale irrigation canal systems, Monograph 2, Asian Institute of Technology, Bangkok, Thailand.

ISBN 974-8201-104

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FOREWORD

The increasing importance of mathematical modeling in the operation and management of irrigation systems has inspired the authors to write this monograph. The monograph primarily collates and summarizes the more relevant literature of the past three decades and presents them in an integrated form giving special emphasis to the appropriateness, present state and directions of the art in particular to the South and Southeast Asian setting. The attributes and capabilities of a model-software appropriate to the requirements of a typical irrigation system are defined. This has been partly based on a comparative description of existing model-softwares. Overall, it is anticipated that this report will become an important reference with substantial knowledge-base for any mathematical modeling study of an irrigation canal system. Not only academics can benefit from this, but also irrigation practitioners and managers as well as those of other sectors who are similarly concerned with the improvement of performance of irrigation systems.

The report is the second in a series of monographs produced under the project "Improved operation and management of large scale irrigation projects" and the authors would like to thank GTZ of Germany and Asian Institute of Technology for providing financial assistance. They would also like to take this opportunity to express their sincere appreciation to the various officials of the Royal Irrigation Department of Thailand particularly to the officials of the Phitsanulok Irrigation Project for their continuous support.

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INTRODUCTION

Many Third World countries have set irrigation as a priority area for development. Aside from providing one of the most basic human requirements it serves as a vehicle for meeting diverse development objectives. However, in irrigation schemes of the developing world where rotational irrigation is widely practiced, the overall efficiencies range from 30-35 percent only, whereas efficiencies of 50 percent or larger are achievable (Sanmuganathan and Bolton, 1988; Maurya and Kuzniar, 1988). This is most unfortunate since the impact of unsuccessful irrigation is most sharply felt by already disadvantaged Third World farmers who along with entire rural communities depend heavily on reliable irrigation for their livelihood and prospects of advancement (IIMI, 1989). The situation becomes more delicate with the world's population expected to double by the middle of the next century while the supply of available land and water remains limited.

Unless a methodology is developed which will guarantee the improvement of the performance of irrigation systems in this region, which comprises more than 70 percent of the world's total irrigated area, global food production will continue to remain below its potential. Despite the fact that billions of dollars have been invested in new irrigation schemes and in restoring existing ones, what turns out to be lacking always is the proper management of such systems to sustain their designed performance.

Searching for the right approach

Irrigation in the past has been considered more of an art than of a science. The implementation of the operational requirements of a gravity irrigation system for example is dictated largely by the experience and discretion of the irrigation operator and not by rules from textbooks. For example, in the case of Sri Lanka, theoretically, at the higher level of management, an administrative mode is envisaged which demands a verbatim implementation of such rules. However, the actual irrigation management at the field level is transformed into an entrepreneurial mode of management which involves the manipulation of the rules in response to changing conditions and opportunities (Colmey, 1988). With the emerging clamor for more efficient and effective utilization of irrigation water, a set of rules and guidelines has to be enforced and strictly followed. While the significance of injecting practical rule of thumbs is recognized, these rules and guidelines have to be founded mainly on the basis of science.

The identification of the problems precedes all these. In the development of a tool for irrigation management, the problem should be approached directly and not be lost in too many generalities. For many years, strategies for improving the overall performance of irrigation have been classified into two schools of thought: i) improvement at the tertiary level; and ii) improvement of the main system.

The tertiary level has received the bigger share in irrigation management research over the past decades. Studies have concentrated on tertiary water distribution or on-farm water management at the point where water is delivered to the end-user. This trend may have been brought about by the assumptions that main systems are performing well within their design capacities, that farmers are poor water managers and thus are wasting a lot of water, and that on-farm facilities are usually in a mediocre state and thus require a lot of rehabilitation. It may be added that the immediate need to satisfy the farmers is more pressing and the research techniques involved do not disturb the farmers' activities a lot.

However, it soon became clear that these on-farm activities will only be effective when water is available at the farm turnout in the right amount, at the right time and for the right duration, which is not always the case, particularly for main systems which are performing way below their expected level. In the Philippines for example, several researchers on water management have proven that problems of water distribution are greater in lateral and sublateral canals than at the farm level. It was their conclusion that a prerequisite for further improvement in terminal level operation is the equitable and dependable flow of water in the main system, which requires better operation and maintenance (Lazaro et al.,1979; Taylor and Wickham, 1979; Wickham and Valera, 1979; ADB, 1980).

The greater need therefore is to improve the performance of the main system, which will have a greater impact over the whole system. The development of an operation and management plan for any possible operational scenario for the main system thus becomes imperative.

Mathematical model as a tool

A cost-effective method is to use a mathematical model to simulate canal flow behavior. Better understanding of the canal behavior provides unlimited opportunities to improve its performance. This could be in the form of predicting the operational constraints of a system while still at its design stage; evaluating operational performance of irrigation plans before actual implementation; developing rules and procedures for better management under different scenarios; as an operational aid for real-time canal operation; and as a tool for training system operators. Moreover, the usefulness of a simulation model is expected to increase as canal systems are required to make deliveries on demand and as the efficient delivery of water becomes important.

The theoretical foundation of modeling steady or unsteady flow in open channels has been well established for several years already. The proliferation of computers and the improvement in numerical schemes have further enhanced its practical application. Moreover, the usefulness of such models are expected to increase as canal systems are required to make deliveries on demand and as the efficient delivery of water becomes more important.

HISTORICAL DEVELOPMENT OF CONCEPTS

Precursors

As a result of the work of Castelli, the equation of continuity for steady flow was established as a basic equation of open channel hydraulics about the year 1660. It took another 200 years for the equation of continuity for unsteady flow to be established on as a firm basis by Saint-Venant in 1871. From then onwards, concepts in open channels have been developed at an increasing pace. By the end of the 19th century, the concepts necessary for the understanding, design and management of water flow in rivers or canals were almost fully understood and documented. These include i) principle of continuity; ii) notion of steady uniform flow; iii) special features of flow in alluvial channel; iv) nature of steady non uniform flow; and v) variety of types of unsteady flow. The year 1935 offers a suitable point for distinguishing between the historical development of these concepts and the more modern evolution of topics and methods (Dooge,J.C.I., 1987).

Research on open channel hydraulics during the past three decades has been unanimously directed towards the complete understanding of the dynamics of transient conditions in open channels. This paved the way for the evolution of new experimental techniques and improved hydrologic instrumentation which with the advances in computers permitted rapid output of results. The emphasis on numerical schemes and computational open channel hydraulics has been so strong that experimental tests are relegated to just supplying the necessary inputs for numerical models and serving as guides for their development. These numerical simulation schemes which were in the past often limited by simplifying assumptions because of deficient and inefficient methods can now be formulated based on pure hydraulic considerations which can then provide more comprehensive flow information. This has stimulated the need for rapid, economic and efficient techniques to compile and appraise prototype data and model results. As Schaffranek (1987) mentioned, it is insufficient for a numerical scheme to be developed merely to the state of being a model program. To achieve a state of usefulness as an operationally oriented investigative tool, the model program must be supported by a comprehensive user-oriented computer data system and must provide a ready means of presenting output results in varied graphical forms.

Today, the development of user-oriented and field applicable models in ready-to-use forms is the norm. For real-time application in irrigation operation, modeling of single channels has been superseded by more complex efforts such as modeling of the whole irrigation network.

The mathematical models

Various mathematical models (empirical, linearization, hydrologic, hydraulic) for approximating the solution of Saint-Venant equations have been used in the past. Among these models, the complete hydraulic model is found to be the most relevant for solving unsteady flow problems in canals (Baltzer and Lai, 1968). This modeling technique includes the Method of Characteristic (MOC), the Finite Difference Method (FDM) and the Finite Element Method (FEM).

A table showing the attributes (poor, fair, good, excellent) of these methods in terms of some modeling properties was prepared by Ligget (1987). In Table 1, the first column indicates the method. The second column indicates the relative programming ease. The third column shows how well the method can treat problem geometry. The ability to handle large problems is indicated in the fourth column. The next column signifies the ability to simulate a varied physical process. Under universal programs is indicated the generality of the software. The quality of the program-user interface is shown in the seventh column. Program efficiency is shown in the last column.

Table 1. Methods of solution and their attributes.

Method	Program Ease	Geomet. Rep.	Problem Size	Physics	Univ. Program	User Friendly	Effi-ciency
FDM	good	poor	poor	fair	poor	poor	good
FEM	fair	good	poor	fair	good	fair	poor
MOC	fair	excellent	fair	poor	good	good	good

Source: Ligget, J.A., 1987.

The FEM has not gained widespread application in the field of open channel hydraulics because while it involves a complex algorithm it fails to show any significant advantage over MOC or FDM. Keuning (1976), however, had shown that a rather complicated problem may after all be solved easily by means of FEM. He used the principle of sectional linearization, i.e., nonlinear terms are linearized in every element to avoid the extensive calculation involved in modeling the complete, nonstationary, and nonlinear equations for one-dimensional open channel flow. Katopodes and Wu (1986) used the method for the computation of discontinuous and supercritical flow. The method, however, is somewhat limited with respect to stability requirements. Courant number must be less than 0.58 for fourth order spatial accuracy or less than 1.0 for second order spatial accuracy. The method must remain second order accurate in time in order to be stable at all.

Between MOC and FDM, the latter has been used more extensively. Although MOC can provide solutions as close as the theoretical solution, its use has been limited to special cases such as for solving boundary conditions. In some instances, it is used as a standard for comparison to check the validity of simpler but less accurate numerical schemes. The method is fundamental and appealing to mathematicians but there remain several difficulties which have prevented its wide use for unsteady flow in open channels (Ligget and Cunge, 1975). The FDM has been found the more appropriate method (Cunge, 1980).

MOC involves the conversion of the original system of the Saint-Venant partial differential equations into an equivalent system consisting of four ordinary differential equations. The fundamental principle regarding its existence, uniqueness, methods of numerical solution and resultant solution was repeatedly reviewed and elaborated by several early workers such as Abbot and Verwey (1970), Strelkoff (1970) and Wylie (1970). Wylie (1969) applied the method to a single channel pool with the objective of finding a predetermined gate operation which will minimize the time required for the pool to return to steady flow conditions without exceeding specified depth limitations or a certain rate of water level fluctuations.

The MOC was likewise used by Lai (1976) to represent one-dimensional, unsteady open channel flow by a set of quasi-linear partial differential equations. A numerical scheme based upon specified time intervals (STI) was employed. He claimed that computer programs which can correctly simulate unsteady flow in various types of waterways and water bodies are written mainly based on MOC. The MOC was also employed by Bodly and Wylie (1978) in calculating the required gate motion based on the concept of gate stroking which will allow a desired change in channel demand to be actuated in a prescribed manner with no residual disturbance existing in the channel when the gate motion is completed. The particular advantage of using this method is that the characteristic lines generated by the solution are essentially the paths, in the x-t plane, of surface disturbances. This knowledge can be most valuable in interpreting solutions.

Many investigators have been continuously improving the MOC to be more viable and useful. Some extended the characteristics outward in distance (Chang and Richards, 1971; Vardy, 1977;, Wiggert and Sundquist, 1977); some extended them backward in time (Wylie, 1980; Goldberg and Wylie, 1983); each with an appropriate interpolation scheme, and each with its accompanying improvement and merits. Schmitz and Edenhofer (1980, 1983; Edenhofer and Schmitz, 1981) were the first to free the STI based flow computation from the courant constraint with their newly developed implicit method of characteristics (IMOC). Goldberg and Wylie (1983) also studied the implicit time-line scheme. While most computer programs for simulating transient flow in open channels make use of FDM, the MOC is finding its way towards more complex reaches. Bertrand and Zech (1984) chose MOC for their model since the method is more suitable to unsteady flow problems at complex boundary conditions and geometric shape such as different cross sections at two continuous sections of the channel, junctions of three branches, local constriction and discharge into or out of the canal. Katopodes (1980) however, argued that in case severe contractions or expansions of channel width are present, the one-dimensional method of characteristics fails dramatically, regardless of the detail of discretization. Refining the space discretization will produce better results only for channels slightly deviating from prismatic channel. Despite the seemingly numerous disadvantages of MOC and the increasing popularity of the implicit FDM, Lai (1988) asserted that the many attractive qualities and features of the MOC are too valuable to forsake lightly. He developed a multimode MOC scheme combining implicit, temporal reachback, spatial reachback, and classical schemes into one.

The most commonly used numerical method is the FDM which involves replacing the partial derivatives of the original system by quotients of finite difference. The formulation scheme can be explicit wherein the unknown quantities of the advanced time line are expressed in terms of known quantities of the current time line and thus, can be solved directly; and implicit wherein the unknown quantities are expressed in terms of still unknown quantities of the current time line producing a set of non-linear algebraic equations. The solution can be obtained either by iteration or by linearization of the non-linear terms and solving the resulting simplified system of linear equations simultaneously with the boundary conditions.

Amien (1976), in his paper about the four point implicit method, contended that the use of non-linear coefficients, the ability to use large time steps compatible with the physical aspects of the problem, and the ensuing matrix of coefficients of the system of equation which permits the use of special routines to solve the linear system very efficiently at each iteration step have contributed to the efficiency of the implicit method. To further improve the efficiency of the four-point implicit method, Verwey (1976) suggested that the advective momentum terms should be written in quasi-linear form, the set of equations can then be obtained by an efficient double sweep algorithm, and can be iterated once or twice to improve the linearized coefficients. However, Amien (1976) argued that the use of quasi-linear coefficients and the double sweep method verges on the trial and error solution which necessitates the use of small time steps and which is ineffective when dealing with flows of long duration.

The requirements of stability

In choosing which particular scheme is more appropriate for a given problem, the matter of stability must take precedence over other considerations related to accuracy because of the great size of errors eventually generated by an unstable scheme (Strelkoff, 1970). Explicit numerical schemes are simpler, but require small steps in time because of stability problems which makes them uneconomic, inflexible and inferior to other available numerical models (Zoppou and O'Neill, 1981). Implicit schemes, on the other hand, while allowing numerical solution over large time steps, require the simultaneous solution of a large set of algebraic equations. With recent advances in computing, this is now hardly a problem, making the implicit FDM the more favored technique. However, some researchers have shown that there are explicit schemes which are more attractive than the implicit scheme. Fennema and Chaudhry (1986) contended that three second-order accurate explicit schemes namely MacCormack, Lambda and Gabutti are more appropriate for real-life applications since they are easier to program and can accurately describe unsteady free-surface flows with shocks. Moreover, a model based on a stable explicit FDM was applied by Chaudhry (1976) to the Seton Canal in British Columbia, Canada. The scheme, also called the diffusive scheme, is stable unlike other explicit schemes because it adds viscosity terms to the original equations.

The stability and convergence characteristics of a four-point implicit finite difference scheme or Priessman scheme are examined by Lyn and Goodwin (1987). Particularly, the effect of weighting factor in space (ϕ), as well as in time (θ), is considered. It has been generally accepted that for unconditional stability it is necessary that $\theta \geq 1/2$. However, this is true only for a scheme centered in space, $\phi = 1/2$ which fortunately became the standard choice since it yields second order accuracy in time. However, for a non-standard choice $\phi \neq 1/2$, the stability will depend on the sign of the courant number, or equivalently, in the direction of travel of a characteristic wave. Because of this, the use of $\phi = 1/2$ should be used with caution in situations where characteristic direction may change, i.e., in transition from subcritical to supercritical flow. On the basis of convergence properties for $\phi = 1/2$, the box scheme ($\theta = 1/2$) is shown by Fread (1974) to be superior to the backward implicit scheme ($\theta = 1.0$) particularly with respect to wave damping.

The use of absolutely stable implicit difference schemes permits the selection of time and distance steps of the difference grid independently. This allows critical observation points along the channel to be located exactly on the grid points. It also provides opportunity for improving further the accuracy of the solution as shown by Vasiliev et al. (1976) when they applied the scheme in branching network (tree-type) as well as in general network systems (loop). They recommended the method for calculation of long transient processes in canal systems. In addition, Chu and Mostafa (1976) concluded that a model based on an implicit scheme is capable of accepting relatively large time intervals (Δt) with a combination of very small and large distance intervals (Δx) without affecting the stability and the accuracy of the solution. This particular characteristic renders the model most suitable for use in cases with side-weir flows encountered along short distances of an open channel. Moreover, the use of non-equidistant difference schemes has been proven superior to equidistant schemes particularly when the cross section of the channel varies greatly (Li, 1983).

The problem of discontinuities

A major problem encountered in open-channel hydraulics is representing a discontinuity in an otherwise continuous flow such as a hydraulic jump. The partial differential equations in this case generate multi-valued solutions that are physically unrealistic. The numerical techniques on the other hand may be accurate for smooth continuous flows but often become unstable with discontinuous flows.

One of the earlier works which elaborated this topic is that of Abbot (1974). He envisaged the development of an efficient numerical scheme for simulating flows with continuous and discontinuous parts. The techniques discussed inlude i) fitting the jump, by introducing the specific local laws of the discontinuity and mating the continuous parts on either side through these laws, simultaneously locating the discontinuity; and ii) using highly dissipative difference schemes. In both cases, the theory is complicated and subsequent programming becomes enormous.

However, the use of highly dissipative schemes, i.e. weighting factor, $\theta = 1.0$, which give maximum possible smoothing does not mean that they are always better and more appropriate than other schemes in representing these discontinuities. Cunge (1976) commented that a scheme which does not fail during computation when a discontinuity occurs does not mean that it reproduces nature more accurately. Rather, it means that it can only smooth real-life discontinuities and is more convenient to use. He advised that the weighting factor θ should not always be chosen as equal to one.

Furthermore, a difference scheme may be stable because of its dissipative and dispersive characteristics, but its convergence may be poor. For example, in a fixed bed problem, it is often chosen to increase the value of the time weighting factor, θ, towards unity in order to dampen parasitic oscillations. This, however, adds some inaccuracies to the solution (Lyn and Goodwin, 1987). The box scheme is shown to be the preferred implicit four-point difference scheme for discretizing the differential equations of unsteady flow (Fread, 1974).

Ellis (1976) recommended a method for handling this discontinuity. Since the transit time of unsteady flow phenomena through any discontinuity will be small in comparison with the commonly chosen time step of most numerical models, he assumed the transient effects impinging upon the upstream and downstream limits of the discontinuity influence simultaneously the entire body of fluid contained in the transition. It becomes possible therefore to apply the familiar steady flow laws of conservation of mass, energy and momentum to the transition at any instant of time. The discontinuities covered by his approach are those caused by broad and sharp-crested weirs, free overfall, abrupt change of cross section and a constriction of flow area.

The development of model-softwares

By 1976, the methods used for solving the Saint-Venant equations appeared to be satisfactory with mathematical models found to be adequate for quite a large number of applications (Priesmann, 1976). With the rapid advances in the computing world and the emergence of more reliable computational methods, the trend has shifted to the development of more global, albeit simple models for real-time applications.

Irrigation and hydraulic engineers have since begun developing computer models in user-oriented forms which can be used for day-to-day decision making in the operation of canal systems. More global and generally adaptable models such as the SIVA of the Computational Hydraulics Center and the HEC-2 and the GVUFP of the Hydrologic Engineering Center have been developed. However, as Abbot (1976) has pointed out, such ambitious models which involve the formulation of highly algorithmic, multilevel, accurate and multiple connected schemes has to be built into a system which can be used repetitively over a dozen models in order to be economically justifiable at all. Moreover, it is important that these large and complex models should be able to handle simple systems since simple problems do exist and that there are often more or less autonomous subsystems that can provide satisfactory answers without invoking a global scale (Askew, et al., 1978). There are numerous cases of one-off modeling, however. These models were developed to handle specific types of systems.

The gate operation for flow control of the California aqueduct of the California State Water Project has been continuously modified since 1967 to keep up with the changing operational demands (Dewey and Madsen, 1976). While the problems and the objectives are typical of unsteady flow modeling, its operational requirements are implemented based on the controlled volume concept. Nevertheless, this may have given early workers an idea of what powerful tools these computer models could become in the operation and management of large irrigation systems. Indeed, one of the first computer models developed for canal unsteady flow analysis is the California Department of Water Resources Hydraulic Transient Model developed by Strelkoff and Amorocho (Balogun and deVries, 1984). The model uses an explicit finite difference solution scheme for the Saint-Venant equations written in characteristic form.

Another pioneering venture is that of the Tennessee Valley Authority (TVA). As early as the late sixties, they had developed and have been continuously improving and testing the Simulated Open Channel Hydraulics (SOCH) model in Tennesse Valley and other similar basins. The model is formulated using the explicit finite difference scheme.

From then onwards, several computer softwares have been developed for one-dimensional unsteady flow simulation for open channel. IAHR (1978) as part of their policy for information exchange made a listing of these softwares. The list included CARIMA by SOGREAH (1978); Four point method of characteristics by IIHEE (1971); Cheric computation of unsteady flow in open channel networks (1976); Unsteady flow in open channel by Delft Hydraulics Laboratory (1975); Netflow by Delft Hydraulics Laboratory (1978); and S11-H6 by DHI (1976). Other softwares that followed suit were: Gradually varied unsteady flow program by TVA (1977); Dynamic wave operational model by National Weather Service (1978); System hydraulic package by HEC (1979); and Branched network flow package by US Geological Service (1981). Some of these softwares such as CARIMA and CAREDAS were used for training and education (University of Iowa,1983).

The development of new models and the improvement of existing ones has continued unabated. Most of these models have one common denominator, i.e. they are in user-oriented and problem-specific forms. The Simulated Open Channel Hydraulics model for instance has been continuously used and improved by the Tennessee Valley Authority (TVA) since the model has proven to be a reliable tool for solving a wide array of flow propagation problems in the Tennessee Valley and similar basins since the late sixties. In 1983, its input option was redesigned to facilitate data preparation. Recently, its output and print options have been modified to enhance it user-friendliness (Granju and Lowe, 1988).

The model that is developed by the US Geological Service is a general and flexible one-dimensional model that can be used for simulating unsteady flow in a single open-channel or in a network of interconnected channels under a wide range of flow conditions for various channel configurations. The model is based on a four point (box) implicit finite difference scheme. To achieve operational modeling capability, the model is linked to a highly efficient storage-and-retrieval module that accesses a database containing time series of boundary values and includes an extensive set of digital graphics routines. These features transform the model into a comprehensive tool for practical use in the conduct of hydrologic investigations (Schaffranek, 1985).

The Aqueduct Control Software (ACS) of the Central Arizona Project is a series of hydraulic models that generate schedules for pumps and gates based on the initial conditions of the aqueduct, the water delivery schedule, and the availability of the pumps and gates in the system. It consists of five interconnected models namely: 1) Initial conditions model (ICM); 2) staggered net unsteady state model (SNUSM); 3) baseload model (BLM); 4) gate setting model (GSM); and 5) reach 1 simulation model (RSM).

The model that was applied to the 108 km Delta Field Division canal of the California aqueduct was developed by Hamilton and deVries (1986). The model can simulate unsteady canal operation in any nonbranching canal system composed of a series of channels separated by control structures. A four-point, implicit finite difference scheme known as Priessman's method is used.

A model which can simulate canal filling, operating and draining phases, bulk lateral inflow or outflow into the section being modeled, and control structure scheduling was developed in 1987 by Gichuki and associates. The model includes three water control options: i) model user decides what inflows and control structure settings should be implemented; ii) program generates system inflow and the control structure settings which achieve predetermined flow depths; and iii) program generates system inflow and the control structure settings which deliver a specified discharge to the lower reach. An improvement on this model is a hydraulic simulation model which was developed at Utah State University. Its applications are in the areas of operation, design and operator training. As an operational tool, the model can be used to schedule control structure settings according to predetermined water distribution strategies while maintaining stable flow levels. As a canal system design tool the model can be used to analyzed proposed designs by simulating the hypothetical canals under expected operational conditions. And as a training tool, it can be used by the canal operators to simulate a wide range of possible flow conditions safely and quickly, thereby allowing the operator to become more familiar with the operational response and limitations of a canal system. This hydraulic model is interactive and menu-driven, with extensive use of color computer graphics for the display of simulation results (Merkley, et al., 1987).

In India, the Narmada Development Department (1987) formulated a computer model which can simulate canal hydraulics for all expected operational conditions of the Narmada canal project in India. This permitted the formulation of efficient rules for normal operation and operation criteria which ensure safe shutdown of the canal during emergencies.

In Saudi Arabia, Husain et al. (1988) developed an open channel flow simulation package which they called channel network model (CNM). It is a modified version of dynamic wave operational model (DWOPER). While it retains such capabilities of DWOPER as simulation of flow through a river system with contributing tributaries, it can also be applied to distributing junctions, used to divert water into various branches of the system. The model is also capable of simulating flow through submerged and free flow types of gates with continuity as well as discontinuity vin the bed slopes and at critical flow sections. An important feature of CNM is that a part of the irrigation system can be discarded from the analysis without altering the input data files. The package was applied to the Al-Hassan irrigation system.

Another model which is used for simulating the flow in a single reach or a confluence of two reaches, using the four point implicit finite difference formulation was developed by Abdul-Salam (1988). He applied the model to the Tigris river in Baghdad and its confluence with the Diyalah river. The Salvaterra canal of the Sorraia irrigation project in Portugal is likewise used as a case study for another simulation model development. Rijo et al. (1988) used an implicit finite difference technique in this model.

A user-friendly package, HD-System RUBICON for the simulation of unsteady flow in open channel networks was developed by Haskoning BV and Delft Engineering Software (Verwey and Haperen, 1988). The model can be used for studying a wide range of hydraulic engineering problems such as flood wave propagation in channels, rivers, food plains and reservoirs; tidal flows in rivers and estuaries; optimum design and operation of irrigation and drainage systems. Some of its user-friendly features are separate processing for input, execution and output subsystems; extended free format data input; and possibility to add user-defined subroutines and functions.

IIMI (1989) has been developing a model which is mainly based upon the well-established computer software of CEMAGREF, France. It is made up of three software units which can run independently or sequentially. These are i) topography unit which enables the user to input and verify all the topographic data pertaining to the canal; ii) steady flow unit which allows for the calculation of regulator and offtake gate openings as well as water surface profiles in the different canal reaches for any steady state scenario of water supply and demand; and iii) unsteady flow unit which allows the simulation of unsteady flow conditions that prevail over certain periods of time in response to changes in water supply or demand, and modifications to gate settings. A major component of the model is the development of a user-friendly conversational procedure so that it can be used by non-specialized staff. The model has been applied to the Kirindi Oya RBMC in Sri Lanka.

A simulation model which includes the effects of hydraulic structures and leakage through the riverbed in open channel networks is developed by Swain and Chin (1990). They compared the model to the Branch model of Schaffranek (1985,1987). A section of the Potomac river near Washington D.C. was used for the model verification.

The Irrigation Conveyance System Simulation (ICSS) developed by Manz (1990) is capable of simulating all the important hydraulic and operational characteristics of known types of lined and unlined irrigation canals under steady or unsteady conditions. There are no restrictions in the number or type of canals or hydraulic structures that can be included in the model. Canal characteristics such as shape, roughness, slope, seepage, precipitation inflow and other distributed lateral inflows or outflows may each be varied along the canal length as frequently as required. The model has been successfully applied to several projects in southern Alberta over the last four years.

A model which can be used for gate scheduling was developed by Merkley et al. (1990). This is an extension of their original model. Gate scheduling, which is one of the three operational modes, determines control structure and turnout settings with the objective of quickly stabilizing transient flow conditions, while maintaining flow levels at or near target values. Gate scheduling can be used to accurately match planned turnout deliveries and eliminate operational uncertainties. Field application of the model was performed in an irrigation project in northeast Thailand.

The near future

It thus becomes obvious that the direction of model development is going towards user-friendly and problem-oriented model-softwares. It should be noted that with rapid advances in computing, it is no longer acceptable to produce the results of simulation models in the form of endless reams of numbers. The users should be able to use the simulation model in a way similar to operating an actual irrigation system in the field. Often the first development involves a simplified system which can be played as a game and then further developed as a real operating procedure so that field operators can see the consequences of a proposed action before introducing it to the real system (Stoner et al., 1989).

MODEL AND MODEL ATTRIBUTES

Desired model attributes

The usefulness of these mathematical models was extended further as the development of user-oriented softwares became the norm. The development has been increasingly rapid and competitive, and users are faced with an ever widening dimension of choice. Correspondingly, the capabilities and attributes of these softwares have been continuously improved and updated.

The thoroughness of the equation formulation on which a model is based largely governs the range of complexity of flows it can accommodate. The choice of numerical computation scheme primarily determines whether the model will be stable, convergent, accurate and computationally efficient. Presently available models for river systems and also the few irrigation specific models need to be modified to i) be able to simulate the whole range of possible flow conditions in the irrigation system; ii) permit schematization of different yet limited types of channel geometries; iii) allow the incorporation of relevant hydraulic and operational characteristics of different control structures and types of canals typical of the region; and iv) be capable of incorporating in the analysis the effects of all controlled inflows and outflows such as those in the distributing junctions and uncontrolled inflows and outflows such as seepage, precipitation, evaporation and runoff. However, as mentioned repeatedly, for any model to be useful it must be translated into a functional user-oriented software with its accuracy, reliability, flexibility and efficacy adequately proven and demonstrated. Such software should be able to provide the following functions:

1. evaluation of operational performance of irrigation plans before actual implementation.

2. assessment of the ability of the system to deliver water according to proposed allocation schedules.

3. development of operational rules and procedures for different irrigation scenarios, e.g. determination and scheduling of gate settings of control structures to effect desired water levels.

4. development of long term operational and management rules and guidelines such as establishment of emergency measures during floods and droughts; and evaluation of maintenance and rehabilitation programs.

5. evaluation of proposed canal system design and verification of canal hydraulic parameters.

6. research tool for studying irrigation water delivery systems.

7. training tool for system operators.

The software should be supported by interactive programming and maximum graphic utilities to enhance its usefulness and practical appeal to the users, particularly to non-specialized users who have minimum computer knowledge. The user should be able to define the hydraulic and operational characteristics of his system with minimum programming effort and produce results in clear tabular and graphical outputs. While the software should be generalized such that various problem scenarios can be accommodated over different systems, it should permit single-function subroutines to be run independently. This allows economic and easy verification of parts of a solution which are in doubt without invoking the whole program. Similarly, it should allow parts of the system to be isolated or discarded in the analysis without reconfiguring input data.

While the model-software is explicitly intended to serve as a tool for real time operation and management of the system in its entirety, it can likewise be used for irrigation planning and design. In this case, the model should be able to evaluate proposed canal system designs, assess the ability of the system to deliver water according to proposed allocation schedules, define emergency operation measures during floods or droughts, evaluate maintenance and rehabilitation programs, and assist in the development of long-term operational and management rules and guidelines.

At any rate, it should be noted that the popular use of a model is not solely dependent on its intrinsic attributes but also on the support that is given to it by the developer. The provision of user-friendly manuals i.e., not too long and complicated since this might discourage the user from using the model, but not so short that nothing can be learned from it, is necessary. The program should be easily transferable and should be appropriate to the prevailing technology of the user.

The computer should not decide. A user decision-control mechanism should be provided which permits the user to choose from among several decision alternatives the most appropriate and relevant decision based on the user's perceived social, economic and political implications.

Finally, emphasis should be placed on the modeling result rather than on the modeling itself. While a comprehensive model construction may prove to be advantageous in the long run it should always be remembered that the best theoretical model does not represent the best solution.

The incorporation of all the features mentioned above in one software could be difficult to achieve. However, there now exist a number of model-softwares which may serve as take-off points for new developments.

Branch Network Flow Model by US Geological Survey

This is a comprehensive one-dimensional numerical simulation model that is fully supported by a user-oriented system for modeling. It is capable of simulating unsteady flow in a single reach or throughout a network of reaches composed of simple or multiple connected one-dimensional flow channels governed by various time dependent forcing functions and boundary conditions.

Model formulation. The flow equations are expressed in finite-difference form using a weighted four-point (box) scheme. The model permits the users to use unequal segment lengths and allows them to choose from box-centered to fully forward discretizations.

Solution algorithm. A unique transformation operation is applied to the segment flow equations in the model to lower the order of the coefficient matrices and thereby reduce computer time and storage requirements. A general matrix solution algorithm is used to simultaneously solve the resultant branch-transformation and boundary condition equations. An optional iteration procedure, controllable by user-defined tolerance specifications, is additionally provided to permit improvement in the accuracy of the computed unknowns.

The solution process begins at time t^o by use of specified initial conditions and proceeds in Δt time increments to the end of the simulation at time t^n. Gauss elimination using maximum pivot strategy is employed to solve the system of equations. Iteration within a time step is performed to provide results with user-defined tolerances. The primary effect of the iteration is to improve on the quantities taken as local constants within the time step, which in turn increases the accuracy of the computed unknowns.

Stability. The use of the four-point implicit method allows the stability and convergence properties of the solution to be controlled.

Initial and Boundary conditions. To solve the branch transformation equations implicitly, boundary conditions must be specified at internal junctions located at branch confluences within the network as well as external junctions located at the extremities of branches. Equations describing the boundary conditions at internal junctions are automatically generated by the model, whereas boundary condition equations for external junctions are formulated by the model from user-specified functions. Discharge and stage compatibility conditions can be expressed for internal junctions by neglecting velocity-head differences

and turbulent energy losses. On the other hand, various combinations of boundary conditions can be specified for external junctions. A null discharge condition, known stage or discharge as a function of time, or a known, unique stage-discharge relationship can be prescribed. Together, the internal and external boundary conditions provide a sufficient number of additional equations to satisfy requirements of the solution technique. Although, these make the system of equations determinant, initial values for the unknown quantities are required to start the solution process. These values may be obtained from measurements computed from some other source such as steady state approximations computed from previous simulations, or otherwise estimated. The model's successive use of newly computed values as initial values permits the computation to proceed step-by-step at the specified computation time interval until the simulation is completed.

Model application: The model can simulate the time-varying flows of several coastal and upland water bodies. These represent a broad spectrum of hydrologic field conditions, depicting such diverse hydraulic and fields situations as hydropower-plant-regulated flows in a single upland river reach, tide-induced flows in riverine and estuarine reaches and networks, unsteady flow in a residential canal system, and meteorologically generated seiches and wind tides in a multiple connected network of channels joining two large lakes.

The model can be used for four types of application. The simplest is the single-branch type which is an application to a single reach of channel delimited by a pair of external boundary conditions. The multiple branch type is an application to a channel again delimited by a pair of external boundary conditions, but schematized as a series of sequentially connected reaches. The dendritic-network type is likewise an application to a channel system composed of branches connected in treelike fashion. The multiple connected network type is likewise an application to a channel system, but one in which the branches are interconnected, thereby permitting multiple flow paths between certain locations in the system.

Hardware Features. The program is written in standard FORTRAN IV and can be implemented in an IBM 370/155 or an Amdahl 470V/7.

Channel Network Model by Husain et al. (1988)

This package is an updated version of the dynamic wave operational model by the National Weather Station. The introduction of hydraulic equations on features such as junctions with distributaries, gates of submerged and free flow types, and bottom falls were some of the innovations. Other features, such as interactive capability and use of subsystems, were added.

The model is based on one-dimensional nonlinear differential Saint-Venant continuity and dynamic equations governing unsteady flow in an open channel. In addition to handling tributaries, it can also be applied to distributing junctions, which are used to divert water into various branches of the system. It is also capable of simulating flow through submerged and free flow types of gates with continuity, as well as discontinuity in the bed slopes and at critical flow sections. The gate coefficient can be defined as a function of differential head instead of taking a single-gate coefficient value.

Model Formulation. The model is based on an implicit finite-difference technique.

Solution Algorithm. The model provides more options on boundary conditions and a special computational algorithm to provide numerical stability. In an implicit scheme, a system of 2N algebraic equations is generated from the Saint-Venant equation applied simultaneously to the N net points along the x-axis. The generalized Newton-Raphson method is used. Computations begin by assigning an approximate trial value to the 2N unknowns which yield a set of 2N residuals. The solution to the equation is iterated until the trial values converge to the actual values.

Stability. The computational stability of the package depends on the selection of distance between nodes, Δx, and time step Δt. The ranges of Δx and Δt are based on wave velocity and time of rise of the inflow hydrograph. It is noticed that a network with relatively closely spaced computational nodes is more stable and gives more accurate simulated results than one with sparsely spaced nodes. Similarly, the trend of inflow and gate closure and the opening rates affect time step Δt. In the case of a sharply peaked inflow hydrograph and rapid openings and closures of gates, it is necessary that Δt should be small enough to make the solution of the Saint Venant equations computationally stable.

Initial and Boundary Conditions. Initial conditions need not be supplied at all nodes. The depth and discharge under steady state conditions will be internally computed by supplying the initial discharge or water level at the upstream node. External boundaries are specified at the beginning and end of a particular reach considered for simulation. The upstream boundary condition can be represented as i) stage as a function of time; or ii) discharge as a function of time. The downstream boundary condition can be prescribed as i) stage as a function of time; ii) discharge as a function of time; or iii) stage-discharge relationship. The junctions, gates, and critical drops are treated by the model as internal boundaries. At these locations, the flow is considered rapidly varied rather than gradually varied. The mathematical formulations of these boundaries are included in the model.

Model Application. The model can be used for simulating unsteady flow in an open channel network with features such as multi-branching configurations, gates, tributaries and bottom falls. The reliability of the model depends upon the specifications of the initial conditions, accuracy of the inflow hydrograph, calibration of gates and determination of Manning's roughness coefficients.

Special Features. Some of the model's special features include:

Interactive capability. The program is easy to use. The input parameters which do not change from one execution to another are read in the batch mode, but time dependent parameters, e.g. hydrographs, gate settings, and initial conditions can be modified from the terminal through question and answer sessions.

Use of subsystems. The complete system is modeled by a number of subsystems which can be treated independently from others. This is to minimize errors introduced in data handling of a very large and complex system. The user can also easily simulate one or more interconnected subsystems through the interactive capabilities of the program.

Geometric Data preparation and initial conditions. To define the elevation and geometry of the canal at each node, a subroutine is used that calculates the required physical and geometric information by reading the geometric data at the node where either the geometry of the section or the slope is changing. Similarly, initial conditions need not be supplied at each node since a subroutine calculates these at all nodes by just reading the initial discharge or water level at the first node.

Dynamic node numbering. This has simplified data preparation and has given the user the option of operating any part of the main irrigation system at any time without altering the input data files. The dynamic node numbering and interactive capabilities of the package have given free rein to the user to choose an operation schedule without any constraints on data preparation, memory storage, stability, etc.

Hardware Features: The program is written in standard FORTRAN IV and implemented on an IBM 3033 system.

Utah State University Model

This model can simulate transient flow conditions in branching canal networks with bulk lateral outflow at multiple locations. The model is capable of performing hydraulic simulations using three different operational modes, two of which involve gate scheduling for computing control structure and turnout settings as a function of time and hydraulic conditions. Extensive development of interactive program features and graphic displays enhance the model's utility in nonresearch application.

Model Formulation. The model is based on a deformable control volume approach to solve the St. Venants equations of continuity and motion for open-channel flow.

Solution Algorithm. The solution technique uses computational "cells" which are delineated by nodes in both time and one-dimensional space (parallel to the mean direction of flow). At each node the dependent variables of flow rate and flow depth are determined simultaneously for each time step. Boundary conditions at the upstream and downstream end, or at intermediate locations along the canal reach, will uniquely determine the transient solution to the governing equations along the reach. The simulation of bulk lateral outflow through turnouts is accomplished through the inclusion of additional nodes and local modifications to the continuity equation.

Initial and Boundary Conditions. At time t=0, the initial conditions may be an empty canal, in which case Q(x,0)=0 and A(x,0)=0 for all x, or else predetermined flow conditions are established based on a previously computed steady or unsteady condition. In the latter case, the previously computed flow conditions serve as the initial conditions. For boundary conditions, any of the following can be simulated: 1) upstream boundary can be either specified flow rate and computed flow depth; or specified flow depth and computed flow rate; 2) downstream boundary can be a) specified control structure setting and computed flow cross sectional area; b) specified flow rate and computed flow cross sectional area and control structure setting; or c) specified flow cross sectional area and computed flow rate and control structure setting; and 3) bulk lateral outflow.

Model Application. The model can also be used to simulate flow into a dead pool, sudden opening of a sluice gate, and submerged flow conditions across the control structure. The model cannot be used on nonprismatic canals and spatially dependent roughness coefficients and bed slopes.

Special Features. The model can simulate the filling and operation of canals and can handle bulk lateral outflow from turnouts and wasteway weirs.

Hardware Features. The program is written in Pascal. An IBM microcomputer or a compatible is required with a hard disk and a minimum of 640 KB RAM. A math coprocessor is highly recommended.

MIKE 11 by Danish Hydraulic Institute

MIKE 11 is a integrated software package for the simulation of flows, sediment transport and water quality in estuaries, rivers, irrigation systems and similar water bodies. It is especially developed for microcomputer applications and is based on DHI's well known SYSTEM 11.

The core of the system consists of the hydrodynamic (HD) module, which is capable of simulating unsteady flows in a network of open channels. The result of the HD simulation consist of time series of water levels and discharges. Associated with the HD module, is the rainfall-runoff model NAM, which may be used to generate inflows to the HD module. Transport dispersion (TD) and sediment transport (ST) calculations may be carried out from special modules which utilize the result of the HD computation.

The hydrodynamic model can describe subcritical as well as supercritical flow conditions through a numerical description which is altered according to the local flow conditions (in time and space). Advanced computational modules are included for the description of flow over hydraulic structures, including possibilities to describe structure operation.

Model Formulation. The hydrodynamic module is based on an implicit finite difference model for unsteady flow computations. The computational scheme is applicable for vertically homogeneous flow conditions extending from steep river flows to tidal influenced estuaries. The formulation can be applied for looped networks and quasi two-dimensional flow simulation on flood plains.

The model provides a choice between three different flow descriptions namely: kinematic wave approach; diffusive wave approach; and dynamic wave approach. The user can then select the most appropriate description based on the type of problem. The dynamic and diffusive wave descriptions differ from the kinematic description by being capable of calculating backwater effects. However, none of the wave descriptions include specific descriptions for energy losses in hydraulic jumps, although the chosen formulation ensures a correct description in the vicinity upstream and downstream of the jump.

For supercritical flow, a reduced momentum equation is applied neglecting the non linear terms. The description will still be accurate as long as the flow situation is not changing rapidly compared to the velocity. The advantage is that a stable solution can be obtained within the same algorithmic structure as that used for subcritical flow. For the transition between subcritical and supercritical flow, a gradual reduction of the non-linear term is applied allowing for a smooth description.

Solution Algorithm. The transformation of the Saint-Venant equations to a set of implicit finite difference equations is performed in a computational grid consisting of alternating Q- and h-points. The computational grid is generated by the model on the basis of the user requirements. Q- points are always placed midway between neighboring h-points, while the distance between h-points may differ. The double sweep method is used.

Initial and Boundary Conditions. The initial conditions can either be specified by the user or can be calculated automatically by the system. For the automatic calculation, the Manning's formula is used to determine the water levels and discharges in the model region, so that they correspond to the boundary values at the start of the simulation. An iterative procedure is used to obtain a correct discharge distribution between diverging branches.

The boundary conditions are distinguished between external and internal boundary conditions. The internal boundary conditions include links at nodal points; structures; internal flows; and wind friction. External boundary conditions include constant values for h or Q; time varying values for h or Q; and relation between h and Q.

Model Application. MIKE 11 is an integrated software package for the simulation of flows, sediment transport and water quality in estuaries, rivers, irrigation systems and similar water bodies.

Special Features. Wind effects are included in the analysis.

Hardware Features. The program is written in Pascal. An IBM microcomputer or a compatible is required with a hard disk (10 MB minimum) and a minimum of 512 KB RAM. A math coprocessor is required.

USM (Unsteady Model) by USBR

USM is a canal hydraulic simulation model which uses the method of characteristics to calculate the numerical solution to the complete Saint-Venant equations of unsteady open channel flow. It can be used to analyze the effect of transient conditions being introduced into a canal system by turning a pump on or off, changing gate position, or changing a turnout demand. It models uniform channels only (no river beds) with the following cross sections: trapezoidal, horseshoe tunnel, circular tunnel, trapezoidal with one vertical side, and trapezoidal membrane lined.

Model Formulation. The program models gradually varied unsteady flow. It uses the method of characteristics to solve the continuity and momentum equations. The solution yields flow and depth at incremental distances along the reach and incremental intervals of time, creating a grid of solution points. Two variations of the method of characteristics solution are included, namely the complete grid of characteristics and the specified time interval.

Initial and Boundary Conditions. The canal system to be modeled is referred to as a reach. The reach comprises up to 40 nonbranching pools separated by boundaries. These boundaries include automatically controlled or scheduled gates, weirs, siphons, pumps, turnouts and changes in canal cross-section or in canal properties. The following initial conditions must be also specified: initial depths and flows along the canal, initial gate positions, initial pump flows, and initial turnout flows.

Model Application. Canal operators can use the model to study how an anticipated schedule of changes will result in water overtopping the bank or exceeding drawdown criteria. They can study how soon they would have to introduce the increase in flow upstream to see the effects at the downstream end of the system. Operators can compare different operating scenarios to find out which results in the best operating conditions. The model can also be used during the design phase to check various expected operations. It can be used to find optimal check structure locations, size in-line regulating reservoirs, and determine canal freeboard. In addition, it can also be used to model automatic gate controllers to determine the optimal controller for a system and the optimal constants for that controller.

Hardware Features. USM is written in FORTRAN IV and can use either english or metric units. An IBM microcomputer or a compatible is required with a minimum of 640 KB RAM. A math coprocessor is highly recommended.

ICSS (Irrigation Conveyance System Simulation) Model by Manz, D.H.

The ICSS model is an unsteady flow simulation model which is capable of simulating all of the important hydraulic and operational characteristics of known types of lined or unlined irrigation canals under steady or unsteady hydraulic conditions. Single canals or networks of canals can be simulated with no restrictions placed on the number or type of canals or hydraulic structures that it can accommodate in the analysis. Canal characteristics such as shape, roughness, slope, seepage, precipitation inflow and other distributed lateral inflows or outflows may each be varied along the canal length as frequently required.

*Model Formulation and Solution Algorithm.*_ The steady flow simulation capability is achieved using the equations for gradually varied flow as derived and discussed by Henderson (1966) and solved using the finite difference technique similar to that introduced by Fread and Harbaugh (1971). The unsteady flow simulation capability is achieved through the solutions of unsteady, gradually varying, open channel flow solved using an implicit finite difference technique described by Amein (1968) and demonstrated by Amein and Fang (1970).

Initial and Boundary Conditions. Initial conditions may be supplied by the steady flow simulation module or by the user himself. Each reach can be bounded by groups of one or more hydraulic structures such as check structures (radial, flashboard, weir type) in combination with gate turnouts (free or submerged outlets); expansion and contraction structures; bridges; drop structures; and regulation ponds.

Model Application. The model has demonstrated great utility and versatility in applications to engineering problems concerning the design, operation and performance evaluation of irrigation conveyance systems. It can be used for developing optimum operational procedures for existing or proposed irrigation canals. The model can likewise be used for the calibration of simpler models, performance evaluation of existing systems or operational procedures and the training of operational personnel. Other major application areas include: development and evaluation of maintenance and rehabilitation programs; staging of various rehabilitation programs; and development of operational procedures during drought.

Special Features. The model is run interactively in a similar way to a video game, with required output displayed at the terminal and/or stored on disk. The outputs may be in the form of i) restatement of input data; ii) initial water surface profiles, velocities and discharge throughout each reach; iii) water surface profiles and velocities for each reach as they vary with time; iv) water depth and discharge at the inlet and outlet as they vary with time; v) instantaneous and cumulative statement of the water balance for each reach; vi) record of all operations performed on the hydraulic structures bounding each reach; vii) record of the hydraulic structure physical status and flow conditions at the inlet and outlet of each reach; and viii) performance summary for entire simulation.

Hardware Features. The model is written in FORTRAN 5 and versions are currently being run on a CDC Cyber 860 computer and an IBM 386 personal computer.

HD-System RUBICON by Haskoning BV and Delft Engineering Software

HD-System RUBICON is a modeling package for the simulation of unsteady flow in systems of open channels. Modeling is based on the full Saint-Venant equations.

Model Formulation. The Saint-Venant equations are solved with a highly accurate and efficient modification of Priessmann's implicit finite difference scheme.

Boundary Conditions. The model is very flexible in specifying internal and external boundary conditions. The user can select from a number of system elements to simulate complex flow over flood plains or define structures at any point of the channel system such as weirs, gates, culverts, siphons, spillways, sluices, stormsurge barriers, dikes, etc.

Model Application. The model can be used for a wide range of hydraulic engineering problems such as i) flood wave propagation through channels, rivers, flood plains and reservoirs; ii) tidal flow in rivers and estuaries; iii) effects of structures in channel systems; iv) optimum design and operation of irrigation and drainage systems; v) wave propagation resulting from dam failures; vi) hydraulic parameters in water quality studies. The package is extremely useful both for use within hydraulic engineering practice and for the training of hydraulic engineers either in the university environment or within government institutes and consulting firms.

Special Features. An important feature of this is its user-friendliness which minimizes the time required to prepare data and to set up the system in a modular way, suitable also for installation on the new generation of desk-top computers. This is exemplified by i) a separate processing for input, execution and output subsystems; ii) extended free format data input including comments; iii) possibility to add user-defined subroutines and functions; iv) all user defined model elements; v) automatic generation of computational grid and elements following user's directives; vi) use of special information symbols to minimize input efforts; vii) extensive checking of input data; viii) continuation of input processing after detection of errors; ix) restart facilities in model execution; x) possible generation of output at any point of the channel system and for a large range of parameters.

Hardware Features. The program is written in FORTRAN 77 following the full ANSI standard.

DU(tch)FLOW by International Institute for Hydraulic and Environmental Engineering, Rijkswaterstaat Dienst Getijdewateren and Delft University of Technology

DUFLOW is a computer package for the simulation of one-dimensional unsteady flow in branched open channels. It is intended for professionals in the field of design, operation and management of water courses.

Model Formulation and Solution Algorithm. The model is based on the one-dimensional partial differential equations that describe non-stationary flow in open channels. The equations are discretized and solved using an efficient, unconditionally stable, second order accurate finite difference scheme. The resulting system of equations is solved using a straightforward Gaussian elimination.

Boundary Conditions. Boundary conditions may include i) water levels and discharges, either constant or in the form of a time series or Fourier series; ii) additional or external flow into the network which can be specified as a time dependent discharge or can be computed from a given rainfall using a simple rainfall-runoff relation; and iii) discharge level relations in tabular form.

Model Application. Applications can be in the form of training, design, real-time control and research in irrigation and drainage systems, flood waves, tidal waves, and harbor oscillations. Typical examples of projects include design of control structures; prediction of consequences of water management decisions; forecasting of flood in river basins; real-time prediction of velocities in closure operations; determination of pumping station capacity; and training of experts in water resources development.

Special Features. DUFLOW is a flexible microcomputer-based package which is menu driven and uses interactive programming. It allows for a quick and easy definition of a system of interconnected open water courses with control structures like weirs, culverts, siphons and pumps. User-defined operation rules allow the simulation of managerial measures depending on specific hydraulic conditions.

Hardware Features. DUFLOW is a stand-alone package designed for microcomputers - IBM compatible PC (XT, AT or 386 class) running under MS-DOS. The program is written in GW-BASIC and FORTRAN 77 and requires 640 KB of memory, two floppy disk drives and a graphics card (CGA, OLIVETTI or Hercules). A mathematical coprocessor and hard disk are strongly recommended.

MODEL APPLICATION IN SOUTH AND SOUTHEAST ASIA

Irrigation systems in South and Southeast Asia are mainly designed and operated for the continuous irrigation of rice during the wet and dry season. With water still an abundant resource relative to the area irrigated, management of the main system is simply a straightforward opening and closing of the diversion weir coupled with routine and simplified operational activities in the main systems which focus mainly on maintaining physical works, avoiding siltation and implementing established routines for rotations. This negligent practice has been uncritically carried until recently, when the demand for water has increased, resulting in short supply and producing unwarranted results. Even in systems where irrigation water is sufficient, avoidable water shortages have often persisted due to low operating efficiencies and widespread inequity in allocating and distributing water (Wickham, T., 1985). There are immense opportunities for improvements in the performance of irrigation projects through management reform and as Botrall (1981) has mentioned, in South and Southeast Asia at least, the key to overall improvement in management lies in better water distribution.

A software for the region

As adequate, equitable and predictable supplies of irrigation water are required for farmers to make the best use of their on-farm application systems, there emerges a strong and urgent need for an improved performance of the main system. A full understanding of the hydraulics of the system is required to meet this urgency competently and sufficiently. With the ongoing modernization of the hardware part of irrigation systems in the region, the introduction of the software part could very well be timely.

The use of computer models which treat the hydraulic behavior of these systems is proving to be an increasingly feasible solution to the irrigation management problem. With nearly infinite feasible combinations of water control (gate openings, discharge variations, water delivery scheduling, canal rotation, etc.) made available by the computer to the decision-maker for a any operation scenario, the degree of freedom for improving the system operation becomes virtually unlimited. The software can be used for planning (simulation of future irrigation operation scenario; manpower (irrigation staff and farmers) training and education; and real-time operation of the irrigation system.

Mathematical modeling of the whole or part of the irrigation network is not unexplored in the region. Mention must be made of the efforts of the International Irrigation Management Institute, who are presently developing a computer model which can simulate the steady and unsteady behavior of flow in the Kirindi Oya system in Sri Lanka. The Overseas Development Unit of Hydraulic Research has just completed a modeling project for the Porac-Gumain system in the Philippines. The Narmada Development Department in India has developed a model for analyzing the unsteady behavior of flow in the Narmada canal. The Asian Institute of Technology in Thailand produced several quality research projects on the modeling of irrigation systems. More recent studies has given particular emphasis to real-time applications to systems in Thailand, Nepal and Vietnam (Jha, 1990, Thapa, 1989, Das, 1988, Binh, 1988).

Management Scenario

In South and Southeast Asia and in many parts of the irrigating world, efforts to improve irrigation performance are classified into two main types. While the improvement of the tertiary level has received greater attention during the past decade, it is now widely recognized that more effort should be devoted to improving the performance of the main system. A problem for instance which is basically due to the mismanagement of the system is the tail-enders problem, where excessive diversions in the upstream reaches cause artificial shortages of water in the downstream reaches. This predicament is mainly caused by the lack of regularity and reliability of the system's water supply which lead even the skillful farmer to over-irrigate his field in order to compensate for the system's unpredictability (Ng and Lethem, 1983). Since reliability decreases as distance from the source increases these unauthorized diversions become rampant, aggravating further the unreliability of the system. Thus, this becomes a vicious cycle wherein system unreliability forces farmers to act negatively making the system more unreliable.

The UN ESCAP in 1987 identified the problems common to irrigation systems in the region. Those particularly concerned with main system operation include i) inefficient operation and maintenance of the irrigation system itself causing excessive conveyance and distribution losses; ii) inadequate canal capacities to serve intended service areas; iii) shortage of trained manpower; and iv) improper utilization of water by the users. We will describe how these problems can easily be mitigated with the help of the hydraulic simulation model-software.

Inefficient operation and maintenance. Inefficient operation of the system is the main reason why systems perform well below their potential. There are several causes:

1. ***discrepancies in design and as-built specifications of the structural components of the system*** - normally systems are operated based on design parameters which are not representative of the actual values in the field. Obviously, this might produce artificial inadequacies in parts of the systems due to inadequate diversion, insufficient hydraulic head and non-maintenance of full supply level (FSL), excessive delays in water transfer, inadequate duration for irrigation rotations and other unwarranted results. While verification of the design parameters is supposed to be a standard operating procedure, this is not the case in practice. The situation becomes more serious when such parameters change with time due to canal siltation and restoration, weed growth, and scour. With the hydraulic model-software, such changes can easily be detected and the parameters readily recalibrated. The use of such a model reminds irrigation officials of the significance of regular verification of the system and encourages them to perform such operating procedures assiduously.

2. ***inaccurate estimation of coefficient of roughness, discharge coefficient and conveyance loss*** - similarly, these parameters must be regularly verified. The use of the hydraulic model-software will enable easy calibration of these parameters in steady as well as unsteady flow conditions.

3. ***improper farm turnout locations*** - many farm turnouts are directly located in the main and distributary canals rather than in minors or in laterals. This increases the complexity of the operation of the main system. Moreover, it is difficult to maintain the FSL in the mains to supply the design discharge through these turnouts. This leads to uncoordinated opening and closing of such turnouts by farmers, which worsens the tail-end problem. The hydraulic model-software can be used in defining an opening and closing procedure for the upstream turnouts without the necessity of providing additional structures to improve control.

4. ***insufficient intermediate controls*** - many farm turnouts are functioning below their desired discharge capacities because the water level is not at FSL. To obtain the desired hydraulic head, checks are normally introduced in the downstream side. However, most often there is no check structure immediately downstream of the turnout or, if there is, it is dilapidated. Whether it is necessary to provide additional control structures at certain locations of the system can be easily evaluated with the aid of the model-software. How and when to operate such control structures can likewise be readily defined.

5. ***improper gate settings*** - determination of gate settings of cross regulators and distributaries are often based on steady state flow conditions. However, because of these control structures, canal flow is far from remaining steady and, as mentioned, the design parameters are not usually representative of the actual field values. In many cases, efforts are never made to validate these hydraulic assumptions and check the conformity between expected performance and actual performance. Even when the flow requirement of each unit is known exactly, the problem still persists particularly when the gate adjustment of structures along the main system is done both for the in-line and off-line control structures. Furthermore, most gate operations are adjusted manually by trial and error based on calibration curves which are usually different from actual conditions. A simulation model-software capable of representing real conditions can be used in prescribing appropriate decisions.

6. ***night irrigation*** - most irrigation canals in the region flow continuously at night and much water is being misused and wasted. While irrigation at night may be reduced and water may be stored instead, improving irrigation at night may be the more acceptable alternative particularly for run-off-the-river systems where water not diverted is water lost by the system; where on-farm ponds and intermediate reservoirs are not installed; and where time allotted for irrigation is limited. Moreover, downstream farmers sometimes prefer night irrigation because the water supply is more likely to come, free from the interference of upstream irrigators. However, to improve it is to make the flow predictable and manageable. Again this can easily be achieved with the help of the hydraulic model-software.

7. ***communication problems*** - in large scale irrigation systems, communication between irrigation officials and between officials and farmers remains a serious problem. Normally, water tenders and farmers already have their agreed day-to-day water plans. However, in cases of emergencies such as the occurrence of heavy rains or collapsed dikes, or in cases where water because of its transient behavior does not follow its normal course, it will be difficult for the many water tenders to react expediently. These extreme cases can easily be simulated by using the model-software, and emergency measures can be provided to the water tenders in advance.

While the operational problems above can easily be mitigated by the use of the model-software, sound and appropriate maintenance procedures can likewise be defined through the help of the software. It will be easy to identify any changes in the performance of the system by the use of the model. Any significant drop in the performance can be detected and the cause can be isolated and corrected immediately.

Inadequate canal capacities. While this is a hardware problem and can be more likely mitigated by a hardware solution, a software solution is nonetheless relevant. Actually, it is in this kind of situation where the use of a hydraulic simulation model can be of paramount importance since there emerges a need to delicately optimize the carrying capacity of the canal. Moreover, it might be possible that an acceptable combination of gate settings with time from among the numerous possible combinations that will be simulated by the model software will come out to be within the carrying capacity of the canal at any time.

Shortage of trained manpower. Training is a costly investment. Moreover, the money invested is sometimes not utilized properly because of inefficient training methodologies, nonchalant attitude of trainees, and the fact that it is practically impossible for trainers to provide trainees on-site experience of canal operation problem scenarios. With a hydraulic model software which can simulate all these problem scenarios, trainees can develop skills and gain experience in a shorter time and in a cheaper way. Moreover, farming operations will not be disturbed nor hampered.

Improper utilization of irrigation water by the user. Farmers tend to overirrigate their fields whenever they get the chance not mainly because they want to control weed growth without resorting much to herbicides; nor because they want to spread their operations over time so that they and their families can provide the labor without hiring others; nor because they believe that flowing water which is cooler produces better yields than warm water. The main reason is that they see continuous flooding as an insurance, that if the field is full and is continuously replenished the risk and anxiety due to non-availability of water during the next schedule are reduced. Thus, the reliability of the system should be improved to encourage farmers to utilize irrigation water properly. The use of the hydraulic model software will be a useful tool in improving the reliability of the system.

Typical hydraulic controls

Control structures and turnouts, particularly the in-line control structures, need to be calibrated so that the value of the discharge coefficients can be determined. Calibration activities for control structures and turnouts should take place under steady flow conditions. Some of the adjustable and non-adjustable control structures that are common in irrigation systems of the region are discussed below. The equations which describe the flow characteristics of these control structures are provided in Appendix A.

Weirs. Weirs can be either adjustable or non-adjustable, the difference being that non-adjustable (fixed) weirs are not eligible for gate scheduling or manual operation. For a sharp-crested rectangular weir, the discharge over it is proportional to the flow depth over the crest raised to the power of 1.5. It is often found that when no water flows over the weir, the value of flow from the calibration curve is not equal to zero. That value is called the correction factor.

Culverts. Culverts are non-adjustable circular or rectangular control structures which normally operate under submerged conditions. Their performance characteristics are similar to siphon structures.

Constant head orifice. The adjustable control structures have either rectangular or circular orifices which may be operating in the free flow or submerged flow regimes. The CHO however, is essentially a submerged orifice measuring device. The upstream or orifice gate controls discharge while the downstream or turnout gate controls the submergence of the upstream gate. As a means of standardizing the device, it is arbitrarily decided to always submerge the orifice gate sufficiently to produce a 6 cm difference in water surface elevation or differential head across the upstream or orifice gate. The CHO is usually operated by opening the orifice gate for the desired discharge obtained from the discharge tables. The turnout gate is adjusted until the differential head across the orifice gate is at the required constant head of 6 cm.

Submerged flow. In most of the distribution systems in Thailand such as the Chao Phraya project, Mekong project and Lam Nam Oon project, the flow through the control structure occurs under submerged conditions due to the small bed slope of the channels. The operation of a control structure affects the downstream control structure in the same canal. The relationship between discharge coefficient and the ratio of the difference of downstream head and the sill elevation to the gate opening of a control structure or turnout is linear throughout the normal operational range when plotted on logarithmic paper. An example of such a control structure is the vertical sluice gates. In some cases, particularly in Thailand, these sluice gates are constructed with side weirs the flow characteristics of which can be described by a sharp-crested rectangular weir.

The Thailand Scenario

Thailand possesses the largest potential for irrigation development in Southeast Asia. A large number of large and medium scale projects have been constructed in the past and many of them are reported to have faced operational problems resulting in a low efficiency of water use. Plusquellec and Wickham (1985), classified the common operational problems of irrigation systems in Thailand into five, namely: canal system design; canal size; operation at full capacity; type of control structures ; and terminal distribution point. The following discussion is based on their book.

Canal System Design: Until the early 1970s, canal systems were primarily used for supplemental wet season irrigation and they generally achieved this objective. More recently, the Royal Irrigation Department's (RID) designs were patterned on the design standards of the U.S. Bureau of Reclamation, but were less comprehensive. The RID designs are acceptable in a situation of abundant water during the wet season, but pose several operational problems for dry season irrigation.

Canal Size: Until the early 1970s the main and lateral canal systems were designed for peak demand during the wet season. The design capacity in the Northern Chao Phraya was 0.13 l/s/rai (0.81 l/s/ha) for the canals and 0.14 l/s/rai (0.91 l/s/ha) for the structures. This is only about half the capacity needed to accommodate the 0.24 l/s/rai (1.4 l/s/ha) required to achieve a 100% cropping intensity for paddy during the dry season. The present transit capacities thus constrain the allocation of dry season water. Without remodeling part of the main and lateral systems, it would not be possible to supply enough water for dry season irrigation of paddy over the planted area (assuming use of an annual rotation method which provides water for one dry season out of two or three, on half or one third of the total project area). The most recent projects have been designed to avoid this problem, and capacities have been calculated on the basis of the average dry season cropping pattern. A margin of additional capacity is even provided to allow flexibility in the choice of cropping patterns. In the Mae Klong Project the design capacity of about 1.7 l/s/ha is adequate for cultivation of rice and sugarcane on 80% of the project area.

Operation at Full Capacity: Until recently the canal systems including the tertiary systems were designed to be operated only at or near full capacity with a minimum of control structures. The result is that at flows below the design capacity, water levels in the canals may not be high enough to ensure command of the entire service area and to provide full supply to the subsequent order canals. In principle, the solution is to build additional control structures to ensure a minimum water level at the diversion points independent of the flow in the parent canal.

Type of Control Structures: Operational staff is needed to control the water levels and flows of all structures built in the irrigation canal system. With the exception of some main regulators in the northern Chao Phraya area which are electrically controlled, all structures are manually operated and any significant variation in supply or demand of water in the system requires manual readjustment. Almost no form of automation has been built into the design of these structures. The maneuverability of gates equipping head and cross regulators is, in some cases, limited and any change in their setting is time consuming. The determination of flows through each structure requires calibration of the gates and frequent measurement of water levels in the absence of automation. In recent years, RID has initiated a program of gate calibration.

Terminal distribution point: The last water distribution control point is the farm turnout delivering water to service areas averaging 30 to 60 ha. Until the late 1960s, farm turnouts were equipped only with simple gates. This prevented any flow measurement, which was in any case unnecessary at that time because the systems were intended for wet season irrigation only. Constant head orifice (CHO) gates and movable weirs (Rominj type) were introduced in the late 1960s and 1982, respectively, to control and measure flows through the farm turnouts. These two devices, especially the movable weirs, are sensitive to variations in the upstream water level and therefore require frequent adjustments to maintain constant flow deliveries to the service units. Both devices are designed to measure the flows at a given time but are not self-flow controlling. An additional disadvantage to the CHO gate is its relative complexity of operation and need for frequent readjustment with the result that CHO gates are rarely used properly. The Rominj weir has so far been introduced only in the Mae Klong Project. Although simpler to operate, its use is also limited to canal sections with strict control of water levels.

The following conclusions have been drawn from the past studies of Thai irrigation systems (Plusquellec and Wickham, 1985; Paudyal and Loof, 1988).

Operation of existing control structures, either the main regulators or farm turnouts, is an unwieldy task. Day-to-day regulation of water levels and flows in the distribution network involves frequent routine setting of all gates because of the unstable hydraulic conditions in the systems. RID has been successful in organizing routine operations of the head and main cross regulators in the Chao Phraya project, but less so in operating main structures in other projects. As for minor canals and farm service areas, no water control has been organized except for a few pilot scale projects. This failure is related to the cumbersome operation of a number of farm turnouts and even more to the unreliability of existing devices. These factors often cause farmers to interfere in farm turnout operations and ultimately to remove the gates. The combination of all these factors places an upper limit on the level of performance which management and operational staff are able to achieve. The findings of the Irrigation Program Review in 1976 are still generally valid: "Water control within most irrigation projects leaves a lot to be desired". With the exception of the main system, there is little or no control on water flows. In general, the distribution system controls neither the timing nor the amount of water delivered and the on-farm water management practices are wasteful of water.

In addition, unclear operational manuals offering excessive information and unnecessarily complex procedures for the operation of simply designed control equipment are an obstacle to the training and subsequent motivation of operation staff.

The Philippine Scenario

In the Philippines, most of the facilities constructed before the 1970's are of the run-of-the-river diversion types, relying on the direct diversion of unregulated streamflows to supplement the water requirements of the wet season rice crop. Such systems were then incapable of supporting dry season cropping. In 1964, the National Irrigation Administration was created to improve the standard of the existing systems and to increase the irrigated area. Its function is to investigate, improve, construct and operate all national irrigation systems in the country. The design concepts and criteria used for these systems, however, have been largely borrowed from other countries. A summary of the present design concepts and their origins is shown in Table 2.

Design is critically important for two reasons. Firstly, design parameters determine the size and cost of the system infrastructure. Secondly, when design does not fit locally achievable levels of performance, facilities deteriorate, production decreases and recurrent and rehabilitation costs rise.

The present set of design concepts and standards are presently inappropriate because of the way they are applied, i.e. without the full appreciation of the implications for water control (Wensley, 1989). However, this does not mean that the concepts are wrong, rather, that they should be more selectively chosen and applied. Nevertheless, the systems do operate, although not as designed. This indicates that although the control structures are operated incorrectly, they are able to deliver a discharge resembling the required flow. This is due to the operator's experience rather than any formal set of rules that provide the key to system adjustment. However, very little of this experience is ever recorded so that others can operate the system.

Wensley (1989) identified some of the problems that are present in three Philippine irrigation systems. It can be safely assumed that these problems are representative of the present state of Philippine irrigation systems.

1. Irrigation system inflows fluctuate considerably, particularly during the wet season. These can cause highly variable canal discharges particularly if the gates are not correctly set to anticipate the changes in conditions.

2. A large number of control structures do not perform as they were designed because of broken or missing components, poor design (under sizing, for example) and poor siting within the canal. This severely affects the operation and performance efficiency of other structures, and consequently, the system becomes impossible to adjust as it was designed.

3. Canals are designed to flow within a constant range of depths. In practice, this requirement is not observed, and both cross sections and their hydraulic properties change due to weed growth, siltation or scour. Canal embankments erode and freeboard is lost, reducing the maximum flow the channel can convey.

4. Farmer checking and the construction of extra turnouts makes it virtually impossible to maintain design control conditions. Not only do the checks and turnout create considerably variable water level, when the checks wash out, the debris causes canal and structure blockages downstream.

The operational objective of early irrigation systems in the Philippines was simply to provide adequate amount of water on a continuous basis to the farm level in the wet season. However, during the 1970's, daily rotation of irrigation deliveries within the turnout service area was introduced. At the main distribution system, additional structural facilities for control were provided to maintain continuous flow. The rotational method created a lot of operational problems and many questions were raised as to its applicability in the Philippine setting.

Sediment-related problems are prevalent in many of these national systems. Sediments reduce the carrying capacity of the canals and make the operation of sluice gates difficult particularly at the diversion dam. The desiltation process disturbs the delivery schedules of the system and involves a considerable portion of the operation and maintenance budget of the project. The installation of sediment ejectors at the system headworks requires the diversion of larger volumes of water at the headwork. This means that water availability relative to diversion requirement will be less than expected not only because of lower than expected river flows but also because of larger required diversion due to heavy siltation and lack of sufficient desilting capability and maintenance.

Many systems have substantial deficiencies in the type, number, location, design and maintenance of gates and other structures. Such deficiencies make the system difficult to manage. As an illustration, flumes and double-gated turnouts are widely used in Philippine systems. These flumes are often tampered with or destroyed by farmers because they are perceived to limit the flow into their fields. In the same manner, with illegal tapping of canals prevalent upstream, the water level in the canal above the double-gated turnout constantly fluctuates making it nearly impossible for farmers to check whether the distribution of water among turnouts along the same supply canal is equitable. This motivates farmers to tamper with or adjust the turnout gates in an uncoordinated manner. Under these circumstances effective water management from system intake down to field plots becomes extremely difficult.

The situation is worsened by the fact that in most of the national systems, control structures are not operating according to the mode for which they were designed. The gates of the control structures are frequently broken or missing, and stoplogs are rarely provided. Control gates typically suffer from very poor detailing and suffer from problems with the lifting mechanisms and the guide tracks for the gate. Also, gates are the prime targets of farmers who wish to re-adjust deliveries. Considerable damage to gates is observed as a result of unauthorized adjustment, by using incorrect lifting gear, by forcing the gate open, or by intentional damage to keep the gate permanently open or closed.

Rehabilitation of these structures has been limited to restoration to their original conditions. However, this approach fails to incorporate the significance of the original design assumptions and the changing physical and organizational character of the systems over time. Moreover, it fails to correct for changes in hydrology and water availability, and bureaucratic and institutional constraints. There seems to be an apparent inertia in the design process that perpetuates the use of similar sets of design criteria each time a new system or rehabilitation project is started. An appropriate solution to this problem would be a comprehensive appraisal of how projects actually operate followed by a rewriting of irrigation design and rehabilitation criteria.

Since 1978 NIA has been promoting a participatory approach. However, considerable work has still to be done to make it fully effective under actual conditions. While there are indications that farmers are actively involved in construction, maintenance and repair

Table 2. Design concepts of Philippine irrigation systems and their origin.

Design Criteria	Implementation in Philippine Systems	Origin
Method of water delivery	Main system- continuous flow Tertiary level - rotation within turnout service areas	USBR - Western, U.S.A. Taiwanese irrigation practices
Water duties	Landsoaking and maintaining standing water during crop growth	NIA-ADB pilot areas and IRRI rice production manual
Water control in canals	Main system - design water levels maintained for continuous flow Tertiary level - rotation using division boxes	USBR - Western, U.S.A. USBR - Western, U.S.A.
River diversion structures	Generally ogee-crested weirs with sluice gates for silt extraction	Indian type-design
Canal regulators	Gated cross-regulators, often combined with bridges or canal crossings	USBR - Western, U.S.A.
Turnout structures	Single- or double-gated pipe turnouts through canal bank (requiring constant level for efficient operation)	USBR - Western, U.S.A.
Tertiary level facilities	Main and supplementary farm ditches with concrete division boxes, ditches often laid out in a grid pattern	NIA-ADB pilot areas and special projects
Measurement structures	Parshall flumes provided at heads of laterals, and staff gauges below turnouts	USBR - Western, U.S.A. and NIA designs
Density of facilities	Prescribed for all types of facilities, including canals, regulators, turnouts, tertiary facilities, roads, bridges and drainage works	various including Japan, U.S.A., India and Taiwan

Source: Wensley, 1989.

work, conflict resolution and water distribution, there was still hardly any direct participation in design activities, and in fee collection. Moreover, farmer participation in water scheduling and delivery is not a countrywide occurrence but varies according to the type of management adopted in the system.

The Nepal Scenario

Irrigation systems in Nepal can be represented by two major types, those that are located in the Terai region, and those that are situated in hills. The Terai region covers only 17 percent of the land area of the country but has two-thirds of the cultivated land. It is the strip of the Gangetic plain that lies along Nepal's southern border with India. On the other hand, the systems located in the hills are traditionally communal systems developed and managed by farmers for centuries.

Irrigation development in Nepal can be classified as either the construction of new systems to irrigate new areas or the rehabilitation of existing systems to expand existing areas or to irrigate winter crops. These strategies include activities such as the improvement of the main canal system including lining of some portions of the canal; and construction of turnouts, secondary canals and headworks. However, because of the topographic peculiarities particularly of systems located in the hills and to some extent in the plains of the Terai region, a lot of management problems have evolved related mainly to system design. An example which is an indirect effect of the topographic variations is the very distinct seasonal variations in climate which in turn causes very large changes in river discharge between the wet and dry season. In the Chandra irrigation system in the Terai region for instance, for a design flood of 3400 m^3/s, the headworks capacity is designed at 14.2 m^3/s. However, the typical dry season flow in February is 2 m^3/s which is only 15 percent of the main canal capacity. This creates a lot of problems in operating the system.

As indicated, many of the common operational problems are related to the design inadequacies of the system. These inadequacies can be traced from the canal system design; canal alignment and size; types of control structures; and design of secondary and distribution canals (Tiwari, D.N., 1987).

Canal system design. Canal system design involves arranging and fixing the source location; canal alignment and size; structures for water control and measurement; and on-farm facilities. Performance of the whole system is usually dependent on the extent to which design of the system reflects local needs. While designs of medium scale irrigation projects are based on standard design practices, both design and construction problems have arisen due to lack of knowledge of the local conditions.

Canal Alignment and size. In most cases the improved canal system follows the original farmer's canal network with or without changes in canal capacity. Except in some of the minor scale systems, the determination of canal capacity is based on soil type; canal conveyance loss; crop water requirements; rotation method of water distribution; and command area as 85% of gross command area. In case of farmer managed irrigation systems (FMIS), canals were built for high conveyance capacity due to the i) preference for continuous cold water instead of rotational practice; ii) entry of first flood water which increases soil fertility; iii) sufficiency of water at the source; and iv) high percolation due to the slope towards the river.

Control structures. Beadle et al.,1986 identified the inadequacy or inappropriateness of irrigation structures which make management of irrigation difficult as one of the main constraints on agricultural productivity in the Terai region of Nepal. Normally, head regulators are proposed in medium scale irrigation projects. The maximum water release capacity for these head regulators is 2.0 m^3/s. Escape structures are present although in the original design, these structures were not included.

Secondary canals and distribution canals. Upstream control in secondary and tertiary canals are proposed to regulate the flow by fixing the inlet size. The design does not provide any solution for controlling the discharge during fluctuations in water demand. In most of the systems, the main canal is supposed to supply water directly to the farmer's original branch canal system.

Typical Case 1. The Phitsanulok Irrigation System

The Phitsanulok irrigation project consists of the concrete diversion dam constructed across the Nan river and a distribution system which can provide water to both banks of the Nan river covering parts of three provinces of Thailand (Figure 1).

The project development is divided into two stages. Stage 1 which has been completed already involved the construction of a complete irrigation system on the right bank of the Nan river (92,000 ha) and on the upper left bank or Thung San area (15,200 ha) for a total

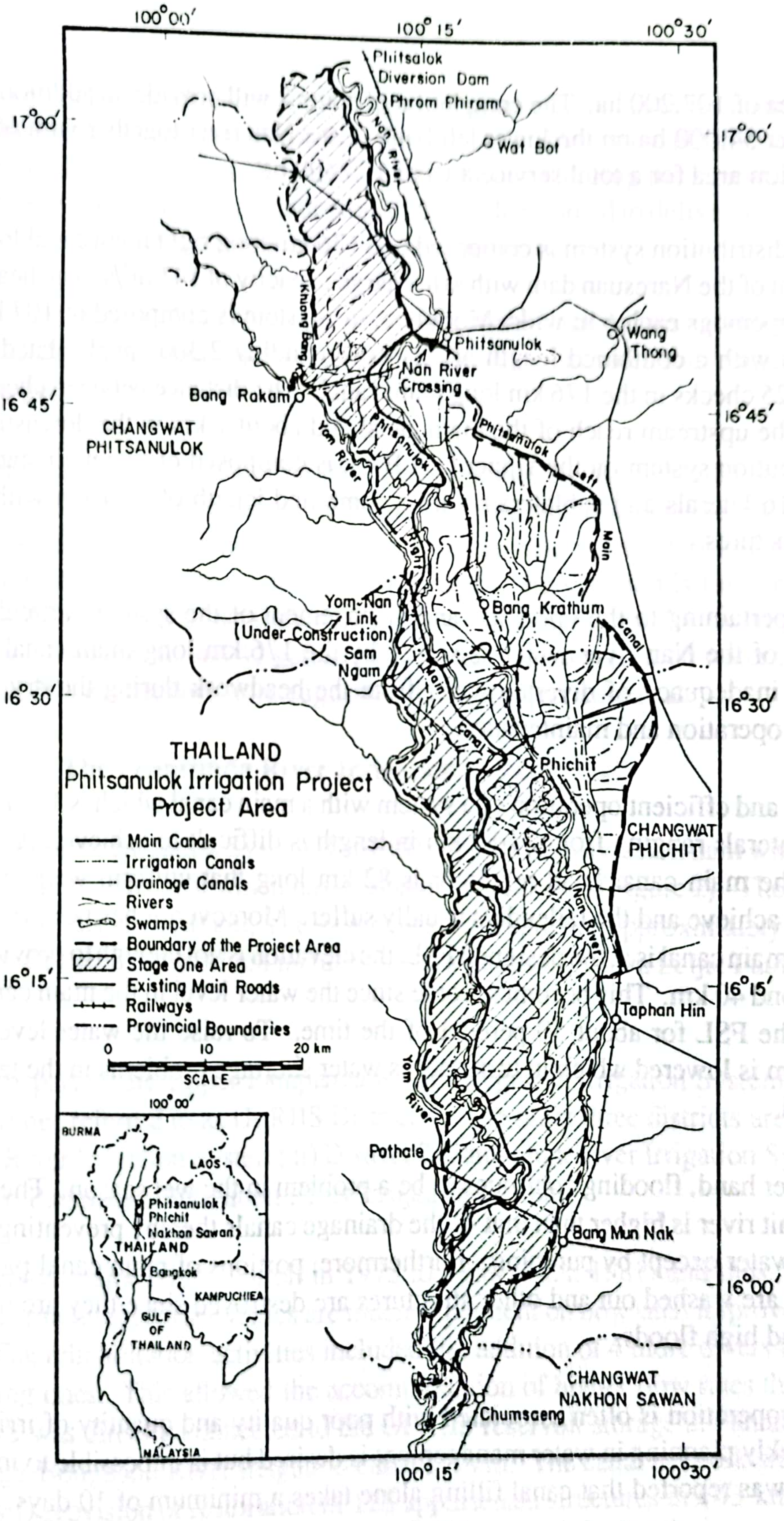

Fig. I General Layout of Phitsanulok Irrigation Project, Thailand service area of 107,200 ha. The completion of stage 2 will provide an additional irrigated area of over 54,000 ha on the lower left bank of the Nan river together with 66,400 ha of the extension area for a total service area of 120,400 ha.

The main distribution system is composed of a 176 km long right main canal located 1600 m upstream of the Naresuan dam with a discharge capacity of 141 m^3/s. The head regulator has three openings each 6 m wide. Moreover, the system is composed of 107 laterals and sublaterals with a combined length of 721 km including 2,300 canal related structures. There are 25 checks in the 176 km long main canal. The distance between checks is about 10 km in the upstream reach of the main canal and about 7 km in the downstream reach. The distribution system on the Upper Left Bank is composed of two main canals of 42.8 km each, 16 laterals and sublaterals with a combined length of 74.7 km with 360 canal related structures.

Problems pertaining to the operation and management of the system particularly on the right bank of the Nan river which is served by the 176 km long main canal are mainly caused by inadequacy of diverted water from the headwork during the dry season and inefficient operation and maintenance.

Controlled and efficient operation of a system with a main canal which is 176 km in length and with laterals ranging from 24-82 km in length is difficult to achieve. A lateral at 80 km from the main canal head regulator is 82 km long that uniform water allocation is difficult to achieve and the tailenders usually suffer. Moreover, it has been found out that even if the main canal is at the designed FSL, the elevation is not enough to provide adequate water beyond 40 km. This becomes worse since the water level in the main canal is 20 cm less than the FSL for about 50 percent of the time. To raise the water level the check downstream is lowered which in turn causes water shortage problems in the tailend of the main canal.

On the other hand, flooding continues to be a problem in the wet season. The water level in the Pitchit river is higher than that of the drainage canals thereby preventing evacuation of excess water except by pumping. Furthermore, portions of main canal particularly in the tailend are washed out and other structures are destroyed since they are not designed to withstand high floods.

Inefficient operation is often associated with poor quality and quantity of irrigation facilities. Weekly planning in water management is desired but is impossible to implement in reality. It was reported that canal filling alone takes a minimum of 10 days. During the dry season, a complete breakdown of the irrigation water plans ensues because of illegal planting of unprogrammed areas. Farmers at the start of the dry season are able to cultivate unprogrammed areas by using shallow tubewells. However, during the later stages of the crop, these tubewells dry up and irrigation officials are then forced to deliver water to these to sustain the growth of crops in these unprogrammed areas.

The lowering and raising of the head regulator of the main canal causes severe water level fluctuations in the main canal reach. Normally, canal flow becomes stable again 10 days after the change.

Communication is likewise a major problem. Communication is mainly by walkie-talkie which can be used for a mere distance of 7 km. Only the watermasters are equipped with these instruments. Communication among zonemasters and gatetenders is facilitated by the use of bicycles and in some cases of motorcycles. The situation is further worsened since the whole system is understaffed. In one of its subprojects for example, only 3 watermasters, 16 zonemen, and 45 gatetenders are assigned whereas 10 watermasters, 79 zonemen, and 300 gatetenders were suggested during project appraisal.

Typical Case 2. The Penaranda River Irrigation System

The Penaranda River Irrigation System (PenRIS) consists of a diversion dam which diverts water to a 98.1 km main canal which has a capacity of 41.1 cms (Figure 2). The combined length of the laterals is 108.0 km. The system serves an area of approximately 25,300 ha of riceland in Central Luzon which includes the provinces of Nueva Ecija, Pampanga and Bulacan.

The system is part of the Upper Pampanga River Integrated Irrigation System (UPRIIS) and is sometimes referred to as UPRIIS District 4. The other three districts are i) District 1 : Talavera River Irrigation System; ii) District 2: Pampanga River Irrigation System; and iii) District 3: Pampanga-Bongabon River Irrigation System.

PenRIS received major rehabilitation in 1975 to 1980 (NIA, 1981) and thus, its present physical and structural characteristics are much dependent on how such improvements are sustained. The rehabilitation activities included the addition of 4 more diversion gates to the 12 existing ones. This allowed the accommodation of higher flow rates that resulted when PenRIS was directly connected to the UPRIIS reservoir storage at Pantabangan via the Pampanga-Bongabon River irrigation canal system. The canal network was likewise improved by i) provision or restoration of 128 appurtenant structures in a 43-km stretch of

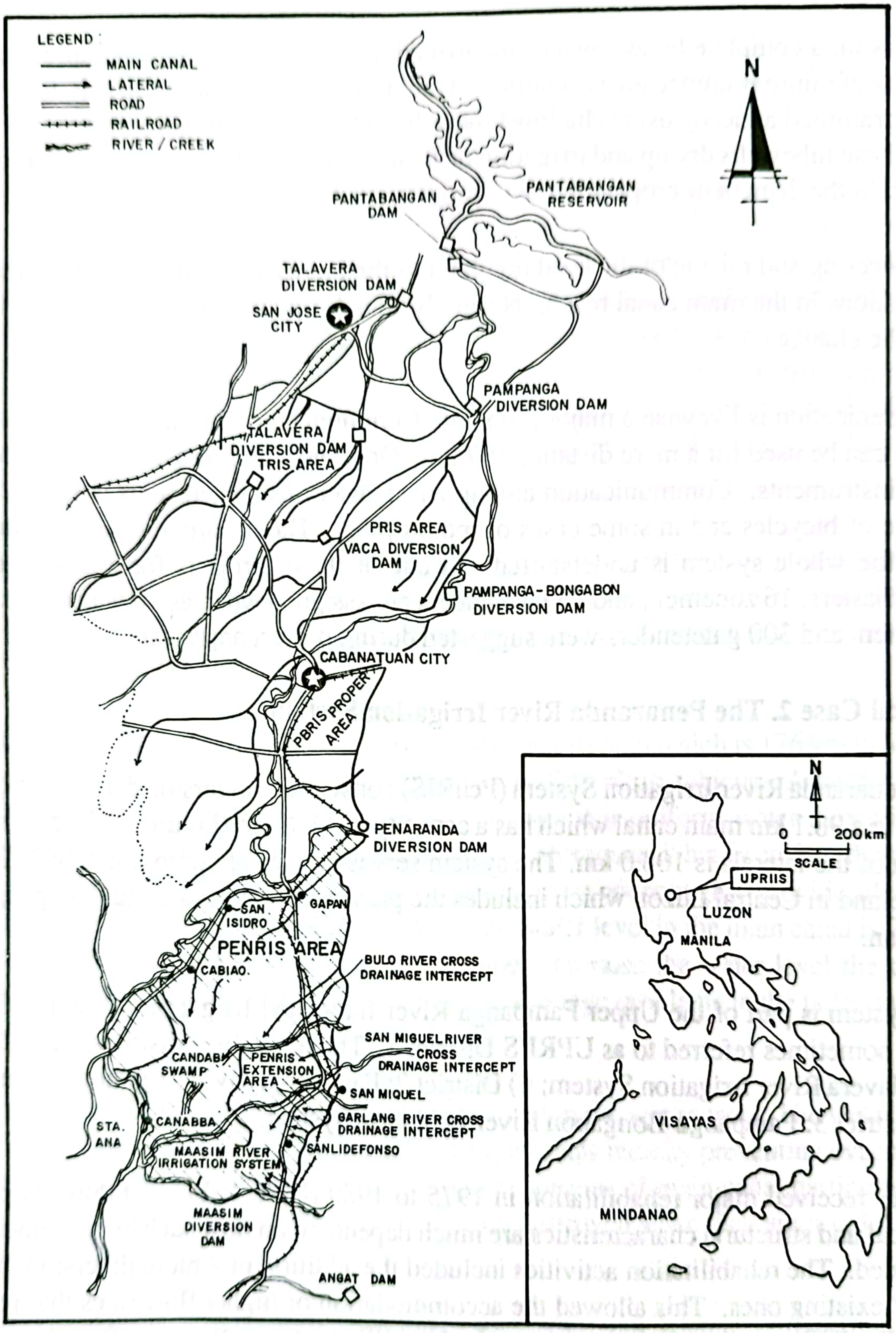

Fig. 2 General Layout of Upper Pampanga River Irrigation System, Philippines

the main canal; ii) lining for scour protection of the main canal at portions where it is required; iii) construction of 95 distributory canals with combined length of 416 km; iv) provision of 1,241 structures of various types, including structures for measuring flow; v) construction and improvement of 29 drainage canals with combined length of 92 km; and vi) development of 683 rotational areas with turnout structures, main and supplementary farm ditches, measurement devices, division boxes, offtakes and end-check structures. Other activities included the construction or improvement of 437 km of O&M roads along the embankment of the main and lateral canals; and the construction of service buildings such as staff quarters, training centers and warehouses.

The primary adjustable structures located at the headgates of laterals and each turnout service area play the most important role for controlling water. Several check structures were nevertheless provided at intermediate points along the canals in support of these primary control structures. However, the physical and structural conditions of the gates within the system had deteriorated considerably since the completion of the rehabilitation program in 1980. The gates are frequently broken or missing due to unauthorized adjustments. In such cases stop logs are rarely provided. Similarly, at the turnout level, not a single double-gated turnout was operating as designed, i.e they are used as single-gated orifice turnouts, gates are incorrectly set in relation to operating water level elevations; gates are missing, broken or full of obstructions. Because of this many farmers had constructed extra illegal turnouts and pipe outlets which made water distribution more difficult. The magnitude of the problem has grown to a point wherein out of the 231 control points provided, NIA presently retains effective control over only approximately 17 points which are found at the headgates of sublaterals and regulate deliveries to areas of 500 to 800 has. These indicate many constraints during the operation of the system. A large number of these constraints can be linked directly to the choice of the design of system facilities, and the high degree of operational sophistication envisaged when the system is rehabilitated. Although system operators appear to cope remarkably well with these problems, the question is raised concerning the appropriateness of the design guidelines. (Wensley, 1989).

Continuous irrigation is practiced in main and secondary canals while rotational irrigation is used along the main farm ditch. The size of a rotation area is approximately 50 hectares and is further subdivided into rotation units of about 10 ha each. Each rotation area is served by a gated turnout which can discharge as much as 75 liters per second, the approximate water requirement of 10 hectares for 5 days. This amount of water is delivered within a 24-hr period, after which it is delivered to the next rotational unit until the 5-day rotation cycle is completed. Irrigation plans are solicited from these rotation areas and are integrated progressively to form a seasonal allocation plan for the whole system. However, it has been observed recently that this procedure is not working satisfactorily under the prevailing conditions, so the allocation procedure is relaxed to one based on simultaneous distribution within rotational areas.

PenRIS as in other national systems, has adopted the twin strategy of providing the services of trained water management technicians (WMTs) and promoting the organization of irrigator groups (IGs). This strategy aims at improving water discipline among the users so that the water delivery schedule can be maintained, and thus achieving an optimal distribution and allocation of water to meet the irrigation requirements of the service area.

WMTs provide a direct contact point between the project management and the farming community at large. Their primary function is to implement the plans and programs on water delivery and distribution within the assigned area of jurisdiction comprising up to ten rotational areas of approximately 50 ha each. They are also responsible for organizing maintenance and repair of farm-level facilities and structures. Training in water management is provided to the water users on a continuous basis, and the farmers are given guidance on irrigation and related matters. Another major function of a WMT is the gathering, analysis and evaluation of field-level irrigation related data on a holding by holding basis in the area under his responsibility. These data serve as feedback to system management, culminating in the drafting of a seasonal irrigation plan for his area, in consultation with the farmers, as an input to the preparation of a system-wide water delivery schedule that would be compatible with the intended cropping pattern and farming activities of the water users.

Typical Case 3. The Chandra Irrigation System

The Chandra irrigation system lies between latitudes 26.45 °N and 26.75 °N and longitudes 86.50 °E and 86.90 °E. It is located in eastern Terai of Saptari District, Sagarmatha zone under the Eastern Develoipment Region of Nepal. It has a service area of approximately 10,000 ha (Figure 3). The headwork is a vertical weir which is compose of scour sluices and a head regulator. The main canal is 28.60 km long and was built 50 years ago. However, most of the distributaries and gated structures have been constructed more recently. Apparently, the original design concept was to spread the available wet season flow over a wide area to benefit as large a number of farmers as possible. However, based on more modern design criteria, the present command area is too large relative to the available water supply. Beadle et al. (1986) have done a comprehensive documentation of the system. The subsequent discussions are mainly based on their findings.

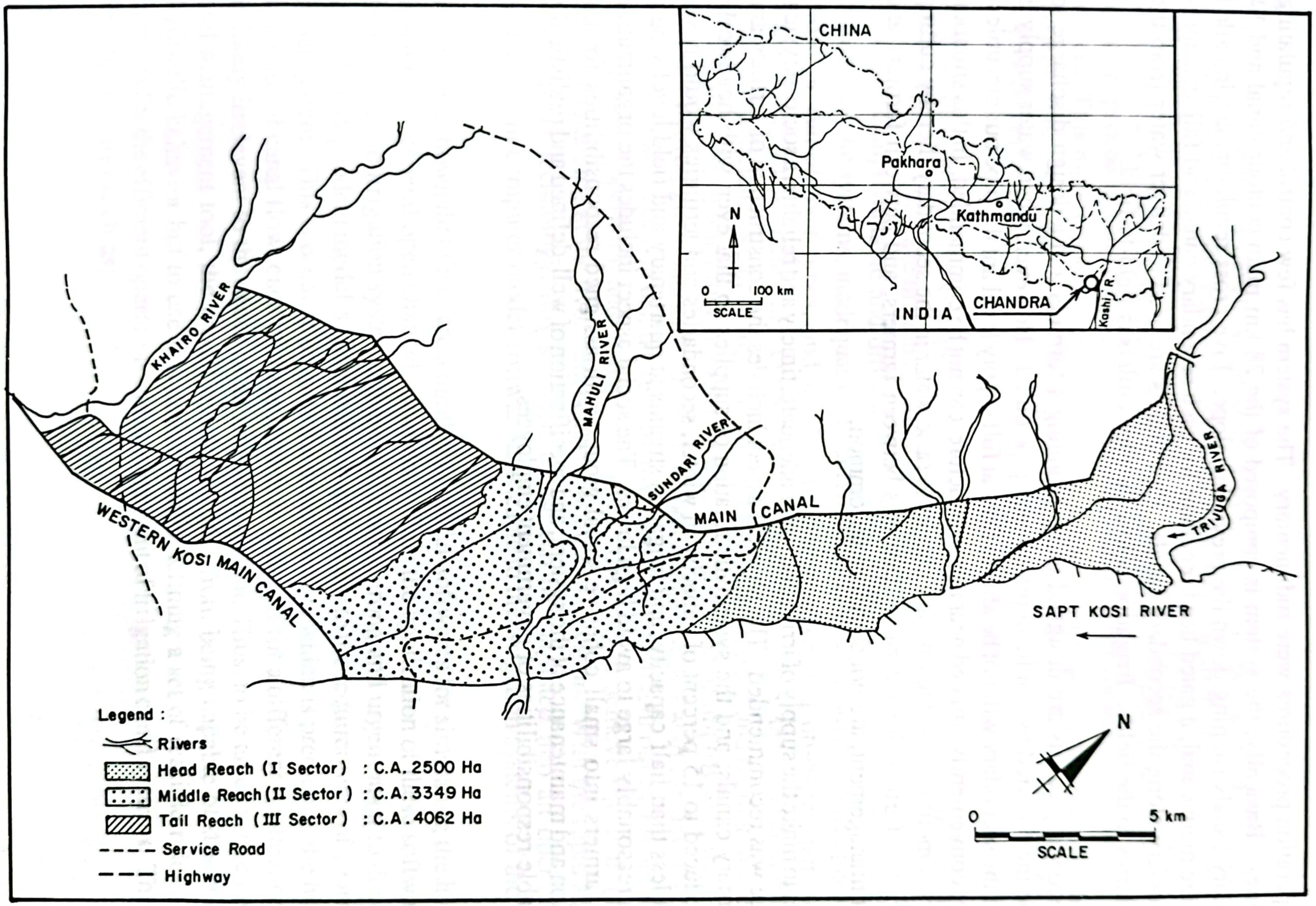

Fig. 3 General Layout of Chandra Canal Irrigation Project, Nepal

The operation procedures were rudimentary. The system has few control and regulating structures. Basically, the system is composed of the 28 km main contour canal and 14 secondary canals running down the prevailing slope. Five cross regulators are installed along the main canal, a gated head regulator for each secondary canal, and fall structures as necessary along the secondary canals. Pipe outlets are used to deliver water from the secondaries to the fields. Irrigation below the pipe outlets is field-to-field.

The secondary canals are designed using Manning's formula and a design discharge to provide the required hydraulic depth of each pipe outlet. However, when water supply is limited the secondary will not be able to flow at full supply level ant the pipe outlets, unless they are conveniently situated near a drop structure, can not be commanded. In this situation farmers sometimes build temporary checks to raise water in the secondary thereby cutting off supplies further downstream. Disputes between farmers, and between farmers and project management, are not therefore uncommon.

In order to make the supply of irrigation water adequate, timely and reliable, a socio-physical package was recommended. The physical aspect includes the construction of tertiary and quarternary canals; and the systematic rotation of supplies so that even with headworks flow reduced to 15 percent of capacity in winter, secondaries and tertiaries would never flow at less than half capacity, and irrigation streams at quartenary and field level would remain reasonably large to avoid high losses. The social aspect includes the organization of the farmers into small cohesive social groups with assigned responsibilities for the operation and maintenance of the system; and specification of well-defined and realistically achievable responsibilities of government employees.

SUMMARY

Recent research and development studies to improve performance of irrigation canal systems have shown that emphasis has been shifting from the strengthening of the hardware part which includes physical restorations of less efficient performing infrastructural components to the software part which involves the development of operation and management procedures and strategies appropriate to the present hydraulic state of the irrigation system. This monograph concentrated on the software approach particularly for the main system. It has been shown that the improvement of the main system is the key to an overall improved performance of the whole irrigation system.

A typical irrigation canal system in South and Southeast Asia can be considered relatively more difficult to manage than in the more advanced systems of the world since canal flow is far more unpredictable and difficult to control. This is because of the hardware shortcomings of the systems, inadequacy and unreliability of water from the source particularly those of run-of-river types, and inefficient, inappropriate and obsolete operation and management rules mainly due to traditional restrictions. While the undesired effects of hardware deficiencies can be minimized by physical restorations and regular maintenance works; and system reliability and relative water adequacy can be improved through the use of better hydrologic models for streamflow forecasting coupled with the development of sound water allocation plans; the real improvement can be brought out by revolutionizing the traditional operation and management rules. One way to achieve this is through the full understanding of the hydraulic behavior of the canal system. With the rapid advances in computer technology, this knowledge-base can be translated into an operation and management computer model-software applicable to the irrigation systems of the region.

It has been shown that there are a number of model-softwares for simulating the hydraulic behavior of general open channel networks. While the application of these softwares can be extended to irrigation systems, there still exist a strong and urgent need to develop an entirely independent model-software that is appropriate to the requirements of an irrigation canal system. This is because beyond the basic function which is predicting the hydraulic behavior of canal flow, each of these models is designed for a different application which is likely irrelevant to the needs of an irrigation system. Thus, to be an effective operation and management tool, the model should not stop from being capable of describing the hydraulic behavior but to use this description in defining a set of feasible rules and procedures for the efficient operation and management of irrigation canal systems. These may include features such as:

- provision for real-time operation and management
- simulation and evaluation of irrigation plans before actual implementation
- assessment of the ability of the system to deliver water according to proposed allocation schedules
- development of long term operational and management rules and guidelines such as establishment of emergency measures during floods and droughts; and evaluation of maintenance and rehabilitation programs
- evaluation of proposed canal system design and verification of canal hydraulic parameters
- tool for research and training

Finally, the model should be translated into a user-oriented management tool. This can be achieved by providing the following features: i) easy to use which can be achieved through interactive programming and provision of graphic interface; ii) sufficient documentation and user-friendly manuals; iii) modularity, i.e., facility in defining problem scope; and iv) user-decision control mechanism, i.e., facility in defining irrigation scenarios and provision of mechanism which allows the user to overrule or modify decisions according to his social, economic and political prejudices.

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APPENDIX A

THEORETICAL CONSIDERATIONS

The physical laws which govern the flow of water in a channel are:

1) The principle of conservation of mass, and
2) The principle of conservation of momentum

The differential equation describing conservation of mass is:

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=q \tag{1}$$

where Q and A are functions of distance along the channel x and time respectively; and q is the lateral flow in m^3/s-m. The equation is usually referred to as the continuity equation.

The equation describing conservation of momentum can be written as

$$\frac{\partial Q}{\partial t}+2\frac{Q}{A}\frac{\partial Q}{\partial x}-\frac{Q^2}{A^2}\frac{\partial A}{\partial x}+gA\frac{\partial y}{\partial x}+\frac{gn^2Q\,|Q\,|}{AR^{4/3}}-q\frac{Q}{A}\cos\phi=0 \tag{2}$$

where ϕ is the angle made by lateral with the main channel, and the lateral velocity is assumed as the main channel velocity. The above equation is usually referred to as the dynamic equation.

By neglecting the lateral flow and rearranging terms of (1) a more simplified expression can be obtained.

$$\frac{1}{gA}\frac{\partial Q}{\partial t}+\frac{1}{gA}\frac{\partial}{\partial x}\left[\frac{Q^2}{A}\right]+\frac{\partial y}{\partial x}-S_o+S_f+S_e=0 \tag{3}$$

The various terms in the dynamic equation are defined as follows:

$\frac{1}{gA}\frac{\partial Q}{\partial t}$ = local acceleration of flow

$\frac{1}{gA}\frac{\partial}{\partial x}\left[\frac{Q^2}{A}\right]$ = convective acceleration of flow

$\frac{\partial y}{\partial x}$ = pressure gradient term

S_o = bottom slope term

S_e = expansion or contraction loss term

The continuity and the dynamic equations are known to be the Saint-Venant equations. The Saint-Venant Equations are derived to describe the one-dimensional unsteady flow under the following assumptions:

1. The pressure distribution along a vertical line in the flow is hydrostatic
2. Friction losses in unsteady flow can be computed using steady-flow friction loss formulas
3. The velocity distribution across a channel section does not affect wave propagation
4. The water surface across the section is horizontal
5. The average channel bottom slope is small

As can be noted, the one-dimensional unsteady flow can be described by two dependent variables water stage y and the discharge Q at any given channel cross section. These dependent variables define the state of the fluid motion as a function of two independent variables x for space and t for time. Depending on the nature of the problem, other pairs of dependent variables such as Q(x,t), z(x,t); u(x,t), z(x,t); and u(x,t), y(x,t) can be defined. These other forms can be of practical importance since certain numerical techniques may be better adapted to some of these forms; or since physical features of a given water course may suggest the form which is more appropriate to use (Cunge et al., 1980).

In some cases, it becomes necessary to drop some of the terms in equation (3) to simplify the numerical computation. There are situations that these simplified variants are more appropriate. These variants can be categorized into three approaches namely i) kinematic wave approach; ii) diffusive wave approach; and iii) dynamic wave approach.

The kinematic wave approach assumes that the flow can be adequately described by a balance between the friction and gravity forces only, i.e. inertia term and hydrostatic term are neglected. This simplification is limited in the sense it cannot simulate backwater effects. However, it is appropriate in channels with sufficiently steep slope. The diffusive wave approach, in addition to the friction and gravity forces includes the hydrostatic gradient term allowing it to be used for simulating backwater effects. The dynamic wave approach is equivalent to the full dynamic equation. This approach can be used to simulate fast transients and tidal flows in the system.

The Method of Characteristics

The method of characteristics may be described as a technique where by the problem of solving two simultaneous partial differential equations can be replaced by the problem of solving four ordinary differential equations. This method makes use of the properties of the hyperbolic type, quasi-linear partial differential equations. The pertinent properties are: there occur two characteristic directions in x-t plane, along each of them there occurs one ordinary differential equation. The procedure for the solution by this method consists of locating the characteristics curves and then integrating the ordinary differential equations along the characteristic curve. In general, solutions of this method may be performed in two ways, graphical method and numerical approximation. The graphical method is tedious while the method of numerical approximation requires the use of computer. The numerical process can be performed on a grid of characteristics or at specified intervals of the independent variables. The latter method has the advantage that it gives results directly in the form which is most likely needed, such as, the hydrograph at each position along the channel and also the water surface profile at any given time (Winyawonk 1970).

The differential equations describing the unsteady free surface flow are:

$$\frac{\partial Q}{\partial x}+T\frac{\partial y}{\partial t}-q=0 \tag{4}$$

and

$$\frac{\partial Q}{\partial t}+\frac{2Q}{A}\frac{\partial Q}{\partial x}+\left[gA-\frac{Q^2T}{A^2}\right]\frac{\partial y}{\partial x}-\frac{Q^2T}{A^2}S_o-q\frac{Q}{A}+\frac{gn^2Q\,|\,Q\,|}{AR^{4/3}}=0 \tag{5}$$

where y is the flow depth, and lateral velocity is assumed as main channel velocity.

The equations of unsteady free-surface flow (4) and (5) can be written into a single matrix equation with four unknowns $\frac{\partial Q}{\partial x}$, $\frac{\partial Q}{\partial t}$, $\frac{\partial y}{\partial x}$ and $\frac{\partial y}{\partial t}$

$$\begin{bmatrix} 1 & 0 & 0 & B \\ 2\frac{Q}{A} & 1 & \left[gA\frac{Q^2B}{A^2}\right] & 0 \\ dx & dt & 0 & 0 \\ 0 & 0 & dx & dt \end{bmatrix} \begin{bmatrix} \frac{\partial Q}{\partial x} \\ \frac{\partial Q}{\partial t} \\ \frac{\partial y}{\partial x} \\ \frac{\partial y}{\partial t} \end{bmatrix} = \begin{bmatrix} q \\ -\left[\frac{gn^2Q\,|\,Q\,|}{AR^{4/3}} - q\frac{Q}{A} - \frac{Q^2B}{A^2}S_o\right] \\ dQ \\ dy \end{bmatrix} \tag{6}$$

Two characteristics equations can be obtained by setting the determinant of the matrix (6) equal to zero.

$$\left(\frac{dt}{dx}\right)_+ = \frac{1}{\frac{Q}{A}+\sqrt{g\frac{A}{B}}} = \alpha \tag{7a}$$

$$\left(\frac{dt}{dx}\right)_- = \frac{1}{\frac{Q}{A}-\sqrt{g\frac{A}{B}}} = \beta \tag{7b}$$

where β is the momentum coefficient.

By replacing one of the four columns on the left of (6) with the column on the right and setting the determinant equal to zero, two ordinary differential equations can be obtained.

$$\frac{dQ}{dx} + F\alpha\frac{dy}{dx} + G^+ = 0 \tag{8a}$$

$$\frac{dQ}{dx} + F\beta\frac{dy}{dx} + G^- = 0 \tag{8b}$$

where

$$F = gA - \frac{Q^2B}{A^2}$$

$$G^+ = \left[\frac{gn^2Q\,|\,Q\,|}{AR^{4/3}} + \frac{qQ}{A} - \frac{Q^2B}{A^2}S_o - \frac{q}{\alpha}\right]\alpha$$

$$G^- = \left[\frac{gn^2Q\,|\,Q\,|}{AR^{4/3}} + \frac{qQ}{A} - \frac{Q^2B}{A^2}S_o - \frac{q}{\beta}\right]\beta$$

Figure A-1 shows the two characteristic directions given by (7a) and (7b) at a point in the x-t plane. Equations (8a) and (8b) are the ordinary differential equations for Q and Z along the characteristics α and β, respectively.

Method of Characteristics with Specified Intervals

In this method, Q and Z are considered as known functions of x and t, either being as given initial conditions or as the results of previous stages of computations. For example, it is assumed that Q and Z are known along the distance x at time t. Figure A-2 represents rectangular grids in (x, t) plane with intervals Δx and Δt in x and t coordinates, respectively. In this case, Q and Z at points M1, A1, B1,..., N1 are known. The values of Q and Z at time $t+\Delta t$ particularly at the points A2, B2, C2,... can then be computed from the set of equations (7a&b) through (8a&b). At points M2 and N2, Q and Z can likewise be computed, respectively from (8b) and (8a) with the appropriate boundary conditions. Similarly, Q and Z at the time $t+2\Delta t$ at various points along the distance x can be computed. This process can be continued as far as desired.

Initial Conditions

The initial conditions required for the computation of the unsteady free surface flow problems are the values of discharge and stage at every grid point along the reach at the initial time of computation.

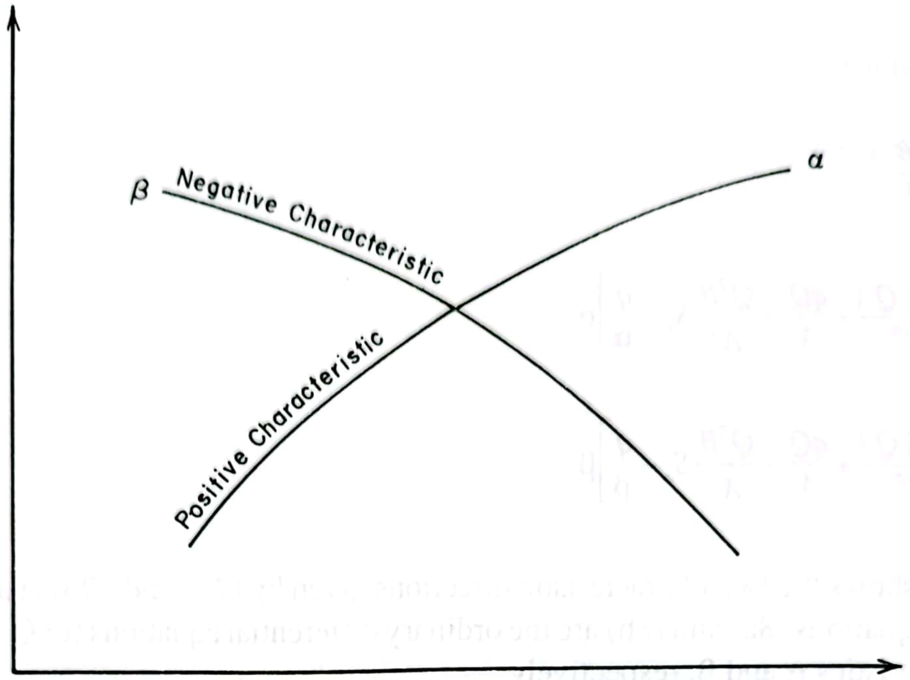


Fig. A-1 Characteristic Curves

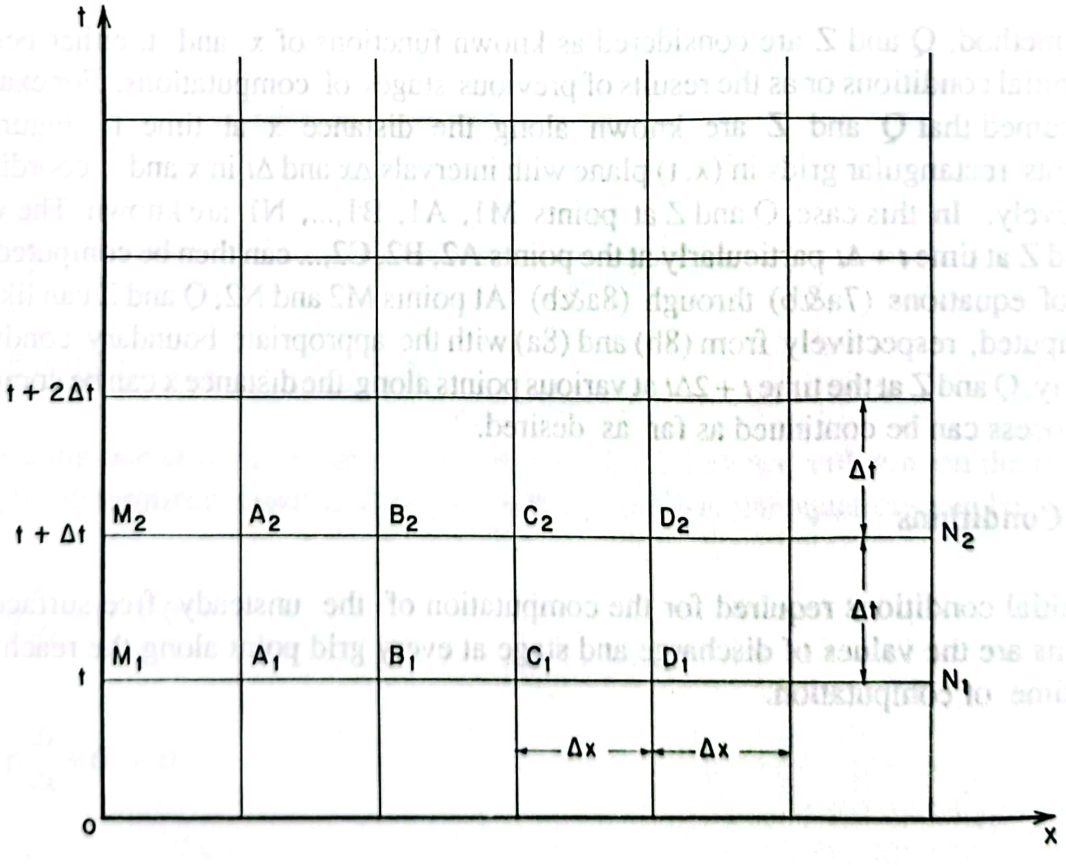


Fig. A-2 Network of Characteristics by the Method of Specified Intervals

Numerical Solution

In Figure A-3, α and β are two characteristics at P which intersect AC and CB at R and S, respectively. Denoting the value of x at R by X_R, and so forth, for known values of Q_A, Q_B, Q_C and Z_A, Z_B, Z_C, the values of Q_P and Z_P can be determined by applying the linear approximation to the set of equations (7a&b) through (8a&b). In this manner, Q and Z at all grid points except the two boundary points, one upstream and another one downstream, at the time $t+\Delta t$ can be obtained. For the specified intervals, the sizes of Δx and Δt can be defined by

$$\Delta t \le \frac{\Delta x}{\left| \frac{Q}{A} \pm \sqrt{g \frac{A}{B}} \right|}$$

It is assumed that Δt is sufficiently small that the parts of the characteristics between P and R and between P and S are straight lines. Then the positive characteristic at P, α_P, is the positive characteristic at C, α_C; and the negative characteristic at P, β_P, is the negative characteristic at C, β_C. In equation form,

$$\alpha_P = \alpha_C = \frac{PC}{RC} \tag{9a}$$

$$\beta_P = \beta_C = -\frac{PC}{CS} \tag{9b}$$

Boundary Conditions

Boundary conditions depend on the nature of the control structures such as dam, gauging station, weir and regulator. For the points at the boundary, only a single characteristic curve is used because only one unknown has to be determined.

At the upstream end, the available boundary conditions are usually the stage or discharge hydrographs. The β-characteristic as shown in Figure A-4a is used in calculating the unknown Q_P or Z_P depending on the type of the upstream boundary conditions.

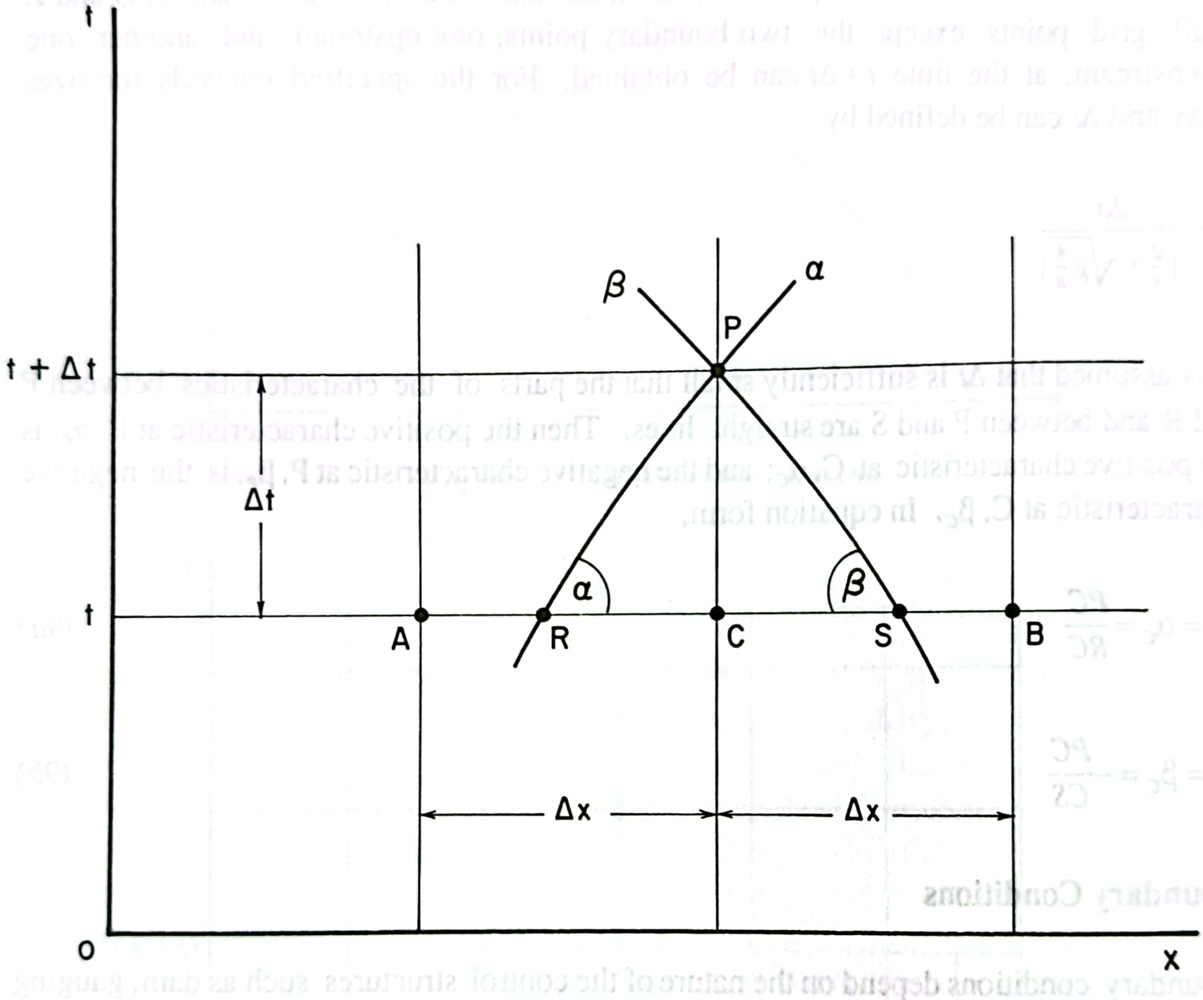


Fig. A-3 Two Characteristics on Specified Intervals

At the downstream end, the stage discharge relationships are normally used as the boundary condition. The α-characteristic as shown in Figure A-4b is used with the stage-discharge relationship as the boundary condition in computing the values of Q_P and Z_P.

The Finite Difference Method

There are two basic schemes of finite difference, namely the explicit scheme and the implicit scheme. In the explicit scheme the unknown quantities from the initial condition or from the previous calculation are used for forward calculations. In the implicit scheme the value at the advance time, which is still unknown for the time being, is entered into the formula for solution. Then the solution at the advance time is obtained either by iteration or by linearizing the nonlinear terms and solving the resulting simplified system of linear equations simultaneously with the boundary conditions.

The main advantage of the implicit scheme is that the stability of the solution is not limited by the condition that the grid ratio, $\frac{\Delta t}{\Delta x}$, must be equal to or less than $1/\left| Q/A \pm \sqrt{g\frac{A}{B}} \right|$.

Consider the continuity and the dynamic equation with no lateral flow.

$$\frac{\partial Q}{\partial x}+T\frac{\partial y}{\partial t}=0 \tag{10}$$

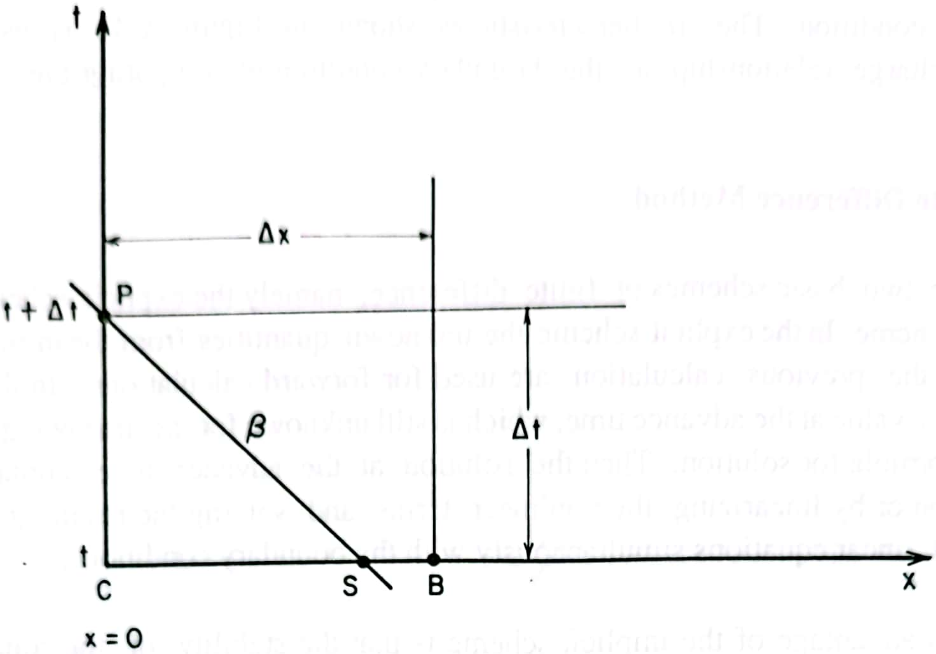
where y is the water depth

and

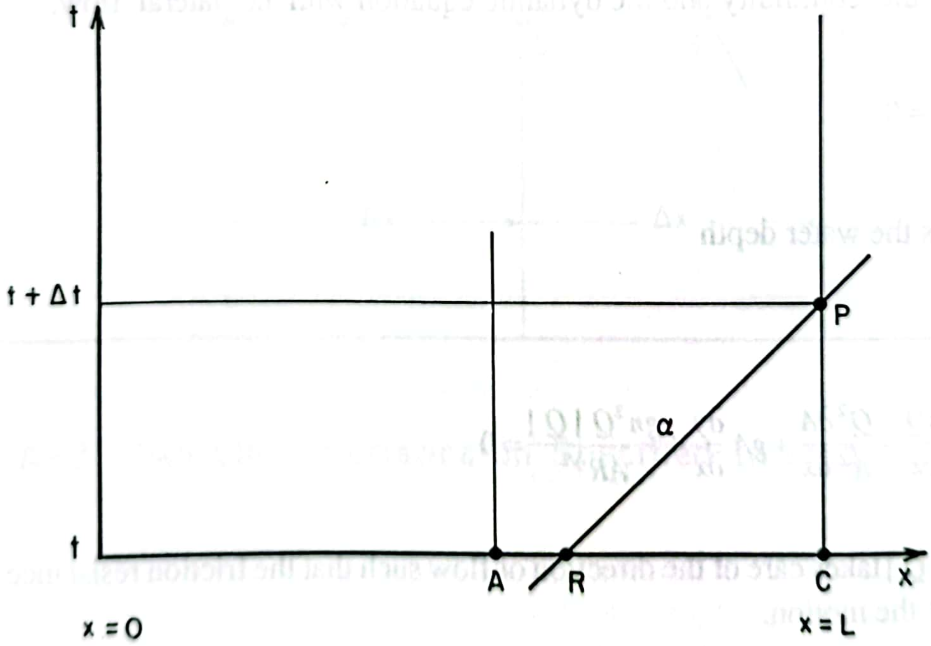
$$\frac{\partial Q}{\partial t}+2\frac{Q}{A}\frac{\partial Q}{\partial x}-\frac{Q^2}{A^2}\frac{\partial A}{\partial x}+gA\frac{\partial y}{\partial x}+\frac{gn^2Q\,|\,Q\,|}{AR^{4/3}}=0 \tag{11}$$

where $Q\,|\,Q\,|$ takes care of the direction of flow such that the friction resistance will always be against the motion.

It can be seen that (10) and (11) are non-linear equations, so also are the difference equations obtained from them by using the approximation of the following equations (Figure A-5).



a) Upstream Boundary



b) Downstream Boundary

Fig. A-4 The Net Points at the Boundaries

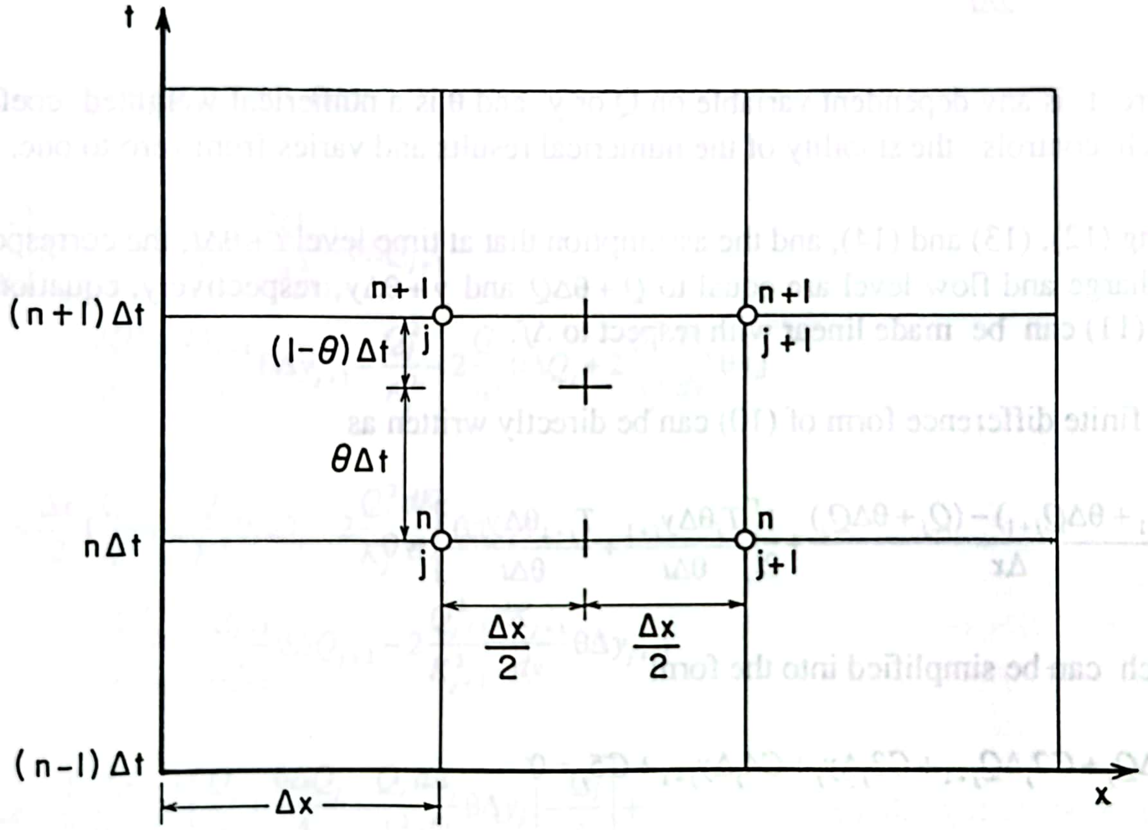


Fig. A-5 Rectangular Grid Showing Approximation to Derivatives

$$f(x,t)=\frac{\theta}{2}(f_{j+1}^{n+1}+f_j^{n+1})+\frac{(1-\theta)}{2}(f_{j+1}^{n}+f_j^{n}) \qquad (12)$$

$$\frac{\partial f}{\partial x}=\theta\frac{(f_{j+1}^{n+1}-f_j^{n+1})}{\Delta x}+(1-\theta)\frac{f_{j+1}^{n}-f_j^{n})}{\Delta x} \qquad (13)$$

$$\frac{\partial f}{\partial t}=\frac{(f_{j+1}^{n+1}-f_{j+1}^{n}+f_j^{n+1}-f_j^{n})}{2\Delta t} \qquad (14)$$

where f is any dependent variable on Q or y and θ is a numerical weighted coefficient which controls the stability of the numerical results and varies from zero to one.

Using (12), (13) and (14), and the assumption that at time level $t+\theta\Delta t$, the corresponding discharge and flow level are equal to $Q+\theta\Delta Q$ and $y+\theta\Delta y$, respectively, equations (10) and (11) can be made linear with respect to Δf.

The finite difference form of (10) can be directly written as

$$\frac{(Q_{j+1}+\theta\Delta Q_{j+1})-(Q_j+\theta\Delta Q_j)}{\Delta x}+\frac{1}{2}\left[\frac{T_j\theta\Delta y_{j+1}}{\theta\Delta t}+\frac{T_{j+1}\theta\Delta y_{j+1}}{\theta\Delta t}\right]=0 \qquad (15)$$

which can be simplified into the form

$$C1_j\Delta Q_j+C2_j\Delta Q_{j+1}+C3_j\Delta y_j+C4_j\Delta y_{j+1}+C5_j=0 \qquad (16)$$

where

$$C1_j=-\theta \qquad (16a)$$

$$C2_j=\theta \qquad (16b)$$

$$C3_j=\Delta x_j\frac{T_j}{2\Delta t} \qquad (16c)$$

$$C4_j=\Delta x_j\frac{T_{j+1}}{2\Delta t} \qquad (16d)$$

$$C5_j=Q_{j+1}-Q_j \qquad (16e)$$

On the other hand, (11) should be first modified to the form

$$\frac{\partial y}{\partial x}\Delta x+\frac{\partial}{\partial x}\left[\frac{V^2}{2g}\right]\Delta x+\frac{Q\,|Q\,|}{k^2}\Delta x+\frac{1}{g}\frac{\partial v}{\partial t}\Delta x=0 \tag{17}$$

where k is the conveyance of the channel and V is the mean velocity.

The finite difference equivalents of every term of (17) can now be expressed as

$$\frac{\partial y}{\partial x}\Delta x=y_{j+1}+\theta\Delta y_{j+1}-y_j-\theta\Delta y_j \tag{18a}$$

$$\begin{aligned}\frac{\partial}{\partial x}\left[\frac{V^2}{2g}\right]\Delta x=\frac{1}{2g}[\frac{Q^2_{j+1}}{A^2_{j+1}}+\frac{2Q_{j+1}}{A^2_{j+1}}\theta\Delta Q_{j+1}-\\ \frac{2Q^2_{j+1}}{A^3_{j+1}}\frac{dA_{j+1}}{dy}\theta\Delta y_{j+1}-\frac{Q^2_j}{A^2_j}-2\frac{Q_j}{A^2_j}\theta\Delta Q_j+2\frac{Q^2_j}{A^3_j}\frac{dA_j}{dy}\theta y_j]\end{aligned} \tag{18b}$$

$$\begin{aligned}\frac{Q^2}{K^2}\Delta x=\frac{\Delta x}{2}[\frac{Q^2_j}{K^2_j}+2\frac{Q_j}{K^2_j}\theta\Delta Q_j-2\frac{Q^2_j}{K^3_j}\frac{dK_j}{dy}\theta\Delta y_j+\\ 2\frac{Q^2_{j+1}}{K^2_{j+1}}+\frac{Q_{j+1}}{K^2_{j+1}}\theta\Delta Q_{j+1}-2\frac{Q^2_{j+1}}{K^3_{j+1}}\frac{dK_{j+1}}{dy}\theta\Delta y_{j+1}]\end{aligned} \tag{18c}$$

$$\begin{aligned}\frac{1}{g}\frac{\partial v}{\partial t}\Delta x=\frac{1}{2}\frac{\Delta x}{g\theta\Delta t}\{\left[\left(\frac{Q_j}{A_j}+\frac{\theta\Delta Q_j}{A_j}-\frac{Q_j}{A^2_j}\frac{dA_j}{dy}\theta\Delta y_j\right)-\frac{Q_j}{A_j}\right]+\\ \left[\left(\frac{Q_{j+1}}{A_{j+1}}+\frac{\theta\Delta Q_{j+1}}{A_{j+1}}-\frac{Q_{j+1}}{A_{j+1}}\frac{dA_{j+1}}{dy}\theta\Delta y_{j+1}\right)-\frac{Q_{j+1}}{A_{j+1}}\right]\}\end{aligned} \tag{18d}$$

Equation (17) with every term expressed in finite difference equivalents, can be simplified into the form

$$D1_j\Delta Q_j+D2_j\Delta Q_{j+1}+D3_j\Delta y_j+D4_j\Delta y_{j+1}+D5_j=0 \tag{19}$$

where

$$D1_j = -Q_j \frac{\theta}{gA_j^2} + Q_j \frac{\Delta x_j \theta}{K_j^2} + \frac{\Delta x_j}{2gA_j \Delta t} \tag{19a}$$

$$D2_j = Q_{j+1} \frac{\theta}{gA_{j+1}^2} + Q_{j+1} \frac{\Delta x_j \theta}{K_{j+1}^2} + \frac{\Delta x_j}{2gA_{j+1} \Delta t} \tag{19b}$$

$$D3_j = -\theta + \frac{Q_j^2 T_j \theta}{gA_j^3} - \frac{\Delta x_j Q_j^2}{K_j^3} \frac{dK_j}{dy} \theta - \frac{\Delta x_j T_j Q_j}{2gA_j^2 \Delta t} \tag{19c}$$

$$D4_j = \theta + \frac{Q_{j+1}^2 T_{j+1} \theta}{gA_{j+1}^3} - \frac{\Delta x_j Q_{j+1}^2}{K_{j+1}^3} \frac{dK_{j+1}}{dy} \theta - \frac{\Delta x_j T_{j+1} Q_{j+1}}{2gA_{j+1}^2 \Delta t} \tag{19d}$$

$$D5_j = y_{j+1} - y_j + \frac{Q_{j+1}^2}{2gA_{j+1}^2} - \frac{Q_j^2}{2gA_j^2} + \frac{\Delta x_j}{2} \left[\frac{Q_j^2}{K_j^2} + \frac{Q_{j+1}^2}{K_{j+1}^2} \right] \tag{19e}$$

Method of Solution

Equations (16) and (19) make up a system of simultaneous equations of four unknowns, namely; Δy_j, Δy_{j+1}, ΔQ_j and ΔQ_{j+1}. When applied to a system of N grid points 2(N-1) equations will be produced to solve 2N unknowns. The other two sets of equations required to make the system determinate are provided by upstream and downstream boundary conditions.

The procedure for solving this system of equations consists of: firstly, making use of one boundary condition in order to reduce the unknowns to three; secondly, making use of the concatenate property of the system of equations for the transmission of the calculation to another boundary; thirdly, making use of the second boundary to get rid of one more unknown to reduce the equations to the determinate system of equations; fourthly, solving the equation backward up to the upstream boundary, so the water elevations along the channels are computed.

Double Sweep Algorithm

Equations (16) and (19) can be solved by the usual method of simultaneous equations but a better solution particularly for large number of equations is by the use of the double sweep algorithm. In this method, the equations are linearized using the assumption

$$\Delta Q_j = E_j \Delta y_j + F_j \tag{20}$$

where E_j and F_j are constants at a particular point and time. Similarly, this relationship is assumed to exist for the next computational point j+1.

For the segment (j, j+1), (19) can be written as

$$\Delta y_j = L_j \Delta y_{j+1} + M_j \Delta Q_{j+1} + N_j \tag{21}$$

where

$$L_j = \frac{D4_j}{G_j} \tag{21a}$$

$$M_j = \frac{D2_j}{G_j} \tag{21b}$$

$$N_j = \frac{D1_j F_j + D5_j}{G_j} \tag{21c}$$

$$G_j = -[D1_j E_j + D3_j] \tag{21d}$$

By using (20) and (21), the terms ΔQ_j and Δy_j can be removed from the continuity equation (16) which can then be written as

$$\Delta Q_{j+1} = E_{j+1} \Delta y_{j+1} + F_{j+1} \tag{22}$$

where

$$E_{j+1} = \frac{[C1_j E_j L_j + C3_j L_j + C4_j]}{R_j} \tag{22a}$$

$$F_{j+1} = \frac{[C1_j (E_j N_j + F_j) + C3_j N_j + C5_j]}{R_j} \tag{22b}$$

$$R_j = -[C1_j E_j M_j + C2_j + C3_j M_j] \tag{22c}$$

The resulting equation (22) only shows that the assumed relationship is valid for any other computational point.

Upstream Boundary Condition. The upstream boundary condition can be specified either by a discharge hydrograph or a stage hydrograph.

When the upstream boundary condition is $Q_1 = Q_1(t)$

, the change in flow between t and t+1 time levels is given by $\Delta Q_1 = Q_1^{t+1} - Q_1^t$. To make (20) valid, i.e. for any value of computed Δy_j, ΔQ_j will always be equal to the boundary value, ΔQ_1, the constants E_1 and F_1 should take the values

$E_1 = 0$ and $Q_1^{t+1} - Q_1^t$

When the upstream boundary condition is $y_1 = y_1(t)$, the change in water level between t and t+1 time levels is $\Delta y_1 = y_1^{t+1} - y_1^t$. Similarly, to make (20) valid Again, i.e for any value of computed ΔQ_j, Δy_j will always be equal to the boundary value, Δy_1, the constants E_1 and F_1 should take the values

$E_1 = \beta$ and $F_1 = -\beta y_1^{t+1} - y_1^t$

where β is a very large number of the order of 10^4 to 10^6. This can be easily proven by rearranging (20) into the form

$$\Delta y_1 = \Delta \frac{Q_1}{E_1} - \frac{F_1}{E_1} \tag{23}$$

However, the use of β introduces another parameter which has to be calculated for every time level by iteration and this makes the calculation more tedious. A more straightforward method is to apply the concept of liniarization for ΔQ being true also for Δy (Jha, 1990). Thus, equations (16) and (19) can now be linearized with the assumption

$$\Delta y_j = E_j \Delta Q_j + F_j \tag{24}$$

By following the procedure for the derivation of (21) and (22), the following equations will be produced as their equivalents when the upstream boundary condition is specified as stage hydrograph.

$$\Delta Q_j = M_j \Delta Q_{j+1} + L_j \Delta y_{j+1} + N_j \tag{25}$$

where

$$L_j = \frac{D4_j}{G_j} \tag{25a}$$

$$M_j = \frac{D2_j}{G_j} \tag{25b}$$

$$N_j = \frac{D3_j F_j + D5_j}{G_j} \tag{25c}$$

$$G_j = -[D1_j + D3_j E_j] \tag{25d}$$

and

$$\Delta y_{j+1} = E_{j+1} \Delta Q_{j+1} + F_{j+1} \tag{26}$$

$$E_{j+1} = \frac{[C3_j E_j M_j + C1_j M_j + C2_j]}{R_j} \tag{26a}$$

$$F_{j+1} = \frac{[C3_j(E_jN_j + F_j) + C1_jN_j + C5_j]}{R_j} \tag{26b}$$

$$R_j = -[C3_jE_jL_j + C1_jL_j + C4_j] \tag{26c}$$

With this modified method, no additional parameter is required.

Forward Sweep. This process involves the calculation of coefficients E and F at each node for a specified time level.

Downstream Boundary Condition. The downstream boundary condition can be either a discharge hydrograph or a stage hydrograph. Depending on the type of upstream condition, ΔQ_N and Δy_N can be calculated using the appropriate equations as qualified by the four possible cases.

Case 1. Upstream and downstream boundary conditions are both discharge hydrographs.

$$\Delta Q_N = Q_N^{t+1} - Q_N^t \tag{27a}$$

$$\Delta y_N = \frac{\Delta Q_N}{E_N} - \frac{F_N}{E_N} \tag{27b}$$

Case 2. Upstream boundary condition is discharge hydrograph and downstream boundary condition is stage hydrograph.

$$\Delta y_N = y_N^{t+1} - y_N^t \tag{27c}$$

$$\Delta Q_N = E_N \Delta y_N + F_N \tag{27d}$$

Case 3. Upstream boundary condition is stage hydrograph and downstream boundary condition is discharge hydrograph.

$$\Delta Q_N = Q_N^{t+1} - Q_N^t \tag{27e}$$

$$\Delta y_N = E_N \Delta Q_N + F_N \tag{27f}$$

Case 4. Upstream and downstream boundary conditions are both stage hydrographs.

$$\Delta y_N = y_N^{t+1} - y_N^{t} \qquad (27g)$$

$$\Delta Q_N = \frac{\Delta y_N}{E_N} - \frac{F_N}{E_N} \qquad (27h)$$

Backward Sweep. The unknowns ΔQ_{N-1} and Δy_{N-1} can be computed by using (20) and (21) or by (24) or (25) depending on the type of upstream boundary condition. This can be repeated from computational point N-1 to N=1. When the upstream boundary is reached, the solution can be advanced to the next time level and the double sweep process repeated.

Initial Conditions

The initial conditions required for the solution of the unsteady flow equations are the discharges and stages at every grid point along the reach at the initial time of computation. This condition is rarely available in most of the open channels since the water elevations at different locations along the channels are usually not measured at the same time. However the water levels along the channel are able to be known, since the water levels are maintained to be steady before the new gate adjustments are started as mentioned before.

Flow Equations of Control Structures

Rectangular Weirs. The weir equations do not take into account flow contractions. Furthermore, the weir equations are for sharp-crested weirs. The accuracy is dependent on the coefficient of discharge which can be obtained by calibration of the particular weir (Figure A-6).

Free flow $\quad Q = C_d W (h_u - Sl)^a \quad$ for $(h_d < Sl < h_u)$ (28*a*)

Submerged flow $\quad Q_s = Q[1 - \left(\frac{h_d - Sl}{h_u - Sl)}\right)^a]^{0.385} \quad$ for $(Sl < h_d < h_u)$ (28*b*)

Zero flow $\quad Q = 0 \quad$ for $(h_u < Sl)$ (28*c*)

where
Q = free flow rate (m^3/s)
Q_s = submerged flow rate (m^3/s)
C_d = weir flow discharge coefficient
a = exponential value normally equal to 1.5

h_u = upstream flow elevation (m)
h_d = downstream flow elevation (m)
Sl = weir sill elevation (m)
W = weir crest width (m)

__Culverts.__ The equations provided are valid for culverts that serve as in-line or cross control structures which normally function under submerged conditions (Figures A-7 & A-8).

Circular
$$Q = NC_d\frac{\pi D_i^2}{4}\sqrt{2g(h_u-h_d)} \qquad \text{for } (h_u > h_d > Sl + D_i) \tag{29a}$$

Rectangular
$$Q = NC_dWH\sqrt{2g(h_u-h_d)} \qquad \text{for } (h_u > h_d > Sl + H) \tag{29b}$$

where
Q = flow rate (m^3/s)
C_d = discharge coefficient
N = number of culverts in parallel
h_u = upstream flow elevation (m)
h_d = downstream flow elevation (m)
Sl = culvert sill elevation (m)
g = 9.81 m/s^2
D_i = culvert inside diameter for circular culvert (m)
H = culvert height for rectangular culvert (m)
W = culvert width for rectangular culvert (m)

__*Constant Head Orifice*__ (Figure A-9).

$$Q = NC_dA\sqrt{2g\Delta H} \tag{30}$$

where
Q = flow rate (m^3/s)
C_d = discharge coefficient
N = number of gates in parallel
A = cross-section flow area (m^2)
g = 9.81 m/s^2
ΔH = difference between upstream and downstream water level of orifice gate (m)

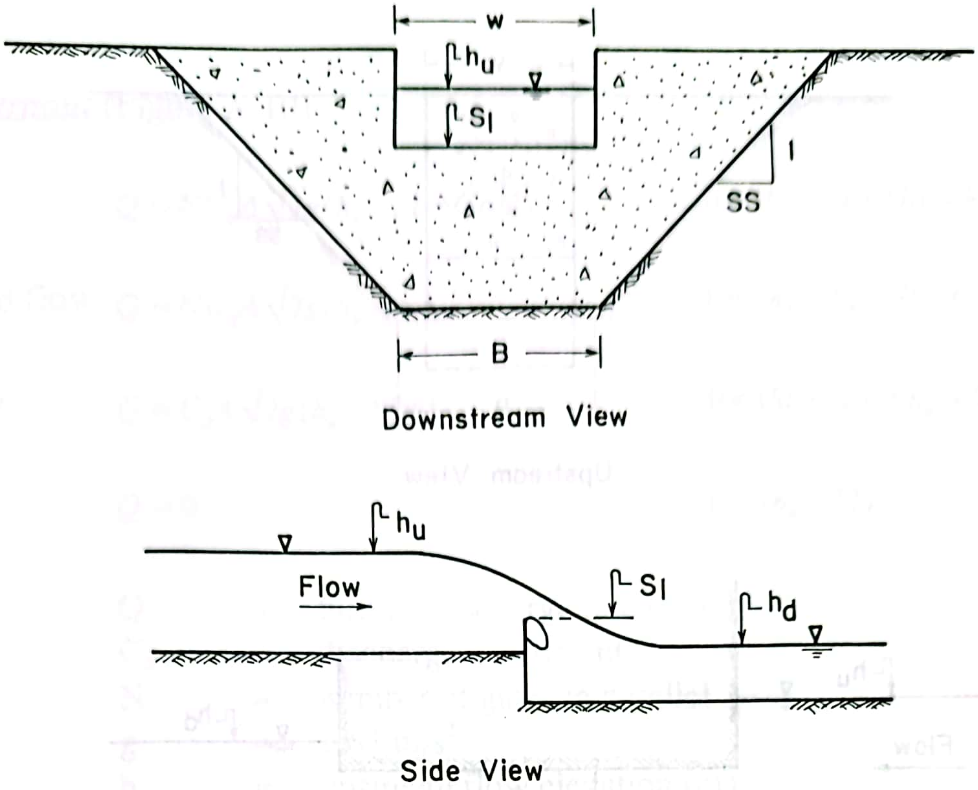


Fig. A-6 Rectangular Weir Control Structure

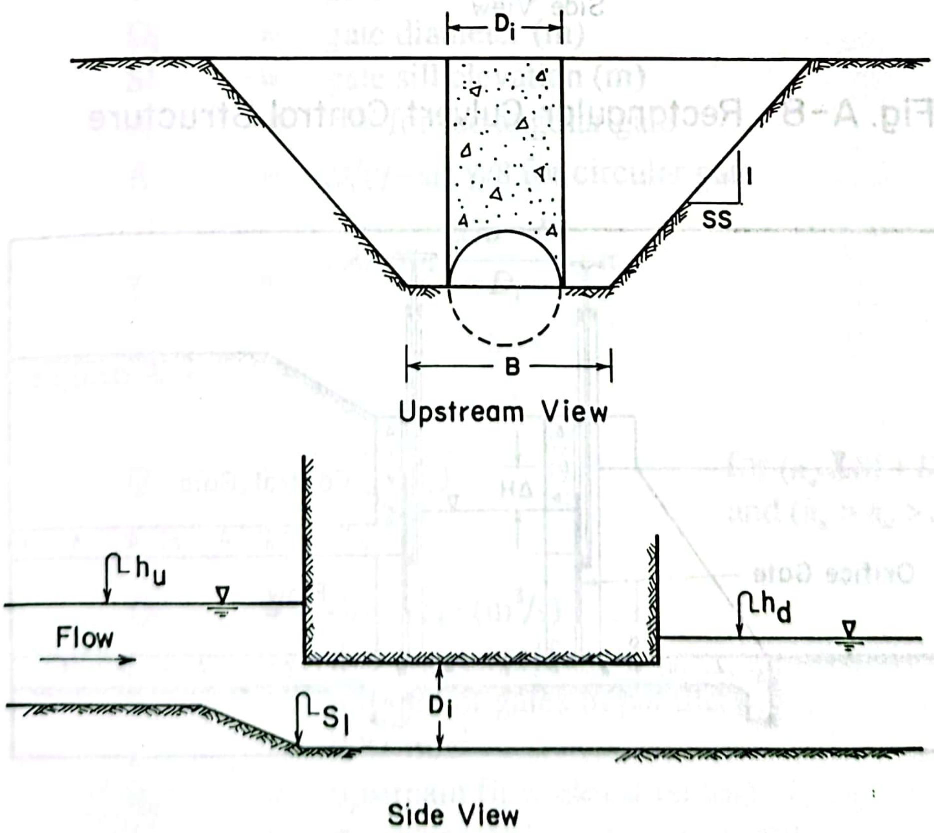


Fig. A-7 Circular Culvert Control Structure

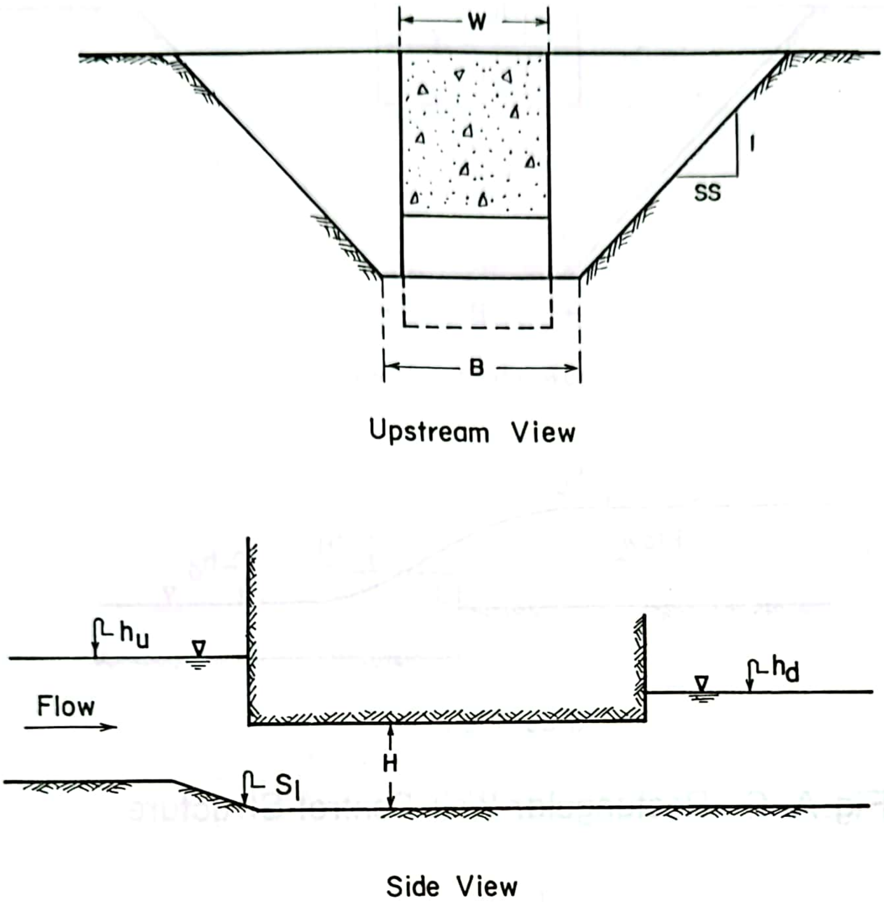


Fig. A-8 Rectangular Culvert Control Structure

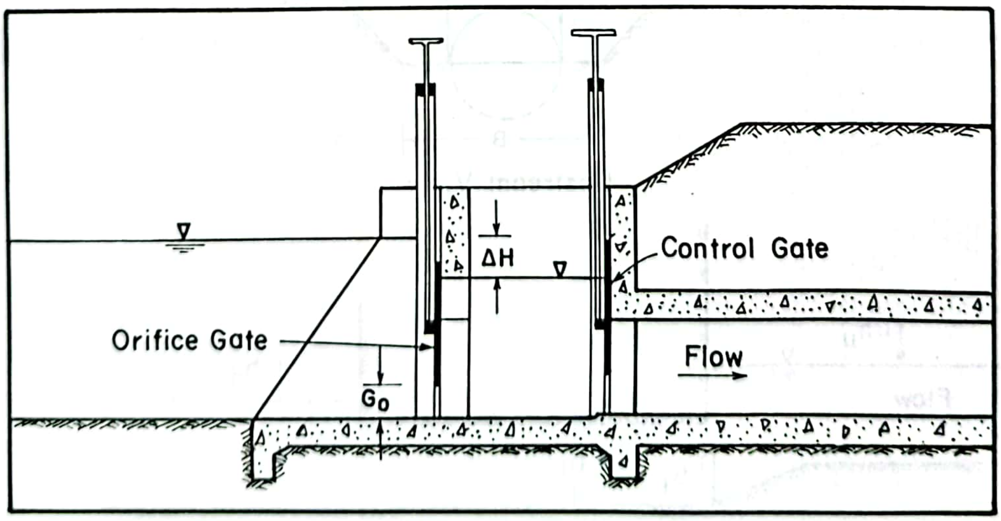


Fig. A-9 Constant Head Orifice Control Structure

Orifice Turnout (Figure A-10).

Free flow $Q = NC_dA\sqrt{2g(h_u - Sl - Go/2)}$ for $(h_u > Sl + Go > h_d)$ (31*a*)

Submerged flow $Q = NC_dA\sqrt{2g(h_u - h_d)}$ for $(h_u > h_d > Sl + Go)$ (31*b*)

Open flow $Q = C_dA\sqrt{2g(h_u - Sl)/2}$ for $(Sl + Go \geq h_u > Sl)$ (31*c*)

Zero flow $Q = 0$ for $(h_u \leq Sl)$ (31*d*)

where
Q = turnout discharge rate (m^3/s)
C_d = discharge coefficient
N = number of gates in parallel
g = 9.81 m/s^2
h_u = upstream flow elevation (m)
h_d = downstream flow elevation (m)
Go = gate opening (m)
W = gate width (m)
D_i = gate diameter (m)
Sl = gate sill elevation (m)
A = bW for rectangular gate
A = $D_i^2(\gamma - \sin\gamma)/8$ for circular gate
γ = $2ArcSin\dfrac{2Go - D_i}{D_i} + \pi$

Pipe-drop (Figure A-11)

$$Q = NC_dA\sqrt{2g(h_u - h_d)} \quad \text{for } (h_u > Sl + D_i) \text{ and } (h_u > h_d > Sl) \qquad (32)$$

where
Q = flow rate (m^3/s)
C_d = discharge coefficient
N = number of gates in parallel
g = 9.81 m/s^2
h_u = upstream flow elevation (m)
h_d = downstream flow elevation (m)

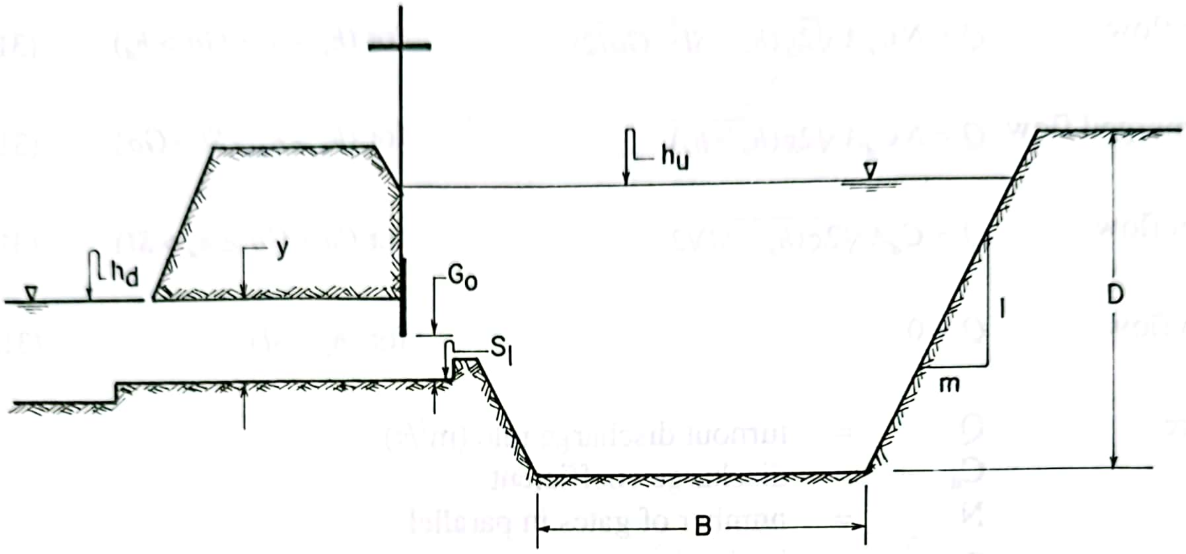


Fig. A-10 Circular and Rectangular Orifice Turnouts.
(Adopted from Merkley, 1987)

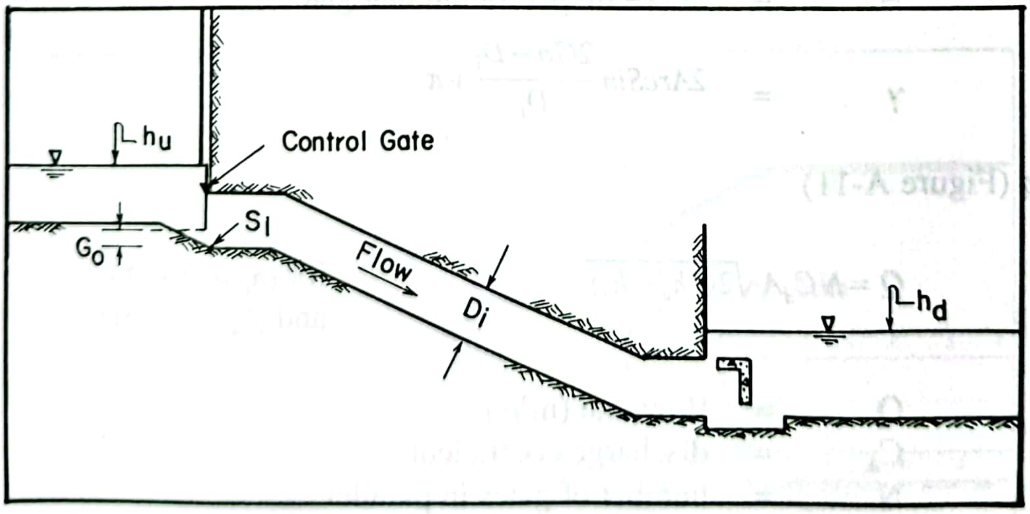


Fig. A-11 Pipe-drop Control Structure

Sl = sill elevation (m)
D_i = pipe inside diameter (m)
Go = gate opening (m)
A = $D_i^2(\gamma - \sin\gamma)/8$
γ =

$$2ArcSin\frac{2G_o - D_i}{D_i} + \pi$$

Submerged Flow. Many control structures such as radial gate, vertical sluice gate, siphon, culvert, and submerged orifice operate under submerged flow conditions. This is typical of systems where the bed slope of the canal is small (Figures A-12 & A-13).

Sluice gate w/ side weirs

$$Q = Q_1 + Q_2 \qquad (33)$$

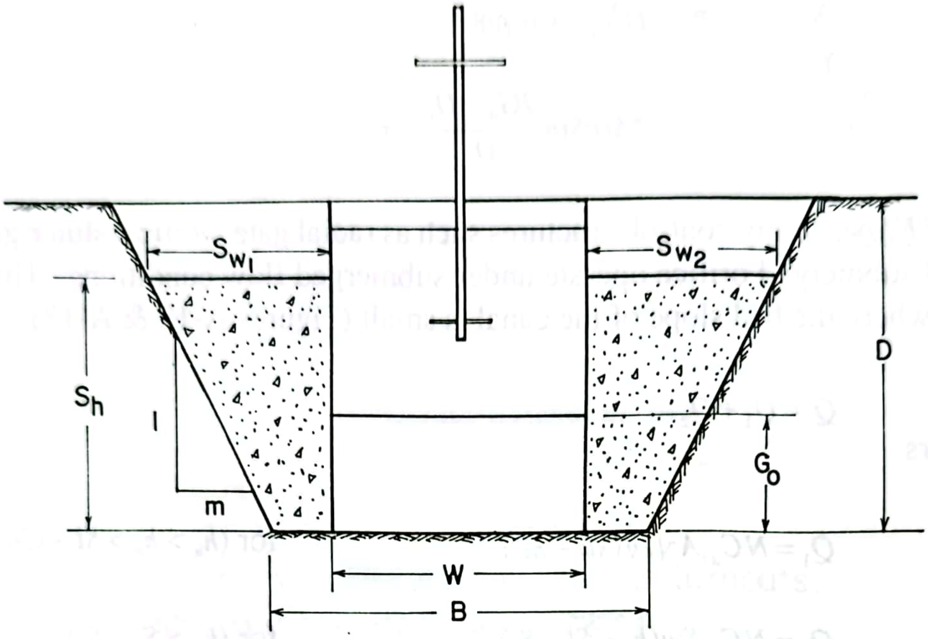
$$Q_1 = NC_{d1}A\sqrt{2g(hu - hd)} \qquad \text{for } (h_u > h_d > Sl + Go) \qquad (33a)$$

$$Q_2 = NC_{d2}Sw(h_u - Sl - S_h) \qquad \text{for } (h_u > S_h + Sl) \qquad (33b)$$

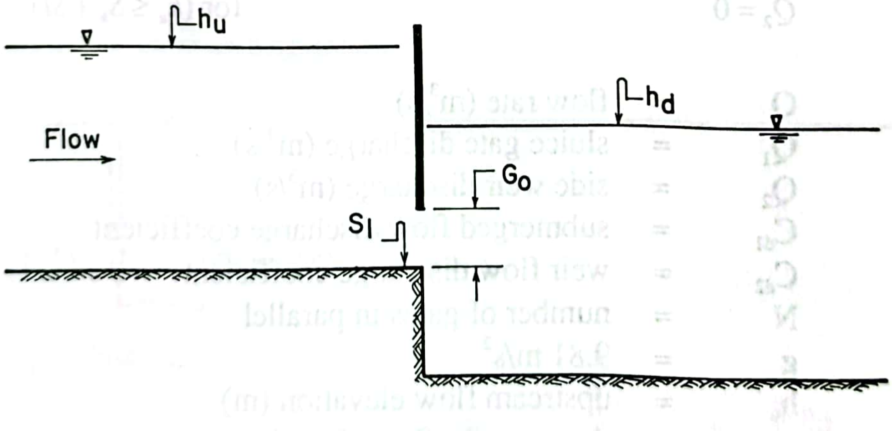
$$Q_2 = 0 \qquad \text{for } (h_u \leq S_h + Sl) \qquad (33c)$$

where

Q = flow rate (m^3/s)
Q_1 = sluice gate discharge (m^3/s)
Q_2 = side weir discharge (m^3/s)
C_{d1} = submerged flow discharge coefficient
C_{d2} = weir flow discharge coefficient
N = number of gates in parallel
g = 9.81 m/s^2
h_u = upstream flow elevation (m)
h_d = downstream flow elevation (m)
Go = sluice gate opening (m)
W = sluice gate width (m)
D_i = sluice gate diameter (m)
S_w = weir sill width (m)
S_h = weir sill height (m)
S_l = gate sill elevation (m)
A = bW for rectangular gate



Upstream View



Side View

Fig. A-12 Rectangular Sluice Gate Control Structure (Adopted from Merkley, 1987)

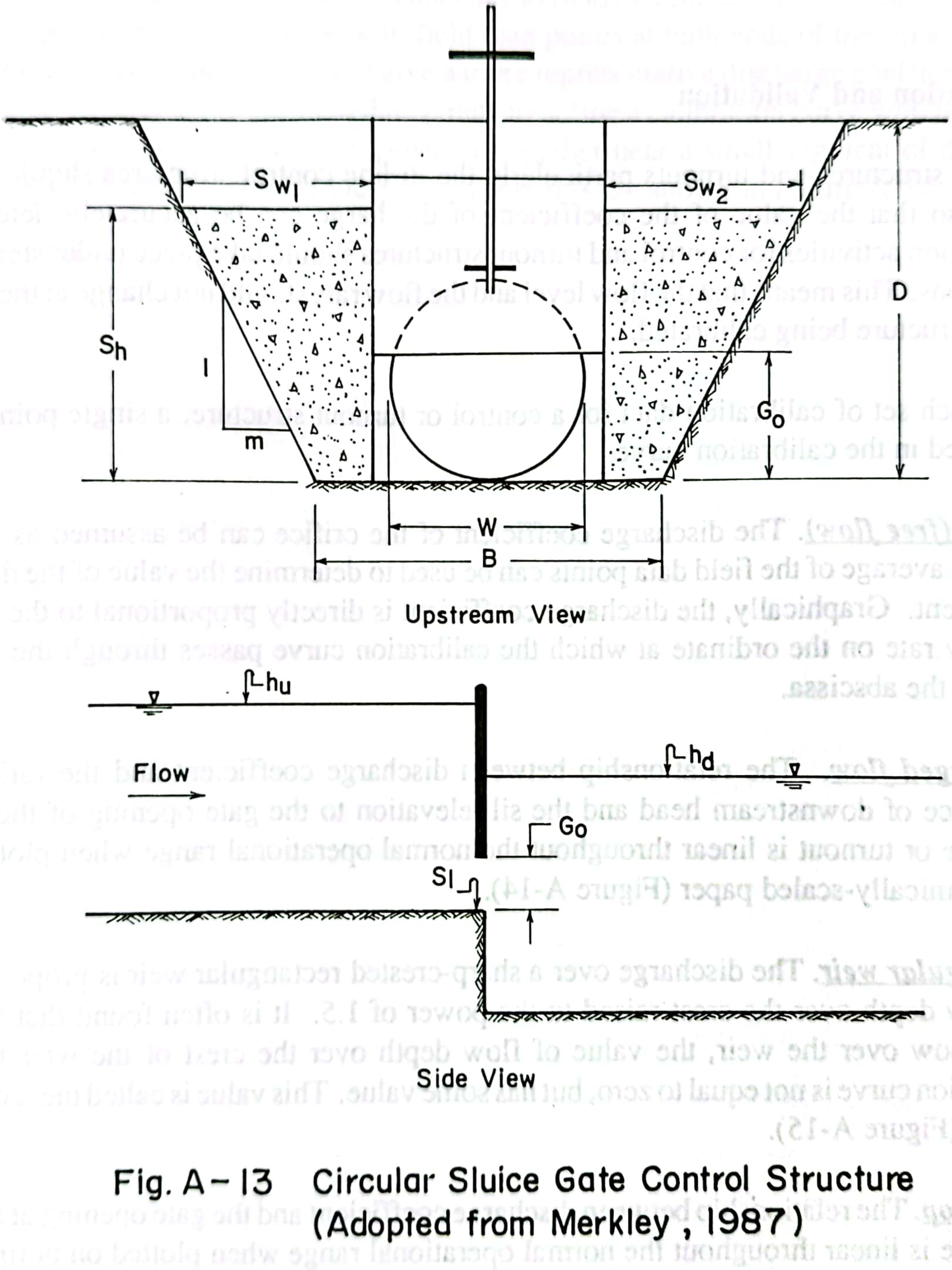


Fig. A-13 Circular Sluice Gate Control Structure (Adopted from Merkley, 1987)

$$A = D_i^2(\gamma - \sin\gamma)/8 \text{ for circular gate}$$

$$\gamma = 2ArcSin\frac{2Go - D_i}{D_i} + \pi$$

Calibration and Validation

Control structures and turnouts particularly the in-line control structures should be calibrated so that the value of the coefficient of discharge can be accurately determined. Calibration activities for control and turnout structures should take place under steady flow conditions. This means that the flow level and the flow rate should not change at the vicinity of the structure being calibrated.

With each set of calibration data for a control or turnout structure, a single point can be generated in the calibration curve.

Orifice (free flow). The discharge coefficient of the orifice can be assumed as constant since an average of the field data points can be used to determine the value of the discharge coefficient. Graphically, the discharge coefficient is directly proportional to the value of the flow rate on the ordinate at which the calibration curve passes through the value of unity in the abscissa.

Submerged flow. The relationship between discharge coefficient and the ratio of the difference of downstream head and the sill elevation to the gate opening of the control structure or turnout is linear throughout the normal operational range when plotted on a logarithmically-scaled paper (Figure A-14).

Rectangular weir. The discharge over a sharp-crested rectangular weir is proportional to the flow depth over the crest raised to the power of 1.5. It is often found that when no water flow over the weir, the value of flow depth over the crest of the weir from the calibration curve is not equal to zero, but has some value. This value is called the 'correction factor' (Figure A-15).

Pipe-drop. The relationship between discharge coefficient and the gate opening at a control structure is linear throughout the normal operational range when plotted on normal scale paper (Figure A-16)

It is preferable to make at least three separate calibration measurements which will generate three different points on a calibration curve. Separate sets of filed data for a single structure provide greater degree of confidence in the accuracy of the field work when the results are consistent, and to help identify invalid data due to field measurements or calculation errors. Furthermore, a calibration curve with field data points at both ends of the flow range for which the structure can operate will give a more representative discharge coefficient. This means that it is better to have field data points to define a calibration curve which is spread out than to have points which are clustered together near a small segment of the curve. Examples of clustered and non-clustered data points are shown in Figure A-17.

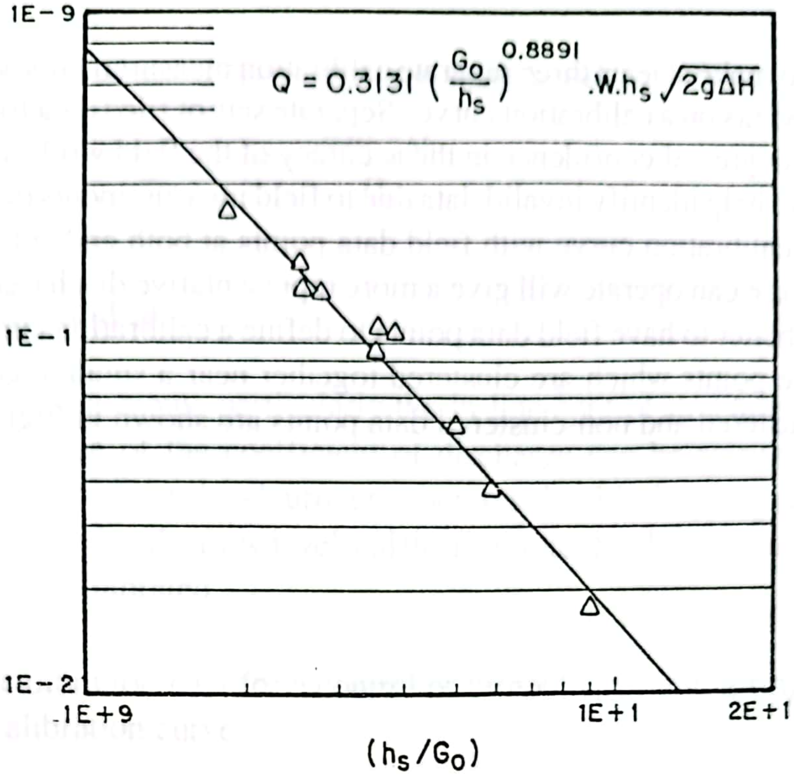


Fig. A-14 Example of Turnout Calibration

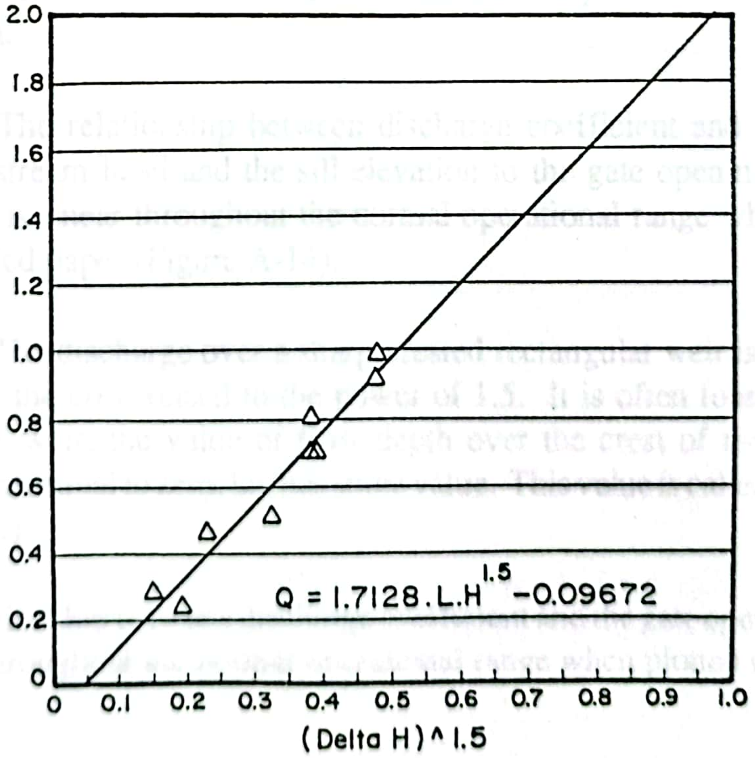


Fig. A-15 Example of Weir Calibration

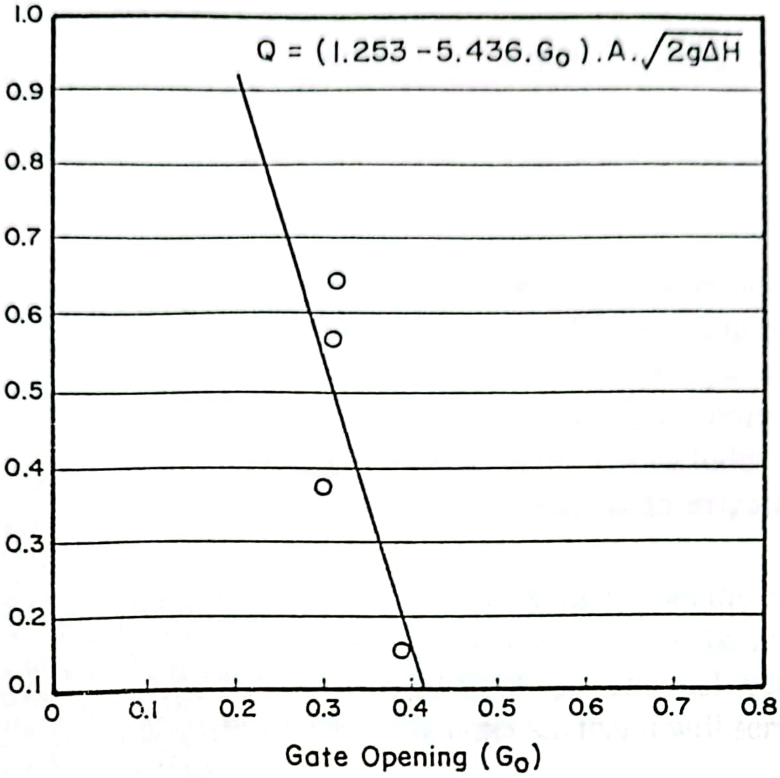
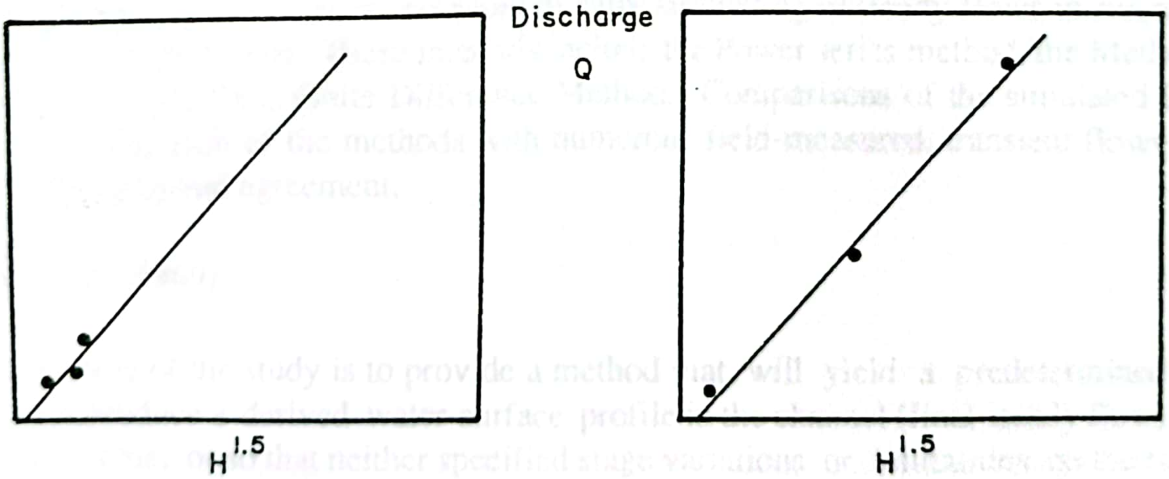


Fig. A-16 Example of Pipe-drop Calibration



Clusterered Data Points
(Not Good)

Non-Clustered Data Points
(Good)

Fig. A-17 Calibration Data Plots of Sharp-crested Rectangular Weir

APPENDIX B /

ABSTRACTS AND SUMMARIES OF RELATED LITERATURE

A fair share of research has been devoted to the hydraulic simulation of one-dimensional flow in open channels. The types of study have matured from being basic and theoretical (development of methods of solution; algorithm enhancement; parameter identification and modeling; stability and convergence concepts; simulating discontinuities; etc.) to more global yet practical approaches. The more modern approach includes the development of tools for the management of open channel networks such as an irrigation system.

In this section, the abstracts or summaries of related studies from the early seventies up to the present time are included. This compilation is believed to be representative of the events that are related to the mathematical modeling of open channel systems or of irrigation systems that took place during this period. It is expected that it will serve as a useful guide for any similar study in the future.

Baltzer, R.A. and Lai, C. (1968)

Three methods for accurately and economically simulating unsteady flows in rivers and estuaries were presented. These methods include the Power series method, the Method of Characteristics and the Finite Difference Method. Comparisons of the simulated flows obtained using each of the methods with numerous field-measured, transient flows generally indicate good agreement.

Wylie, F.B. (1969)

The objective of the study is to provide a method that will yield a predetermined gate motion to produce a derived water-surface profile in the channel (final steady flow) with minimum time, or so that neither specified stage variations or limitations on the rate of change of the levels are exceeded. It is intended that when the final control motion ceases, the flow is steady along the entire length of the channel. Specifically, the operational problems covered by the study are i) operation of a gate at the end, or at both ends of the channel to establish a steady condition in a minimum time, or to change the flow condition from one given condition (steady or unsteady) to another derived condition; ii) the operation of valves that control forbay channels at a pumping station or hydroelectric plant to initiate, alter, or stop the flow adequately; and the operation of control devices in a canal system to alter the flow from one discharge to another while remaining within specified depth limitations or without exceeding a certain rate of allowance fluctuation of water level. The method of characteristics which makes use of the conversion of the Saint-Venant equations into four particular ordinary differential equations is employed. The method is applied to a single channel pool.

Abbott, M.B. and Verwey A. (1970)

The numerical solutions of conservation laws have been investigated from the point of view of optimum accuracy (essentially for maximal order of truncation error), while maintaining stability. These optimum solutions have at the same time, comprehended the formation of discontinuities such as hydraulic jumps and other fronts. The subject herein is also an optimal scheme, but one optimized not only in the sense of accuracy, but also in the sense of program organization. Its starting point is the four-point symmetry of the normal or characteristic forms of the conservation laws in two independent variables. The method is to notate the symmetry and translate it into cyclic numerical operators of a type most conveniently programmed. The method is described primarily for the case of long waves, i.e., for free surface incompressible, nearly horizontal flows.

It was found out that the four-point method of characteristics is more accurate than the common Lax and Lax-Wendroff schemes, and the implicit schemes. The method can be extended to account for energy-dispersing terms, such as bed slopes, resistance, changes of section and lateral discharges. The bed slope and resistance terms have been accommodated without excessive complications into the algorithm, but changes of section proved less tractable. In general it would appear that, for the most extended flow system such as those arising in natural watercourses, the implicit numerical techniques are more appropriate to use.

Biernacki, T. and Piwecki, T. (1970)

The Saint-Venant equations are solved by an explicit finite-difference scheme for the case of a channel regulated by hydraulic structures. Computations are made for a channel of 20 km length with three spillways. A very general description of the computer program is given.

Ellis, J. (1970)

Characteristics equations describing one-dimensional unsteady open-channel flow in an irregular channel are developed. The integration of these characteristic equations is presented and methods of approximation for the evaluation of the integrals involved in these characteristic equations are considered. The most promising of these numerical integration techniques is incorporated in a computer program. The program is used in the modelling of the tidal motion in an inlet of the sea. The chosen inlet contained large changes of cross-sectional shape along its length. The varying geometrical properties of the inlet along its length are included in the analysis by preparing tables of sectional properties with water level for a series of cross-sections. This information is read into the computer at the start of computations. Comparisons are made between computed water levels and actual field records.

Strelkoff, T. (1970)

The numerical solutions of the Saint-Venant equations by the method of characteristics (MOC) and finite difference method (FDM) are discussed. The theoretical basis of the MOC is reviewed and used to show that in the general case the speed of long-wave disturbances is given by the slope of the characteristic curves. Finite difference schemes on rectangular nets in the x-t plane, based on the characteristic forms of the Saint-Venant equations as well as based on the direct forms are given and examined for their stability. Explicit numerical schemes, which are simple, but require small steps in time because of stability problems, are contrasted with implicit schemes that permit numerical solution over larger time steps but require the solution of larger sets of simultaneous algebraic equations at each step. The von Neumann technique for stability analysis and the double sweep method are presented in detail.

Brutsaert, W. (1971)

The unsteady spatially varied flow equations (Saint-Venant equations) are solved by an implicit finite difference with an explicit description at the boundaries. Imposition of improper boundary conditions which violate the physics of the problem resulted into either violation of continuity or numerical instability problems. The magnitude of the spatial increment used in this implicit solution scheme was critical on steep slopes (2.0%). The temporal distribution of flow and time to equilibrium was altered considerably depending on the specified value of Δx. Hydrographs on milder slopes (0.5% and 1.0%) were affected to a progressively lesser extent as the channel slope was decreased. The time to equilibrium flow starting from a dry bed was nonlinear with respect to change of channel length, channel slope, and rate of lateral inflow. For a given channel length and slope, time to equilibrium approached a constant for high rates of lateral inflow.

Becker, L., and Yeh, W.G. (1972)

A simple, easily implemented, and rapidly convergent computational procedure, the influence coefficient algorithm is introduced, for the solution of the identification of parameter problem in unsteady open channel flow from field observations on stage hydrograph and velocity distribution at one or more points along the channel. The parameters specifically chosen for identification are the two "friction slope" characteristics namely the channel roughness coefficient and the exponent of the hydraulic radius in the empirical friction slope relation, a number usually assumed to be 4/3. These parameters are not physically measurable and have to be determined from the solutions of the mathematical model using concurrent input and output measurements. This new procedure is related to both quasi-linearization and gradient methods. Moreover, an effective formulation of the algorithm is shown to depend on certain stability and convergence features, which relate to the finite difference solutions of the governing flow equations, but which are often ignored or glossed over.

Dorer, H. (1972)

A systematic survey of the different forms of the Saint-Venant equations is given. Difference methods such as method of characteristics with variable net and direct difference methods with fixed net are described and tested. The computations show that for short steep waves in irregular channels, the results obtained by the method of characteristics with variable net are better than those obtained by the methods with fixed net.

Quinn, F. H. and Wylie, E. B. (1972)

A hydraulic transient model of the Detroit River is developed by using the implicit method to solve the complete equations of continuity and motion. The river is modeled in the shape of a Y and has one main channel and two branching channels. The stability of the numerical solution, which uses the Newton-Raphson algorithm, is found to be dependent on the selection of a weighting coefficient. This coefficient determines the position at which the equations are evaluated on the x-t grid. The model inputs consist of water surface hydrographs at the head and mouth of the river. The outputs consist of flows at each end of the three channels and water surface elevations at the junction of the Y. Transient flows of the Detroit River induced by a severe wind tide on Lake Erie were simulated to illustrate the model. Good agreement was obtained between measured and computed water surface elevations at the junction of the Y.

Abbott, M. B. (1974)

One major problem encountered in computational hydraulics is that of representing a discontinuity in an otherwise continuous flow, such as a hydraulic jump, or a front in nearly-horizontal flows. In these cases, the partial differential equations generate multi-valued solutions that are physically unrealistic, while numerical techniques may develop parasitic waves such that, if the technique is accurate for smooth continuous flows, it will often become unstable. One way out of this difficulty is to fit the jump or front, by introducing the specific local laws of the discontinuity and mating the continuous parts on either side through these laws, simultaneously locating the discontinuity. Another approach is to use dissipative difference schemes. In both cases, the theory and subsequent programming becomes enormous and complicated, especially for two-dimensional and stratified flows.

The objective of the study is to provide a numerical analysis of flows with continuous and discontinuous parts, leading to numerical procedures for efficiently simulating these flows. The paper discusses weak solutions of systems of conservation laws, dissipative difference schemes, difference forms of conservation laws, information transfer and loss through difference schemes, dissipative interfaces to linearly non-dissipative schemes, and examples of applications of dissipative interfaces.

Fread, D. L. (1974)

Linearized model equations of the quasi-linear differential equations of unsteady gradually varied flow are utilized to investigate the effect of the discretization of the continuous partial derivatives with implicit four-point finite difference quotients. Through the use of a weighting factor (θ) which positions the spatial difference quotient between adjacent time levels in the x-t solution region, the investigation is generalized to include the various four-point implicit difference schemes that have been reported in literature.

Numerical stability properties of the four-point difference schemes are analyzed using the von Neumann method. The difference equations are found to be unconditionally linearly stable when $0.5 < \theta < 1.0$ and conditionally stable when $\theta < 1.0$. The convergence properties are qualitatively investigated by determining the truncation error. The backward implicit scheme ($\theta = 1.0$) has a first order truncation error, i.e., $E = O(\Delta t) + O(\Delta x^2)$ whereas, the box implicit scheme ($\theta = 0.5$) has a second order truncation error, i.e., $E = O(\Delta t^2) + O(\Delta x^2)$. The convergence properties are quantitatively investigated by determining analytical expressions for wave damping and wave celerity convergence ratios, e.g., numerical damping/physical wave damping. These expressions are nondimensionalized in terms of convenient dimensionless parameters, and graphs are presented which quantify the convergence ratios for a wide range of the dimensionless parameters. The box scheme is shown to possess superior convergence properties compared to the backward implicit scheme, particularly with respect to wave damping. On the basis of convergence properties, the box scheme is shown to be the preferred implicit four-point difference scheme for discretizing the differential equations of unsteady flow.

Jolly, J. P. and Yevjevich, V. (1974)

Two types of errors which occur in the numerical simulation of gradually varied, unsteady open-channel flows are investigated in this paper. These are i) errors resulting from the choice of the momentum equation; and ii) errors resulting from the degree of numerical sophistication of integration techniques used in solving the governing equations by the specified intervals scheme of the method of characteristics. It was found that three different momentum equations gave identical results when the velocity distribution was uniform. When this distribution was not uniform, each equation gave somewhat different flow depths for similar initial and boundary conditions of gradually varied, unsteady flow. An increase in the complexity of the integration technique increases the accuracy of simulation. To determine the absolute values of accuracy of a technique, the development of accurate flow measuring devices and corresponding experiments are needed for comparison between the results of numerical and physical simulations.

Sakkas, J.G. and Strelkoff, T. (1974)

The Saint-Venant equations governing gradually varied, unsteady flow in an open channel with seepage are put into characteristic form and solved numerically in finite steps along the irregular network formed by the characteristic lines using a simple predictor-corrector scheme. The infiltration into the soil is assumed to depend solely upon contact time between water and soil. In regions of substantial curvature of the characteristic lines, step size is reduced to preserve accuracy. Near the very front of the advancing stream, where the forward and backward characteristics curve extremely and sharply merge with their envelope, the wave front trajectory, the numerical approximations to the characteristic equations break down and are replaced by the assumption that water velocity is independent of the distance coordinate and equals front-propagation speed.

Abbot, M.B. (1976)

The design system is an ensemble of mutually compatible routines organized into a single program documentation entity capable of supplying solutions to any one of a set of problems when presented only with the specific problem description. Through the use of these systems it is possible to solve problem of unsteady open-channel flow much more quickly, accurately and cheaply than is possible with one-off mathematical models.

The design system S11 Siva, S21 Jupiter and S12/13 Neptune of the Computational Hydraulics Center are described together with a sketch of their development and field of application. All these systems treat the extended equations of open-channel flow, including convective terms, and in all cases high accuracy implicit difference schemes are used. The Siva is used for one-dimensional water bodies of homogeneous density in any vertical such as rivers, fiords and well-mixed estuaries. The Jupiter is used for two-dimensional regions without stratification; such as more complex estuaries, coastal regions and even open seas, while applications have also been made to floodings of initially dry regions, following the failure of dams or dikes. The Neptune is applied to stratified fluid.

The difference schemes used in these systems are all implicit, fully-centered in their dominant terms. In the S11, the convective momentum terms are introduced explicitly and, by virtue of the alternate definition of dependent variables, they are subject to some dissipation. This approach simplifies the scheme and its analysis for the rather complicated forms that appear in the one-dimensional, homogeneous fluid case, without causing unacceptable errors. Moreover, the S11 scheme uses no iteration but relies upon their dissipative interfaces to maintain non-linear and long-term stability under adverse conditions.

Amein, M. (1976)

The paper discusses the important features of a four-point implicit method which make it suitable for field application. These features include i) its efficiency; ii) its ability to handle large changes in stage and discharge; iii) its ability to handle large variations in channel geometry; and iv) the time step can be selected on the basis of physical nature of the problem rather than the requirements of numerical stability. The method is shown to be suitable for field applications such as to rivers, reservoirs and estuaries since i) the distance step (Δx) can be varied and the stations can be selected at desired locations; ii) the channel bottom slope can be changed at every station; iii) the time step (Δt) can be selected as desired, i.e. the time step can be chosen on the basis of physical aspects such as to register significant change with time, rather on the basis of numerical stability; iv) large changes in channel properties from section to section can be handled; and v) branching channels and channel junctions can be conveniently incorporated. The method is efficient because of the following features: i) the method is unconditionally stable, therefore allowing larger time steps to be used in the computation which compensates for the required additional computations and iterations; and ii) the matrix of coefficients of the system of equation is sparse and is diagonally banded, thus, special routines can be used to solve the linear system very efficiently at each iteration step.

Bolshakov, V.A. and Kleshchevnikova, T.P. (1976)

Unsteady flow in irrigation canals with and without automatic water supply control based on the transfluent volume scheme is investigated. In conformity with this scheme, the mathematical model of unsteady flow was worked out for single as well as for interconnected canal sections between controlling structure. According to the scheme, large side distributors of water were considered point outflows whereas small distributors and pumping stations are considered as outflowing uniformly distributed along the canal. The scheme is generally based on the balance of water volumes being taken away by consumers and of those flowing into the system. An im-explicit increment scheme is used in obtaining the solution of the system separately for inner points of the sections, on boundaries, and for points of outflow or conjugation. Its advantages are i) higher accuracy of solution than that obtained by explicit method; and ii) simplified computation as compared with the use of implicit method since it is not necessary to deduce coefficient matrices for points of conjugations and for boundary points of each canal section.

Chaudhry, M.H. (1976)

A mathematical model is developed to analyze transient state flows in open channels caused by flow and/or stage changes. These flow changes may be due to load acceptance or rejection by turbines; starting or stopping of pumps; or opening or closing of control gates. Explicit finite difference method is used to numerically integrate the continuity and momentum equations describing the transient state flows. The scheme which is called also as diffusive scheme is stable unlike all other explicit schemes because it adds viscosity terms to the original equations. A computer program was developed to compute transient state conditions caused by discharge or stage changes at either end of the channel. The following boundary conditions were included in the program: i) flow or stage changes of the upstream or downstream end of a channel; ii) constant head reservoir at the upstream on downstream end; and iii) junction of two channels having different cross sections, friction factors and/or bottom slopes. To verify the mathematical model, prototype tests were conducted on the Seton canal, British Columbia, Canada. The results show the computed and the measured water levels agree closely. However, the secondary fluctuations of water surface (Favre waves) could not be computed by the program because of the inherent limitations of the governing equations.

Chu, H.L. and Mostafa, M.G. (1976)

A mathematical model and its numerical procedure using an implicit finite difference method are presented for the solution of equations governing the unsteady flow over side-weirs in an open channel with flows either in the subcritical state or in the supercritical state. The model is capable of accepting relatively large time intervals (Δt) with a combination of very small distance intervals (Δx) along the side weir reach and large distance intervals (Δx) elsewhere without affecting the stability and the accuracy of the solutions. This particular characteristic renders the model most suitable in cases where side-weir flows are encountered along short distances of an open channel. For subcritical flow cases, satisfactory results could be produced using any value between 0.5 and 1.0 for θ. Based on numerical experiments, however, it was found that for achieving test results, θ should be assigned the value of 1.0 in both subcritical and supercritical flow cases.

For a subcritical flow case, the upstream boundary condition can be either a discharge hydrograph or a stage hydrograph. The downstream boundary condition is usually given by a rating curve which provides the relationship between Q versus depth y. For a supercritical case, flow is controlled at the upstream end and normally two upstream boundary conditions are required. The first upstream boundary condition may be a discharge or stage hydrograph while the second upstream boundary condition may be a rating curve such as a Q versus y relationship expressed by the uniform or critical flow criteria. The results confirmed the versatility, stability and efficiency of the model. As demonstrated, the mathematical model developed is capable of handling subcritical as well as supercritical unsteady flows in open-channel with lateral outflow over side weirs under different combinations of boundary conditions.

Cunge J.A. (1976)

It is a common error to believe that a particular scheme of finite difference is more appropriate than others to represent the variable channel geometry or other physical features. Such is not the case. Either the scheme is good enough to integrate numerically the flow equations or it is not. Some of the schemes represent real life 'better' because they do not fail during the computation when discontinuities occur. That does not mean, however, that they reproduce nature more accurately - it only means that they are able to smooth real-life discontinuities and are more convenient to use. Such is a case when θ is chosen equal to one since it is a dissipative scheme and it gives the maximum possible smoothing. However, it is advised that the weighting factor θ, should not always be chosen as equal to one.

Dewey, H.C. Jr. and Madsen, W.R. (1976)

The gate operation for flow control of the California aqueduct of the California state water project has been continuously modified to keep up with the changing operational demands. Initially, serial gate operation was employed. With the increasing operating demands, simultaneous gate operation was implemented which was eventually modified into what is presently used method called timed gate operation. The serial gate operation involves the adjustment of gates to the proper setting to accommodate the flow change as the surge due to this change arrives at each downstream structure. Under this method an increase in water delivery to the customer is a function of the time required for the new sustained flow to reach the customers water delivery points from the water source. The greater the distance from the water source to the aqueduct diversion points, the longer the time required to meet the delivery change. As water demands increased, this method no longer met operational requirements. Flow changes took too much time and increased water demands caused greater hydraulic transients than allowable.

The simultaneous gate operation involves the operation of gates at all structures and on-line pumping plants simultaneously, or nearly so, throughout the length of the aqueduct system. Distance from the water source to the aqueduct diversion point has no appreciable effect on the time required to meet the delivery change. However, with major flow changes, the method caused greater hydraulic transients than allowable. Timed gate operation utilizes the principle of controlled volume concept of simultaneous gate movement with one variation which is time-scheduling of gate movements. Each gate at a check structure is operated on a time schedule instead of moving all gates at all check structures uninterrupted to the predetermined gate position as in the case of simultaneous gate operation. This method meets all operational requirements of the aqueduct.

Ellis, J. (1976)

The discharge and water level in an unsteady flow change from moment to moment at any point of the open-channel and instead of the variables of discharge and water level being explicit, an equation is obtained relating these variables. This study examined the use of this equation, derived from the integration of the equations of unsteady flow, in the solution of some typical transition problems of open-channel flow and highlights the influence of control conditions on the types of solutions adopted. It is evident that the transit time of unsteady flow phenomena through any discontinuity will be small in comparison with the commonly chosen time step of most numerical models. Thus, it is assumed that transient effects impinging upon the upstream and downstream limits of the discontinuity influence simultaneously the entire body of fluid contained in the transition. It thus becomes possible to apply the familiar steady flow laws of conservation of mass, energy and momentum to the transition at any instant of time. The discontinuities covered in the paper are caused by broad and sharp-crested weirs, free overfall, abrupt change of cross section and a constriction of flow area.

When handling irregular or natural channels, the gradient has been taken as the difference in mean invert levels of two adjacent sections divided by their distance apart. Where fixed discontinuities of velocity or water level or both exist in a system, it is logical to compute solutions immediately upstream and downstream of the transition and this is most readily achieved by means of a fixed-mesh technique. Where the state of flow is subcritical throughout the transition then transient effects may propagate through the discontinuity in either direction and the solution will be dependent upon influences for both upstream and downstream (coupled). Where a control section is formed, a zone of supercritical flow will exist which will inhibit the propagation of waves against the flow. The conditions of flow at the control and upstream are dependent only upon transient conditions above the discontinuity. The discharge through the control may then be used in the computation of flow conditions below the zone of supercritical flow together with downstream transient effects (uncoupled). It is possible that a discontinuity may behave in an uncoupled manner for part of the time and behave in coupled manner for the remaining time.

Keuning, D.H. (1976)

The finite element technique and Galerkin principle was applied to the complete, nonstationary, and nonlinear equations for one-dimensional open channel flow. In order to avoid extensive calculations, the principle of sectional linearization is introduced, i.e., nonlinear terms are linearized in every element. It was shown that a rather complicated problem (flow through an open channel with a sluice at the end) may be solved easily by means of finite element method (FEM).

Lai, C. (1976)

The method of characteristics is used to simulate one-dimensional, unsteady open-channel flow. The flow can be represented by a set of quasi-linear partial differential equations, consisting of 2 dependent variables and 2 independent variables. A numerical scheme based upon specified-time-interval (STI) is employed using two different techniques: i) solving the characteristic equations to second-order accuracy by the trapezoidal rule using iteration; and ii) applying an extrapolation procedure to two sets of linear computations using two different sizes of interval. Numerical experiments showed virtually the same results for the two techniques, however, the second technique is preferred for it generally consumes less computer time, and offers the option of selecting first or second order results.

In order to distinguish between the specified-time-interval scheme of the method of characteristics and the diffusive scheme of the explicit method, the former does follow the two characteristics, c+ and c- and solves the finite difference forms of the characteristic equations, whereas the latter solves the direct difference forms of the original partial differential equations without following the characteristics. The latter assumes that the dependent variables vary linearly within the interval of 2Δx, whereas the former assumes the linear variation for the interval of only Δx when the first order approximation is used and applies non-linear interpolation or quadratic interpolation when the direct second order approximation is used. Yevjivich and Barnes (1970) found that the STI scheme was more accurate than the diffusing scheme. Furthermore, in open-channel transients, the peaks and troughs are usually not to sharp as those of closed-conduit transients which means the change of flow in open channels is relatively gradual and thus, permits accurate computation by interpolation particularly if quadratic interpolation is used.

Priessman, A. (1976)

The present methods used to solve the Saint-Venant equations are described to be satisfactory. However, in the application of mathematical models to specific cases, the following problems arise: i) the topographical and hydraulic measurements program required for a mathematical model is very expensive, thus, the measurements should be limited to real essential points for flow simulation; ii) the velocity distribution in a given cross-section cannot be determined by integrating the Saint-Venant equations; and iii) the conventional head loss relationship does not apply in the case of channels or rivers carrying heavy suspended sediment load.

The essential assumptions for which the Saint-Venant equations are valid are: i) hydrostatic pressure distribution; and ii) uniform velocity distribution in any cross section. The first condition is practically met in river/channel flow except in the case of undular jumps, in which the hydrostatic assumption results in major distortion. Vertical accelerations modifying the pressure distribution must be incorporated in the flow equation for this type of flow. Cross-sectional velocity distribution, however, is seldom uniform in river flow. Nevertheless, accurate knowledge of it is most important only when investigating salinity intrusion, pollutant convection, or suspended sediment flow.

Vasiliev, O.F., Voyevodin, A.F. and Atavin, A.A. (1976)

An implicit finite difference method is developed for the calculation of regimes of unsteady flow in branching network (tree-type) systems as well as in general network systems (loop). The application of absolutely stable implicit difference schemes for approximation of differential equations allows choosing the time and space steps of the difference grid independently with a view toward achieving maximum accuracy. It is especially essential for calculation of floods in river systems and of long transient processes in canal systems. Two algorithms, one for each type of network, are developed which take into account the three-diagonal structure of difference equation matrix and are variants of matric method of factorization. For the case of a branching system of open channels, the system of nodes and segments represents a tree-type graph, the basic property of which is that the number of nodes is one unit larger than the number of segments. The algorithm for the solution of the system of algebraic equations is a generalization of the known method of "matrix sweep" which takes into account the above mentioned properties of the system.

Verwey, A. (1976)

A certain difference scheme can be solved with many different algorithms and, inversely, many different schemes can be solved with the same algorithm. Efficiency of a difference scheme can be evaluated using different criteria, such as the computation of coefficients, solution of the system of equations, flexibility in introducing different types of boundary conditions, overall development costs, simulation costs, selected programming language or even less measurable criteria as the capability of the personnel involved, in program development and use. The efficiency of four-point implicit method can be improved by writing out the advective momentum terms in quasi-linear form, solving the set of equations obtained by an efficient double sweep algorithm, and iterating one or two times to improve the linearized coefficients. However, Amien (1976) pointed out that the use of quasi-linear coefficients and the double sweep method verges on the trial and error solutions, which necessitates the use of small time steps, and which is ineffective when dealing with flows of long duration.

Bodly, W.E. and Wylie, E.B. (1978)

The concept of gate stroking permits the motion of control gates in an open channel to be determined such that a desired change in channel demand is effected in a prescribed manner with no residual disturbance existing in the channel when the gate motion is completed. This concept is applied to a series of pools separated by underflow gates with reservoir conditions on the extreme end of the channel. Three typical demand changes were considered: flow initiation, flow arrest, and flow change.

The gate motions determined by the stroking solution generally displayed continuous rate changes and would therefore be difficult to achieve in practice. To overcome this difficulty, the stroking-designed gate motions were represented by single speed or two-speed linear approximations. The most pronounced effect of these approximations was that residual disturbances remained in the channel after the gate motions were completed. These disturbances were not large in amplitude, but displayed very little damping and would persist for a very long time. The closer the stroking-designed motion is matched by the physical gate operation, the smaller will be the residual transient that persist in the system. The solutions presented in this study were obtained by the method of characteristics. It has the particular advantage that the characteristic lines generated in the solution are essentially the paths, in the x-t plane, of surface disturbances. This knowledge can be most valuable in interpreting solutions. The assumptions used are i) energy losses are adequately represented by Mannings equation for steady uniform flow; and ii) a finite depth of water remains at each section in the channel at all time during the transient and that changes are never so rapid so as to create a hydraulic bore.

IAHR (1978)

As part of the International Association for Hydraulic Research (IAHR) information exchange of computer programs, the following softwares were identified for one-dimensional unsteady open-channel inflow simulation.

CARIMA - Unsteady Flow in Looped River/Flow Plan Networks. This package is used for the simulation of unsteady flow in looped and branched network both having one-dimensional channel flow and two-dimensional flood plain flow as well as special features such as dams, power plants and automatic flow regulation. The one-dimensional channel flow is modeled using the complete Saint-Venant equations, solved implicitly using the Priessman method. Two-dimensional flood plains are modeled using storage cells which communicate with each other and with the main channel by weir-type and channel-type flows. The topological linkage is identified, checked and put into calculations order automatically by the program, which also performs extensive data checks and plots data and results. The limitation is that in the calculation of supercritical flow, suppression of convective acceleration terms is required. Computation time is 0.003 sec/pt/time step. The language used is FORTRAN IV which is run in an IBM 370/155. The package is documented in detail in English and French and in metric units and was developed and updated in 1978 by SOGREAH, France.

Four Point Method of Characteristics. This program computes water levels and flow in a channel under unsteady conditions. It can be used for studying the behavior of difference schemes. The four-point method of characteristics is used. The following assumptions are used; no friction, no bed slope, constant width, and no hydraulic jumps. The language used is ALGOL and run in an IBM 370/158. The program is documented in English and in metric units and was developed and updated in 1971 by A. Verwey of IIHEE, The Netherlands.

Cherie Computation of Unsteady Flow in Open Channel Networks. This program computes unsteady flow in open-channel networks, e.g. estuaries and canal networks. It offers various possibilities with boundary conditions, type of channel profile and friction law. The method used is the explicit leap-frog scheme which is generalized for networks. Its limitation is that it is only applicable for small Froude number due to absence of convective acceleration term. The language used is ALGOL and run in IBM 370. It is documented in English and in metric units. It was updated in 1976.

Unsteady Flow in Open Channel. This program can be used for the computation of unsteady flow in a network of open channels with arbitrary geometrical conditions and network layout and various boundary conditions. The method employed is implicit finite difference method with unequal mesh width and with local linearization. The language used is ALGOL and is documented in Dutch and in metric units. It was developed and updated in 1975 by Delft Hydraulics Laboratory, The Netherlands.

Netflow - Unsteady Flow in Open Channels. This package is used for the computation of unsteady flow in a network of open channel with arbitrary geometrical conditions and network layout and various boundary conditions. The method employed is finite difference method with local linearization, variable mesh width, sophisticated treatment of boundary condition and outflow possibilities along the channel. It is however only applicable for subcritical flow. The language used is FORTRAN IV and run in a CDC CYBER 175 machine. The package is documented in English and in metric units. It was developed and updated in 1978 by Delft Hydraulics Laboratory, The Netherlands.

S11-H6. This package is used for the computation of flow and water levels in network of channels of natural configuration. Its possible applications are for tidal wave, flood routing, dam break computations for process control and system analysis. For the solution of the conservation equations of mass and momentum, an implicit staggered difference scheme is derived, solved with a double sweep algorithm for tridiagonal matrices. The system is user-oriented with an independent coding system for grid points, branches, nodes and boundaries. Cross references are specified in a clear format-free input table. Wind stresses and lateral flows can be included. Its restriction is that it assumes nearly horizontal one-dimensional flow. The language used is PL/I and run in an IBM370/165. Documentation is in English and metric units. It was developed and updated in 1976 by Danish Hydraulic Institute, Denmark.

Katopodes, N.D. (1980)

A finer space grid for the method of characteristics will produce better results for channels slightly deviating from prismatic shape. However, if severe contractions or expansions of the channel width are present, the one-dimensional method of characteristics fails dramatically, regardless of the detail of discretization. High volume errors have been observed in such computations, and in general, the method is very unreliable when energy dissipation due to reasons other than bed friction is present. At high courant numbers, the computation is limited by reasons other than stability. In the presence of highly nonlinear regions in the solution domain, the convergence of the Newton-Raphson method becomes poor, due to the strong curvature of the characteristic curves. Thus, refinement of the grid in both time and space direction is necessary, and because the nonlinearities occur only locally, the efficiency of the method of characteristics at prescribed time increments is very low. Another difficulty associated with high courant numbers is that linear interpolation is no longer an option.

Schaffranek, R.W., Baltzer, R.A. and Goldberg, D.E. (1981)

A one-dimensional numerical model, the branch-network flow model, is used for simulating the unsteady flow in singular riverine or estuarine reaches and in networks of reaches composed of interconnected channels. It uses a four-point, implicit, finite difference approximation of the unsteady flow equations. The flow equations are linearized over a time step, and branch transformations are formulated that describe the relationship between the unknowns at the end points of the channels. The resultant matrix of branch transformation equations and required boundary condition equations is solved by Gaussian elimination using maximum pivot strategy. The model is both general and flexible since it can be used to simulate a wide range of flow conditions for various channel configurations. The channel geometry of the network to be modeled should be sufficiently simple so as to lend itself to characterization in one spatial dimension. The flow must be substantially homogeneous in density, and hydrostatic pressure must prevail everywhere in the network channels. The slope of channel bottom should be mild and reasonably constant over its length so that the flow remains subcritical. The model accommodates tributary inflows and diversions and includes the effects of wind shear on the water surface as a forcing function in the flow equations. Water surface elevations and flow discharges are computed in channel junctions, as well as at specified intermediate locations within the network channels.

Yen, C.L. and Hsu, M.H. (1982)

A one-dimensional numerical model was developed for simulating unsteady flow in a short river with several tributaries. Equations of motion and continuity in discretized form are employed for each subreach. At the confluence of the main river and its tributary, junction conditions were imposed, i.e., water stage being single-valued and inflow equal to outflow. With these, a set of nonlinear algebraic equations relating discharges and water stages at specific points in the river system is established for each time step. Solution is obtained by Newton-Raphson iteration technique. The model was applied to simulate the Tamsui river system in Taiwan. Simulated results are in good agreement with the observed data.

Li, Z.C., Zhan, L.J. and Wang, H.L.(1983)

Non-equidistant difference scheme in interior mesh points and the corresponding difference schemes on boundary mesh points are employed to obtain the unsteady flow in channels without branching. In many other schemes, the difference sections are fixed in an equidistant distribution along the channel direction. Therefore, not all important points along the channel could be generally, set just on the difference sections. Thus, in order to obtain their numerical solution, interpolation has to be done. This is inconvenient and produced a new interpolation error. Furthermore, when the cross section of channel varies greatly, the non-equidistant schemes are superior to the equidistant schemes. It is likewise valid for calculating both continuous and discontinuous unsteady flow in channels.

University of Iowa (1983)

The University of Iowa has compiled the following computer programs for one-dimensional unsteady flow in open channels.

CARIMA. This program computes one-dimensional steady or unsteady flow in fixed-bed channels, and quasi-two-dimensional unsteady flow on flooded plains. There is no restriction on the way channel and flood-plain flow paths are connected; branched or looped systems are accepted. Hydraulic works such as weirs, gated flow control structures, culverts and irrigation canal control systems are included in the standard program. CARIMA is typically used for: i) studies of flood propagation for protection works design and flood area delineation; ii) evaluation of effects of local flow modification structures and cutoffs on water levels and flood propagation; iii) design of operating systems for run-of-river hydropower installations; iv) design of irrigation canal flow control devices and operating systems; and v) evaluation of effects of peaking hydropower releases downstream navigation. For a given topographical and hydraulic description of a channel/flood-plain network, CARIMA computes the time-variation of water level, discharge, and velocity at designated computational points.

The steady flow in channels is computed using an implicit finite difference approximation of the de Saint-Venant flow equations. The unsteady flow on the flood plain is computed using non-inertial, simplified flow relationships between adjacent flood plain cells, whose submergible boundaries correspond to natural obstacles to flow such as road embankments, railroads, beams, etc. The relationships between cells, when linearized, discretized, and combined with the finite difference equations for channel flow, form an algebraic system which is solved in each time step using a looped double sweep algorithm.

CAREDAS. This program computes one-dimensional steady or unsteady flow in branched or looped networks of pipes, closed conduits, and canals. Hydraulic works and control structures such as weirs, inverted siphons, manholes, retention basins, and pumping stations are included as standard features. Given input hydrographs of surface runoff from urban catchments (which can be computed by CAREDAS itself if desired), and a topographic, hydraulic, and topological description of the network, CAREDAS computes the time variation of water levels (or piezometric heads), velocities, and discharges at designated computational points. A separate program in the CAREDAS system uses these results to compute pollutant propagation in the network, if desired. Typical uses include: i) analysis of flow distribution in complex networks for the optimization of pipe sizes; ii) sizing of retention basins; iii) design of real-time operating systems for control of flow regulation structures; and iv) verification of overall network design.

The Saint-Venant equations for one-dimensional, free surface unsteady flow are solved using an implicit finite difference scheme with a special double sweep algorithm for looped flow paths. Pressurized flow is computed using the same method, the piezometric head corresponding to the free surface level in a thin slot, or chimney, running longitudinally above each closed conduit. The transition between channel and pressurized flow is smoothed, when necessary, by iterative corrections in the numerical algorithm during one time increment. Backwater effects, flow reversal, and the effects of in-pipe storage capacity are all naturally included in the Saint-Venant formulation. Although CAREDAS includes an optional general routine for generating surface runoff hydrographs, any method or program valid for local conditions can be used in its place.

Bertrand, G. and Zech Y. (1984)

A computer program using the method of characteristics in a computation grid with fixed node points, and various node density is developed for the computation of water flow in complex reaches such as that of canal Brussels-Charleroi. The method is chosen since it is suitable to unsteady flow problems at complex boundary conditions and geometric shape such as different cross-sections at two continuous sections of the channel, junctions of three branches, local constriction and discharge into or out of the canal. The results show good agreement between the computed and observed water levels.

Bribiesca, J.L.S. and Mariles, O.A.F. (1984)

A finite difference procedure which produces a tridiagonal system of linear equations is developed to solve problems on open-channel system under unsteady flow conditions. It involves i) dividing the channel into several consecutive segments and establishing dynamic and continuity equations inside each segment thus, expressing the velocities at the extremes of each segment as linear functions of the depths at those boundary conditions; and ii) setting continuity equations at each one of the sections located between two neighboring segments, thus, establishing for each section a linear equation relating the depth in it with those of the upstream and downstream neighboring sections, and at the same time, to take into account inflows and outflows from the channel. The procedure allows consideration of special boundary conditions along the channel such as control gates, steps, and others. It is also possible to account free or submerged discharge conditions as well as the transition from one condition to the other. The method is also useful for unsteady flow analysis in supercritical regime under certain conditions.

Fenton, J.D. (1985)

A method for the numerical solution of all formulations of the one-dimensional equations for unsteady open-channel flow is developed. The family of schemes which result are related to the grid-oriented characteristics methods. However, they have a number of special features which leave them with practically none of the disadvantages which conventional methods possess. The expressions given, after some lengthy derivation, are simple, explicit, unconditionally stable, of given accuracy (which is exact for linear equations, otherwise of first order only) valid for subcritical and supercritical flows. They describe the propagation and interaction of bores, and treat irregular channels simply. This latter feature is derived from an unusual feature of the method, which is the given time-stepping formulae are independent of the method of spatial interpolation along the channel.

Schaffranek, R.W. (1985)

A one-dimensional model for simulating unsteady flow in a single open-channel or in a network of interconnected channels is formulated. The model is both general and flexible since it can work on a wide range of flow conditions for various channel configurations. It is based on a four-point (box) implicit, finite-difference approximation of the governing nonlinear flow equations with user-definable weighting coefficients to permit varying the solution schemed from box-centered to fully forward. To achieve operational modeling capability, the model is linked to a highly efficient storage-and-retrieval module that accesses a database containing time series of boundary values and by including an extensive set of digital graphics routines. These features transform the model into a comprehensive tool for practical use in the conduct of hydrologic investigations.

Fennema, R.J. and Chaudhry, M.H. (1986)

Three second-order accurate, explicit finite-difference schemes (MacCormack, Lambda and Gabutti) are introduced and compared for the analysis of unsteady, free surface flows having shocks or bores. The details of these schemes, their shock capturing capabilities, stability restrictions, boundary conditions, and use of artificial viscosity to dampen the numerical oscillations near the shock are presented. Results show that based on the computer time required for reproducing the shocks with a similar accuracy, the Preissman's implicit scheme takes four to eight times the CPU time taken by these explicit schemes. Because the latter are easier to program, it makes them more attractive to real-time applications.

Gooch, R.S. and Graves, A.L. (1986)

The Aqueduct Control Software (ACS) of the Central Arizona Project is a series of hydraulic models that generate schedules for pumps and gates based on the initial conditions of the aqueduct, the water delivery schedule, and the availability of the pumps and gates in the system. It consists of five interconnected models namely; i) initial conditions model (ICM), ii) staggered net unsteady state model (SNUSM), iii) baseload model (BLM), iv) gate setting model (GSM), and v) reach 1 simulation model (RSM). The purpose of ICM is to provide a flow and depth profile for the entire aqueduct at the time of the run for use by the other models. SNUSM is an explicit finite difference method solved in a fixed space-time grid. Each point on the grid represents a unique solution of the Saint-Venant equations. The staggered net is used for stability reasons. Boundaries are solved using a boundary definition equation in combination with grid boundary equations.

BLM is the heart of the scheduling process. It is within this model that the control volume philosophy of operation is implemented. Its primary purpose is to generate pump schedules so that each pool maintains a target volume. It is also aimed at generating boundary depths and flows to be used by GSM to generate gate schedules. GSM uses two different schemes-gate stroking and gate setting based on current state. Gate stroking is the movement of checkgates in such a way as to minimize transient activity in the canal. A modified version of the USBR gate stroking model was incorporated in the ACS. A major drawback to the gate stroking model is that the time required to run the model is greater than is allowable.

This problem was solved by using a gate setting model using steady state equations of motion as an approximation of the actual state, and using flow schedules as the basis for calculating water surface profiles. The purpose of RSM is to develop a weekly pump schedule for the Havasu pumping plant that will use as little on-peak as possible. A linear programming optimization model was developed for this purpose.

Hamilton, D.L. and deVries, J.J. (1986)

A computer model for the hydraulic simulation of unsteady canal operation was developed. The model is applicable for any nonbranching canal system compose of a series of channels separated by control structures. A four-point, implicit finite difference scheme known as Priessmans method was used to transform the Saint-Venant equations into a set of linear, simultaneous equations. Double sweep method was employed to solve the equations. The model simulates the operation of a canal system from an area control center or by remote control. The advantage of this method is that canal can be operated more efficiently and with greater flexibility. Flows throughout the entire system can be adjusted simultaneously, and the effects of the control actions can be monitored in real time. Since the study is concerned with the approximate response of a canal system to various techniques of operation, exact reproduction of flow behavior is not essential.

The canal chosen for the study is the 108 km Delta Field Division canal of the California aqueduct which is controlled and monitored from a single location near Tracy, California which is near the upstream end of the canal. It is composed of eleven reaches with twelve check structures, and each check scheme has four radial gates. The simulation describes a typical simulation of off-peak/on-peak operations involving staggered gate change procedure to reduce transient oscillations in the canal pools. A comparison of computed results to observed data indicates good agreement.

Katopodes N.D. and Wu, C.T. (1986)

A method which is suitable for computing discontinuous flow is presented. An explicit finite element model for free surface flow was developed and shown to be second and fourth-order accurate with respect to the time and space increments, respectively. The explicit nature of the method allows an equally accurate computation of supercritical flow. The results indicate that the method is somewhat limited with respect to stability requirements, i.e. courant number must be less than 0.58 for fourth-order spatial accuracy or less than 1.0 for second-order spatial accuracy. It was also shown that the method must remain second-order accurate in time in order to be stable at all. If the required accuracy of the solution demands smaller time increments, as in the case of discontinuous supercritical flow, the present method becomes a powerful and versatile computational tool for free-surface flow simulation. It should be noted the method's dissipative character is globally controlled by the size of the time step selected. This means that algorithmic damping cannot adapt to changing flow conditions unless a variable grid is used for the spatial discretization.

Gichuki, F.N., Walker, W.R. and Merkley, G.P., (1987)

A mathematical model of branching canal network is developed. The model simulates canal filling, operating and draining phases, bulk lateral inflow or outflow, and control structure scheduling. The model has three water control options: i) operator decision control in which the model user decides on what inflows and control structure settings should be implemented; ii) program generates system inflow and the control structure settings which achieve predetermined flow depths; and iii) program generates system inflow and the control structure settings which deliver a specified discharge to the lower reach.

Lyn, D.A. and Goodwin, P. (1987)

The stability and convergence characteristics of a four-point implicit finite difference scheme or Priessman scheme are examined. The analysis is made for a general linear hyperbolic system of the first-order equations, but is restricted to the homogeneous or frictionless case. In particular, the effect of weighting factor in space, as well as in time, is considerable. It has become standard to choose a scheme centered in space, $\phi = 1/2$, since this yield second-order accuracy in time. However for $\phi = 1/2$, it is necessary that $\theta > 1/2$ for unconditional stability. For a non-standard choice $\phi \neq 1/2$, the stability will depend on the sign of the courant number, or equivalently, in the direction of travel of the characteristic wave. Because of this, the use of $\phi = 1/2$ should be used with caution in situations where characteristic direction may change, e.g. in transition from subcritical to supercritical flow. Although a given difference scheme may be stable, its convergence may be poor. This is usually discussed in terms of its dissipative and dispersive characteristics. For example, in the analogous fixed bed problem, it is often chosen to increase the value of the time weighting factor, θ towards unity in order to dampen parasitic oscillations. However, this produces some inaccuracies.

Merkley, G.P., Walker, W.R. and Gichuki, F.N. (1987)

A hydraulic simulation model is developed at Utah State University for improving water management in canal main systems. The model can simulate transient and steady flow conditions in main canal. Its applications are in the areas of operation, design and operator training. As an operational tool, the model can be used to schedule control structure settings according to predetermined water distribution strategies while maintaining stable flow levels. As a canal system design tool the model can be used to analyzed proposed designs by simulating the hypothetical canals under expected operational conditions. As a training tool, it can be used by the canal operators to simulate a wide range of possible flow conditions safely and quickly, thereby allowing the operator to become more familiar with the operational response and limitations of a canal system. The hydraulic model is interactive and menu-driven, with extensive use of computer graphics to display simulation results.

Murthy, J.S.R., Shah, H.H. and DeVries, J.J. (NDD, 1987)

A computer model which can simulate canal hydraulics for all expected operational conditions is developed for the Narmada canal project in India. The model permits the formulation of efficient rules for normal operation and operation criteria which ensure safe shutdown of the canal during emergencies.

Traver, R.G. and Miller, A.C. (1987)

A more robust solution to the unsteady flow equations was developed by reformulating the Saint-Venant equations of open-channel flow. The conservation of mass equation is numerically stable in its complete form and thus was used without any modification. On the other hand, to make the conservation of momentum equation stable, it was reformulated into a friction slope form which included all terms of the complete differential open channel flow momentum equation with the exception of the pressure correction term. The turbulence correction term was estimated by the entrance/exit loss equation. A predictor-corrector scheme was used to solve the Saint-Venant equations. The predictor is an explicit finite difference scheme which relies heavily upon information from the previous time step to predict the next time line's flow and depth. The corrector which is an implicit scheme will be applied repeatedly until the desired accuracy is obtained.

Abdul-Salam, S.M. (1988)

A one dimensional formulation of the Saint-Venant equations is developed for simulating the flow in a single reach or a confluence of two reaches, using the four-point implicit finite difference formulation. The model solves the nonlinear equation using the Newton-Raphson iteration method and a modified Gaussian elimination technique. The Tigris river in Baghdad and its confluence with Diyalah river is used for model calibration. Results indicate the feasibility of such techniques and accuracy of results. One conclusion is that the use of the fully backward difference scheme specified by $\theta = 1.0$ provides computational stability that is independent of the values of the other parameters in addition to the possibility of using a large time step Δt. Values of the weighting factor θ that gives stable and accurate solution appear to be in the range $0.75 \leq \theta \leq 1.0$.

Husain, T., Abderrahman, W.A., Khan, H.U., Khan, S.M., Khan, A.U. and Eqnaibi, B.S. (1988)

The applicabilities and limitations of various unsteady flow models/packages for approximating the solution of Saint-Venant equations were reviewed considering Al-Hassan irrigation system as case study. These are the following:

Gradually varied unsteady flow program (GVUFP). This program was developed by the Tennesse Valley Authority (TVA) using explicit finite difference technique. The program is applicable to single river systems. One of its limitations is that Δx should be kept small and equal.

System hydraulic package (SHP). This package was developed by Resource Management Association for Hydrologic Engineering Center (HEC) using the finite element method. The program is applicable to single river systems of small computation time steps.

Branched network flow package (BNFP). This package was developed by the US Geological Survey (USGS) using an implicit finite difference technique. The program is applicable to single river systems.

Dynamic wave operational model (DWOPER). This package was developed by the National Weather Service (NWS) using an implicit finite difference technique. The program is applicable to single river system. Recent version of the program can also model branch-network problems. The package include options on boundary conditions and special computational algorithm to provide numerical stability.

The channel network model (CNM) which is developed in this study is a modified version of DWOPER. It retains the capabilities of DWOPER such as simulation of flow through a river system with contributing tributaries. It can also be applied to distributing junctions, which are used to divert water into various branches of the system. The model is capable of simulating flow through submerged and free flow types of gates with continuity as well as, discontinuity, in the bed slopes and at critical flow sections. The gate coefficient can be defined as a function of differential head instead of taking a single - gate coefficient value. Another important feature of CNM is that, depending upon the irrigation schedule operation, a part of the irrigation system can be discarded from the analysis without altering the input data files. External boundaries are specified at the beginning and end of a particular reach considered for simulation. The junctions, gates, and critical drops are treated by the model as internal boundaries.

In locations such as junctions, gates, and critical drops along a channel where the Saint-Venant equations are not applicable (flow is rapidly varied rather than gradually varied and cross-sections are specified for the upstream and downstream extremities), Δx can be any value from zero to the actual measured distance between the cross-sections. The required equations include an empirical, rapidly varied flow relation while the second represents the conservation of mass with negligible time dependent storage. In addition, other features can be summarized as i) interactive capability, ii) use of subsystems; iii) geometrical data preparation and initial conditions; and iv) dynamic node numbering.

Lai, C. (1988)

A multimode method of characteristics scheme combining implicit, temporal reachback, spatial reachback, and classical schemes into one, is derived in this study. Three numerical models are developed to implement the implicit and multimode schemes. The IMOCDS model uses an implicit scheme, with which the time step is no longer subject to the courant constraint. NEWMOC and SPRMOC which are two versions of multimode scheme, demonstrate all the advantages previously provided by individual STI schemes over the combined flow range of the various schemes involved, and display newly acquired benefits such as robustness.

Rijo, M., Pereira, L.S. and Almeida, A.B. (1988)

An implicit finite difference method was used to simulate the unsteady flow in an irrigation canal. The system of equations was solved by double sweep method. The model was applied to the Salvaterra canal of the Sorraia irrigation project in Portugal. Results confirm that the implicit procedure is unconditionally stable for $0.5 < \theta \leq 1.0$, being unstable for $0 < \theta \leq 0.5$. A sensitivity analysis was performed to evaluate whether the incorporation of all terms in the dynamic equation is necessary. The result shows that the dynamic model without convective acceleration and the diffusion model with convective acceleration gave very good results, with significant savings in computer time. With the diffusion wave model or complete diffusion, the numerical solution becomes unstable i.e., numerical instability was obtained before the arrival wave and a less accentuated damping than the others after the arrival wave, concluding that it is not convenient for use. For kinematic wave model, no results could be obtained.

Verwey A., Van Haperen, M.J.M. (1988)

A user-friendly package for the simulation of unsteady flow in open channel networks is developed. It can be used for studying a wide range of hydraulic engineering problems such as i) flood wave propagation through channels, rivers, flood plains and reservoirs; ii) tidal flow in rivers and estuaries; iii) effects of structures in channel systems; iv) optimum design and operation of irrigation and drainage systems; v) wave propagation resulting from dam failures; and vi) hydraulic parameters in water quality study.

The model is based on the full Saint-Venant equations solved with a highly accurate and efficient modification of Priessmans implicit finite difference scheme. It is very flexible in specifying external and internal boundary conditions. The user can select from a number of system elements to simulate complex flood flow over flood plains or define structures at any point of the channel system such as weirs, gates, culverts, siphons, spillways, sluices, stormsurge, barriers, dikes, etc. Its user-friendliness is exemplified by the following features: i) separate processing for input, execution and output subsystems; ii) extended free format data input; iii) possibility to add user defined subroutines and functions; iv) all user-defined model elements; v) automatic generation of computational grid and elements following user's directives; vi) use of special information symbols to minimize input efforts; vii) extensive checking of input data; viii) continuation of input processing after detection of errors; ix) restart facilities in model execution; x) possible generation of output at any point of the channel system and for a large range of parameters. The program is written in Fortran 77 following the full ANSI standard.

Barkau, R.L., Johnson, M.C. and Jackson, M.G., (1989)

An unsteady flow model (UNET) for the solution of the full network problem is developed. The program applies a linearized implicit finite difference scheme to solve the full unsteady flow equations. UNET uses modified HEC-2 format for specifying system geometry, can use the HECDSS database for input and output data storage, and uses graphics for its primary mode of output. It supports a full set of interior boundary conditions including bridges, culverts, low water dams, gated spillways and weirs.

Schuurmans, W., (1989)

An analytical formulation based on a simplified formulation of the Saint-Venant equations is used to estimate the system response time and to show the parameters affecting it. The response time is defined as the time required to transit from one steady state into a new steady state. Moreover, delivery performance criteria are formulated which can be used to evaluate the delivery performance for unsteady flow conditions.

Sally, H., Berthery, D., Certain, F. and Durbec, A. (1989)

A model which is mainly based upon the well-established computer software of CEMAGREF, France in the field of open channel hydraulics for the design and operation studies on rivers and canals is developed. It is made up of three software units which can run independently or sequentially. These are i) topography unit which enables the user to input and verify all the topographic data pertaining to the canal; ii) steady flow unit which allows for the calculation of regulator and offtake gate openings as well as water surface profiles in the different canal reaches for any steady state scenario of water supply and demand; and iii) unsteady flow unit which allows the simulation of unsteady flow conditions that prevail over certain periods of time in response to changes in water supply or demand, and modifications to gate settings. A major component of the model is the development of a user-friendly conversational procedure so that the model could be used by non-specialized staff. The model is applied to the Kirindi Oya RBMC in Sri Lanka.

Manz, D.H. (1990)

The Irrigation Conveyance System Simulation (ICSS) Model was used to predict the operational characteristics of irrigation conveyance systems typical of Southern Alberta in Canada. A sensitivity analysis was done to assess the input of various operational procedures, designs, and physical characteristics on the magnitude of water losses due to seepage and operational spillage. The model is design to simulate the hydraulics and operation of lined and unlined irrigation canals. The canals may contain any number of distinct prismatic reaches of varying lengths, cross sectional shapes and roughness. Each reach is bounded by one or more hydraulic structures. No restriction (hydraulic or operational) is placed on the types of hydraulic structures considered. Provision is made to consider seepage, precipitation, drainage inflow, evaporation or other distributed losses or gains.

Swain, E.D. and Chin, D. A. (1990)

A numerical model is developed to simulate unsteady flow in open-channel networks. The model is an improvement over currently available network models in that the effects of hydraulic structures and leakage through the riverbed are incorporated. In addition, a matrix solution method is developed that takes advantage of the banded nature problems and, in some cases, reduces the solution time drastically. A method of reducing the bandwidth of the coefficient matrix is presented. Verification by comparison to other solution methods shows close agreement. The model is useful for simulating gate operational rules, analyzing effective flood-control measures, and comparing water delivery schemes over large distances where there is significant loss to leakage.

Merkley, G.P., Walker, W.R. and Gichuki, F.N., (1990)

A hydraulic model is developed to simulate and help manage branching canal systems. Gate scheduling, one of the three operational modes available in the model, determines control structure and turnout settings with the objective of quickly stabilizing transient flow conditions, while maintaining flow levels at or near target values. Gate scheduling can be used to accurately match planned turnout deliveries and eliminate operational uncertainties. This can be accomplished through manual control structure operation, without infrastructure changes or automation equipment, Following field calibration, the model was applied to the scheduling of canal operations at an irrigation project in northeast Thailand. Field and model-generated data showed that significant operational improvement for canal networks can be achieved with the model. The model is implemented on IBM PC/AT or PS/2-type microcomputers with a five-minute simulation time step.