PRAIRIE HYDROLOGY WORKSHOP

No. 3

OCTOBER 18 - 19, 1988

SASKATOON, SASKATCHEWAN

DECEMBER 1988

PPWB REPORT No. 105

FOREWORD

The Prairie Provinces Water Board Committee on Hydrology held its third Prairie Hydrology Workshop on October 18-19, 1988 in Saskatoon. Previous Workshops were held in 1984 and 1982 in Brandon and Lloydminster, respectively.

The Saskatoon Workshop was organized and co-chaired by COH Members, Mr. A.B. Banga and Mr. F.R.J. Martin, and was attended by 45 people from the Board's member agencies. A list of the participants is contained in the Appendix. During the two-day period, nineteen presentations were made covering the following topics:

1. Prairie Provinces Water Board Activities.
2. Significant Hydrology Events.
3. Hydrologic and Hydraulic Models.
4. Water Management Models.

In addition, Workshop participants heard a presentation by Dr. W. Nicholaichuk, Acting Chief, Hydrology Division, National Hydrology Research Institute (NHRI) on the evening of October 18th describing the organization and activities of the NHRI.

This report contains the papers presented at the Workshop which describe various hydrologic, hydraulic and water managment models used by Board member agencies.

R.L. Kellow
Executive Director
Prairie Provinces Water Board
January 13, 1989

WORKSHOP WRAP-UP

by F.R.J. Martin
PFRA Hydrology Division

The numerous models that have been presented and discussed during this workshop have all evolved to serve somewhat different purposes. As such, it is very difficult to provide a general comparison of the various features of the models because they all contain slightly different features. Furthermore, each agency has a somewhat different perspective of the usefulness of the models, depending upon their own specific needs and applications. Thus, if any comparisons are to be made, they should be made by each agency, and the features of each model should be assessed with due regard to the agencies' perspective.

The following points summarize some of the more salient aspects of this workshop and the workshop subject matter (i.e. models).

1. Three important aspects of selecting an appropriate model are:
   a) understanding the study requirements,
   b) recognizing and understanding the model limitations, and
   c) selecting the simplest model that meets the study requirements.

2. Workshops of this nature help the personnel of COH member agencies to become more familiar with the various computer programs and models that are currently in use. However, model users must also continually browse the literature and interact with counterparts in other agencies to keep abreast of developments.

3. Models are in a constant state of flux. What may be a limitation today may not be a limitation tomorrow.

4. A great deal of time and effort is required to assess or evaluate potential models. Consequently, agencies tend to stick with models that have been selected (for whatever reason) for a wide variety of applications rather than use a number of models. It seems that most agencies prefer to use one or two models of which they have a relatively complete understanding rather than several models of which they only have a superficial understanding.

5. The user should never lose sight of the fact that the model output is only as good as the model input. In other words, garbage in equals garbage out.

CONTENTS

PART II: WATER MANAGEMENT MODELS

APPENDICES

HYMO

The HYMO program is a single event, rainfall-runoff model. It transforms rainfall data into runoff hydrographs and then routes these hydrographs through streams, valleys and reservoirs. The program was designed for planning flood prevention projects, forecasting floods and research studies. HYMO has gained wide spread use and acceptance over similar models because of the ease of data entry and its program structure which permits easy modification. The strong connection between program input and easily measured watershed features have also helped in the acceptance of this model.

Hydrologic Procedures used by HYMO

HYMO uses the Soil Conservation Service (SCS) rainfall-runoff relationship to calculate incremental runoff from the basin for each time step. The SCS method consists of defining a curve number which represents the type of soil, the land use and the antecedent moisture conditions. Once the curve number is defined for a specific event, the rainfall-runoff relationship for the basin is set.

HYMO then converts the calculated incremental runoff into a unit hydrograph. The unit hydrograph used by HYMO was developed by the U.S. Dept. of Agriculture using data from 34 small watersheds in the Western States. The unit hydrograph shape is defined by any two of the following:

- B - the peakyness factor
- Tp - time to peak
- K - the recession constant

For a gauged basin, Tp and K may be determined from analyzing several runoff hydrographs. For ungauged basins, the program calculates Tp and K based on the basin slope and watershed width-length ratio.

The above computations are carried out by one subroutine in the HYMO program. Input required by the model is the average mass rainfall curve over the basin, the time increment, the SCS curve number, the drainage area, and Tp and K or the basin slope in terms of height of basin and length of basin. The output consists of the calculated runoff hydrograph.

Flood routing through streams and valleys is carried out using a modified variable storage coefficient (VSC) flood routing method. The modification made to the VSC method takes into account the variation in the water surface slopes during a flood. Another feature of the VSC method is that rating curves are required at enough locations along the valley to adequately describe the stream and valley hydraulics. Since measured rating curves at a number of sections are usually not available, HYMO provides a calculation procedure for determining rating curves using Manning's equation.

The above computation are carried out by four subroutines in the HYMO program. Input required by the model consists of measured rating curves, definition of valley cross sections, the section Manning's n for the stream, the section

Manning's n the left and right flood plain, the section stream slope, the section flood plain slope, distance and slope between sections and the inflow hydrograph. Output consists of computed section rating curves, time of travel between sections for various flow rates, and the outflow hydrograph.

Routing through reservoirs by HYMO is done by using the storage-indication method. Reservoir routing is carried out by one subroutine in the HYMO program. Input consists of outflow-storage curve and the inflow hydrograph while output consists of the outflow hydrograph.

HYMO also contains several subroutines for entering, adding, comparing, printing and plotting hydrographs. This allows the user to subdivide larger basins and compare results with measured hydrographs during the calibration and verification of the model for a specific basin.

Program Limitations

The program has several limitations which may cause varying degrees of difficulty. These limitations are:

1 The present version of HYMO used by Sask Water carries out all computations in imperial units. Therefore, input data must be converted from metric and output results must be converted back to metric.

2 The HYMO procedure for estimating Tp and K based on basin width-length ratio and slope are not representative of Canadian western prairie conditions. The majority of basins in the western prairies are much flatter and have significantly more depression storage than the small watersheds used to develop the above estimation procedure. Therefore, other methods should be used to calculate the Tp and K for western prairie basins.

3 Some of the hydrologic procedures used by the model also have a number of limitations. For example, the SCS method uses a fixed initial abstraction, is based on runoff from 24 hour storms on small agricultural basins, does not relate runoff to rainfall intensity and is essentially empirical in nature. Another example is the unit hydrograph which is better known for its limitations such as fixed runoff time and hydrograph shape for a wide range of runoffs. These limitations discussed above are due to using simple procedures to explain complicated real world hydrologic events. However, more complicated procedures are not practical since little real data is available to better define the hydrologic process.

HEC-1 FLOOD HYDROGRAPH PACKAGE

by
James B. Yarotski
PFRA Hydrology Division

1. Introduction

The HEC-1 computer program, developed by the U.S. Army Corps of Engineers, simulates the surface runoff response of a watershed to precipitation by representing the watershed as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Each of the components is represented by a set of parameters and mathematical relationships which describe the physical process. The result of the modelling process is the computation of streamflow hydrographs at desired locations in the watershed.

The simulation of the precipitation-runoff process is of central importance to virtually any application of HEC-1 and will form the basis of this presentation. Other capabilities of HEC-1, which are built around this precipitation-runoff process, are as follows.

1. Multiplan-multiflood analysis which allows the simulation of up to nine ratios of a design flood for up to five different plans of a stream network in a single computer run.

2. A dam-break simulation provides the capability to analyze the consequences of dam overtopping and structural failures.

3. An economic assessment of flood damage can be determined for damage reaches defined in a multiplan-multiflood analysis. The expected annual damage occurring in a damage reach and the benefits accrued due to a control plan are calculated based on user-supplied damage data and on calculated flows for the reach.

4. The optimal size of a flood control system can be estimated using an optimization procedure provided by HEC-1. The option utilizes data provided for the economic assessment option together with data on flood control project costs, to determine a system which maximizes net benefits with or without a specified degree of protection level for the components.

The Hydrology division of PFRA has used the precipitation-runoff component of HEC-1 to estimate probable maximum flood (PMF) hydrographs for existing or proposed dams. The program has been used to estimate the runoff hydrograph from either extreme rainfall, snowmelt or rain-on-snowmelt events. The model is a single event oriented model and is not suited for long-term hydrology studies of a river system.

2. Precipitation-Runoff Process

The program begins the simulation of a watershed by determining the average subbasin precipitation from either historical gauged data or hypothetical storms for a specified computational interval selected for the watershed simulation. The amount of excess precipitation is then calculated based on a loss rate function. In the HEC-1 model, precipitation losses refers to precipitation lost to land surface interception, depressional storage and infiltration. The excess precipitation is then distributed in time by a unit hydrograph function or by the kinematic wave method. The duration of unit excess is automatically set equal to the computation interval. To account for base flow, a base flow function is added. This yields the total runoff hydrograph at the subbasin's outlet. The runoff hydrograph from the subbasin is then conveyed downstream to the next subbasin using a streamflow routing function. Runoff from the next subbasin is then calculated and combined with the routed hydrograph. This process is repeated until the entire watershed has been simulated. The number of subbasins in a watershed is somewhat arbitrary; however, in subdividing a watershed, consideration should be given to the varying runoff characteristics of the watershed and the spatial variation of the precipitation.

The model's automatic calibration features can be used to select unit hydrograph and loss rate parameters in a single subbasin or to choose the routing parameters in an individual river reach. In both cases, the calibration is based on comparisons with observed and simulated hydrographs. If observed hydrographs are not available for all the subbasins within the watershed, the estimated parameters for a subbasin can be transferred to other subbasins with no data.

When estimating parameters based on an observed runoff hydrograph, the observed runoff hydrograph should be similar in magnitude to the runoff hydrograph that the estimated parameters will be used to produce. For example, estimated parameters for a relatively low runoff should not be used to estimate the runoff from a probable maximum precipitation (PMP) event. Similarly, the reverse is true.

2.1 Precipitation

The moisture input into the model can be either in the form of rainfall or snowfall. The specified precipitation hyetograph is assumed to be uniformly distributed over the subbasin and computation interval. The precipitation data may be specified as a storm total, along with a temporal pattern for distributing the total precipitation. The total storm precipitation or temporal distribution for a subbasin may be computed as the weighted average of measurements of several gauges.

Snowfall and snowmelt are simulated in each subarea according to temperatures in up to ten elevation zones within a subbasin. These zones are usually considered to be in elevation increments of 1,000 feet, but any equal increments of elevation can be used as long as the air temperature lapse rate corresponds to the change in elevation within the zones. The input temperature data corresponds to the bottom of the lowest elevation zone. Temperatures are reduced by the lapse rate in degrees per increment of elevation zone.

Precipitation is assumed to fall as snow if the zone temperature is less than the base temperature plus two degrees. Melt occurs when the temperature is equal to or greater than the base temperature. Snowmelt is subtracted and snowfall is added to the snowpack in each zone.

Either the degree-day or energy budget method may be used to compute the snowmelt.

2.2 Loss Rate Function

Precipitation losses to interception, depressional storage and infiltration may be simulated by one of the four loss rate functions: initial loss followed by a constant loss, the SCS curve number technique, the Holton loss rate or the HEC exponential loss rate function. Using any one of the loss rate functions, an average precipitation loss is determined for a computation interval and subtracted from the rainfall/snowmelt hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a subbasin. The precipitation loss functions can be used with either the unit hydrograph component or kinematic wave model components.

Two important factors should be noted about the precipitation loss computation in the model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery. This is why the HEC-1 program is a single event oriented model.

2.3 Subbasin Runoff

Subbasin runoff from excess precipitation can be calculated using either a unit hydrograph or kinematic wave method.

2.3.1 Unit Hydrograph

A runoff hydrograph is calculated by multiplying the excess precipitation by the unit hydrograph ordinates. The duration of the unit hydrograph is automatically set equal to the computational interval selected for watershed simulation (i.e. 10 minutes, 1 hour, 6 hours, etc.). Unit hydrograph ordinates can be directly input into the program or a synthetic unit hydrograph can be calculated using techniques proposed by Clark, Snyder or the SCS (Soil Conservation Service). The Clark method uses two parameters (a time of coefficient and a storage constant) and a time-area relationship to define an instantaneous unit hydrograph. Snyder's method uses two parameters, which define the peak of the unit hydrograph. The SCS dimensionless unit hydrograph technique uses a log parameter to define the shape of a triangular unit hydrograph.

The transformation of excess precipitation to a runoff hydrograph using a unit hydrograph is based on two important assumptions. First, the unit hydrograph is characteristic for a subbasin and is not storm dependent. Second, the runoff due to excess from different periods of rainfall excess can be linearly superimposed.

2.3.2 Kinematic Wave

The kinematic wave technique transforms rainfall excess into subbasin outflow. In determining subbasin runoff, three conceptual elements are used: flow planes, collector channels and a main channel. Runoff is calculated from each of the flow planes based on Manning's formula. Flow from the overland flow elements travels to the subbasin outlet through one or two successive channel elements. From the collector channels the flow is collected in the main channel.

2.4 Streamflow Routing

Once a runoff hydrograph has been calculated at the subbasin outlet, the hydrograph may have to be routed to a downstream point. Most of the flood routing methods available in HEC-1 are based on the continuity equation and some relationship between flow and storage or stage. The methods available in HEC-1 are: Muskingum, Working R & D, Straddle-Stagger, Tatum, Modified Pulse or Multiple storage and Kinematic Wave. In all of these methods, routing proceeds on an independent reach basis from upstream to downstream; neither backwater effects nor discontinuities in the water surface such as jumps or bores are considered.

2.5 Parameter Estimation

Estimation of parameters required by the various components in the precipitation-runoff process may be necessary. The parameters are often estimated by selecting values that yield the 'best' reproduction of a measured runoff event with the available measured precipitation data. HEC-1 provides a optimization technique for the estimation of some of the parameters when gauged precipitation and runoff data are available.

Unit hydrograph and loss rate parameters can be automatically determined if the basin average precipitation, basin area, starting flow, base flow parameters and the subbasin outflow hydrograph are known. Unit hydrograph and loss rate parameters can be determined individually or in combination. Parameters that are not to be determined from the optimization process must be estimated and provided to the model.

The 'best' reconstitution of the recorded runoff hydrograph for a subbasin is considered to be that which minimizes an objective function. The objective function is the square root of the weighted squared difference between the observed hydrograph and the computed hydrograph. The minimum of the objective function is found by employing the univariate reach technique that uses Newton's method. The univariate search method computes values of the objective function for various values of the optimization parameters. The values of the parameters are systematically altered until the objective function is minimized.

HEC-1 may also be used to automatically derive routing criteria for certain hydrologic routing techniques. Criteria can be derived for the Tatum, Straddle-Stagger and Muskingum routing methods only.

Streamflow Synthesis and Reservoir Regulation (SSARR) Model

S.J. Figliuzzi[1]

SSARR is a mathematical hydrologic model, which can be used to conduct a continuous or single event simulation. The hydrologic response for an entire river basin system or for a single aspect of the system can be simulated. The model is comprised of three basic components:

1. a generalized watershed model for simulating runoff

2. a river system model for routing streamflows from upstream to downstream points through channel and/or lake storage.

3. a reservoir regulations model, whereby reservoir outflow and storage can be analyzed given inflows and free flow, or any of several modes of operation.

Within Alberta, the SSARR model is extensively used for operational forecasting, system analysis, and channel routing, as well as, in the assessment of multiple reservoir systems and more recently, in the computation of probable maximum floods.

Input data requirements for the model are minimal since the model utilizes hydrologic relationships, some of which are general and applicable to many basins to describe hydrologic processes. The main input requirements include:

1. <u>Nonvariable Characteristic Data</u> which describe physical features such as drainage area, reservoir storage capacity and watershed characteristics that affect runoff.

1. S.J. Figliuzzi, P. Eng., Hydrologist, Hydrology Branch, Technical Services Division, Alberta Environment.

2. Initial Conditions Data for specifying current conditions of watershed runoff indices, flow in each increment of each channel reach, and initial reservoir or lake elevations and outflow.

3. Time Variable Data which includes physical data expressed as time series eg. precipitation data, air temperature, streamflow data, reservoir data, etc.

4. Job Control and Time Control Data which specify such items as total computation period, routing intervals, and computer instructions to control plots and reports.

Specific features of the model include:

1. The ability to fully account for and track all precipitation inputs.

2. The ability to conduct a continuous simulation while permitting a varying time-step for different portions of the simulation.

3. The ability to model the simultaneous occurrence of snow accumulation, snow melt and rainfall runoff.

4. The separation of excess precipitation into fast, intermediate and slow runoff phases.

5. The models ability to take backwater affects into account.

Limitations of the model include:

1. When working with the "split watershed option", a redistribution of snowpack occurs during periods of snow accumulation.

2. Since evaporation can only occur from soil moisture, the model treats sublimation as a lowering of the SMI rather than a reduction in the snowpack. This limitation however, may be overcome by introducing evaporation as a negative precipitation.

SSARR MODEL

1.0 INTRODUCTION

1.1 History

The Streamflow Synthesis and Reservoir Regulations Model (SSARR) has been in the process of development and application since 1956. The model was initially developed to provide mathematical hydrologic simulations for system planning and operation analysis of water control works as required by the Northern Pacific Diversion of the U.S. Army Corps of Engineers. The model was further developed for operational river forecasting and management activities in connection with the Cooperative Columbia River forecasting unit. Since its development the model has been used extensively in the U.S. and abroad. The uses range from simple routing assessments to system analysis, to operational forecasting, to probable maximum flood computations.

The model was initially introduced in Alberta in the early 1970's, by the Prairie Provinces Water Board, during the course of the South Saskatchewan River Basin Studies. In the mid 1970's the River Forecast Centre of Alberta Environment began using the model for both operational forecasting and systems analysis and further developed the model to meet their needs. Hydrology Branch of Alberta Environment, since the early 1980's has used the model extensively in channel routing studies, in the assessment of multiple reservoir systems and more recently in the computation of probable maximum floods.

1.2 SSARR Model Design Considerations

There are many design considerations which have gone into the development of the SSARR Model. The ones which we have found the most useful, and the reason for selecting SSARR over such models as HEC-1 or HSPF, are as follows;

a) The models ability to fully account and track all precipitation input to the system.

b) The use of practical, yet theoretically sound methods in evaluating hydrologic processes.

c) The models ability to provide a continuous simulation and the added ability to change the time step for specific portions of the simulation.

d) The models ability to simultaneously simulate the snow accumulation, snowmelt and rainfall runoff processes.

e) SSARR's ability to model fast, intermediate and slow response phases in the runoff process.

f) The models ability and flexibility in providing output, including data tabulations and plots, that can be readily assessed.

1.3 Conceptual Design of the SSARR Model

SSARR is a mathematical hydrologic model of a river basin system which can synthesizes streamflow by evaluating snowmelt and rainfall. The model is comprised of three basic components.

a) A generalized watershed model for synthesizing runoff snowmelt, rainfall, or a combination of the two.

b) A river system model for routing streamflows from upstream to downstream points, through channel and lake storage.

c) A reservoir regulation model, whereby reservoir outflow and contents may be analyzed for a given inflow and free flow or any of several modes of operation.

2.0 SYSTEM CONFIGURATION

Watersheds, lakes, reservoirs and channel reaches in a given basin can each be represented by one of the three basic SSARR components. The first step in any assessment is therefore, to subdivide the basin, based on data availability and basin characteristics, into units which can be represented by one of the SSARR components. The various components are then organized to produce a river basin model which simulates the hydrologic response of the physical system. A hypothetical river basin system and the corresponding basin configuration scheme are illustrated in figure 1. It is important to note that there may be more than one acceptable subdivision and organization of such a river basin.

3.0 WATERSHED MODEL

The SSARR watershed model incorporates various hydrologic relationships and other factors in the hydrologic cycle to synthesize streamflow. A schematic representation of the basic elements of the SSARR watershed model is presented in figure 2. As indicated in figure 2, the watershed model may be considered as consisting of four sections. These are:

a) a precipitation balancing section,

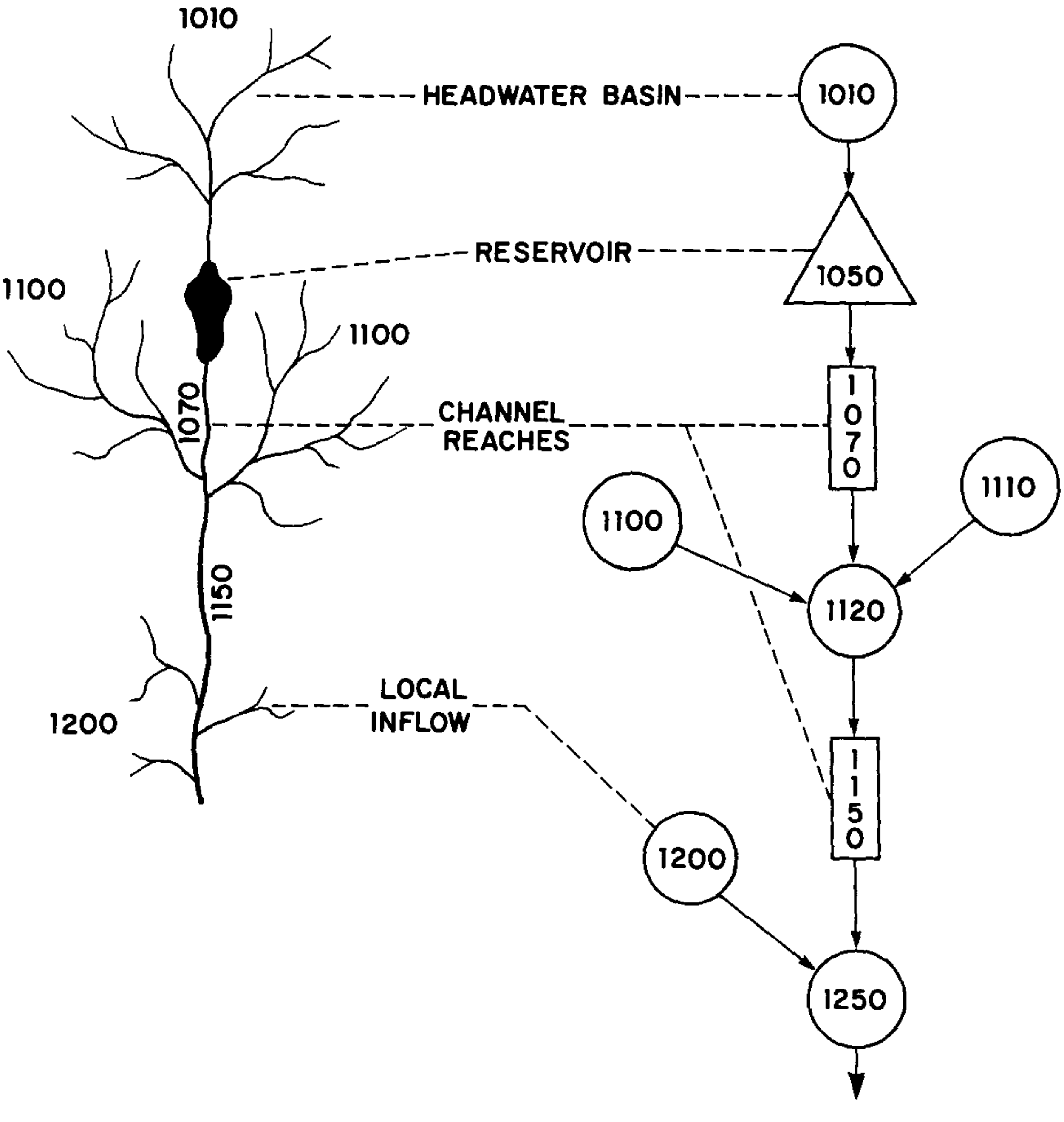
b) a moisture input balancing section,

c) a runoff balancing section, and

d) a hydrograph shaping section.

3.1 Precipitation Balance

The primary function of the precipitation balancing section is to compute the amount of moisture input and the remaining snowpack for each time step. To compute the moisture input and snowpack, three parameters have to be tracked continuously. The three parameters are: the melting elevation, the rain-freeze elevation and the snowline



HYPOTHETICAL RIVER BASIN
AND CORRESPONDING
SSARR MODEL SCHEMATIC

FIG. 1

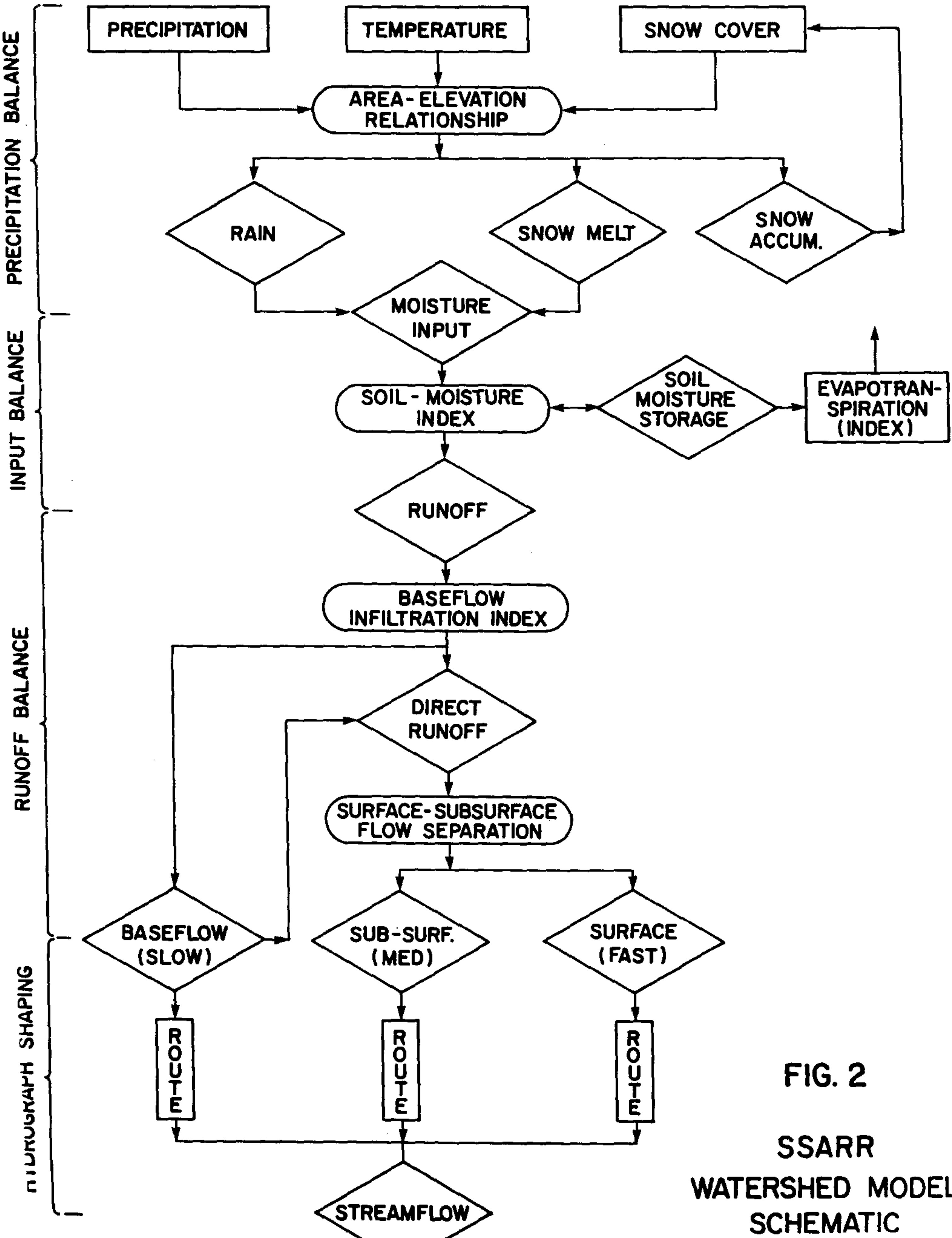


FIG. 2

SSARR WATERSHED MODEL SCHEMATIC

elevation. In determining the melting elevation and the rain-freeze elevation, the SSARR Model simply applies a specified station temperature and either a specified or default temperature lapse rate to a basin area-elevation curve. The model, however, has two options for determining the snowline elevation, the snowcover depletion method and the snow band method. The two options are shown in figures 3a and 3b, along with various conditions of precipitation and snowmelt that may occur simultaneously.

The snowcover depletion method uses a relationship relating the percent of maximum snow water equivalent remaining on the watershed, to percent snowcovered area so as to compute the snowline elevation (see figure 4).

The snow band option tracks the accumulation and depletion of snow in each band, to determine which bands still have snow, and treats each elevation band as a separate watershed. Each band is treated as being snowfree or 100% snowcovered.

3.2 Input Balance

Moisture (rainfall and snowmelt) applied to a basin can either be absorbed by the soil mantle in the form of non-gravitational porosity storage or contribute to runoff. The primary function of the input balance section is to keep track of the soil moisture index and to compute the amount of runoff for each time step. The amount of runoff for each time step is computed by applying the prevailing soil moisture index and the computed moisture input, to an empirically derived relationship of soil moisture index (SMI) versus runoff percent (ROP). Usually, rainfall intensity (RI, in inches per hour) is included in the SMI-ROP relationship. Figure 5, illustrates a typical SMI-RI-ROP relationship. Within the model, soil moisture storage may be depleted by evapotranspiration. Evapotranspiration may be specified to the model and can also be made a function of the prevailing SMI conditions.

In the snow cover depletion method of computing moisture input, the user has the added option of using the same SMI-ROP

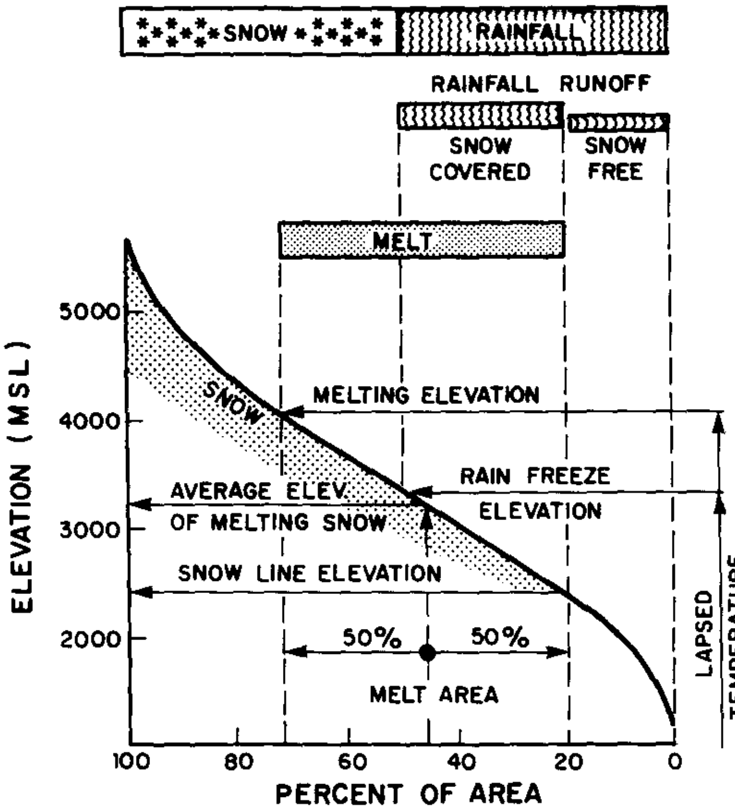


FIG. 3a

SPLIT WATERSHED OPTION OF THE SNOW COVER DEPLETION METHOD

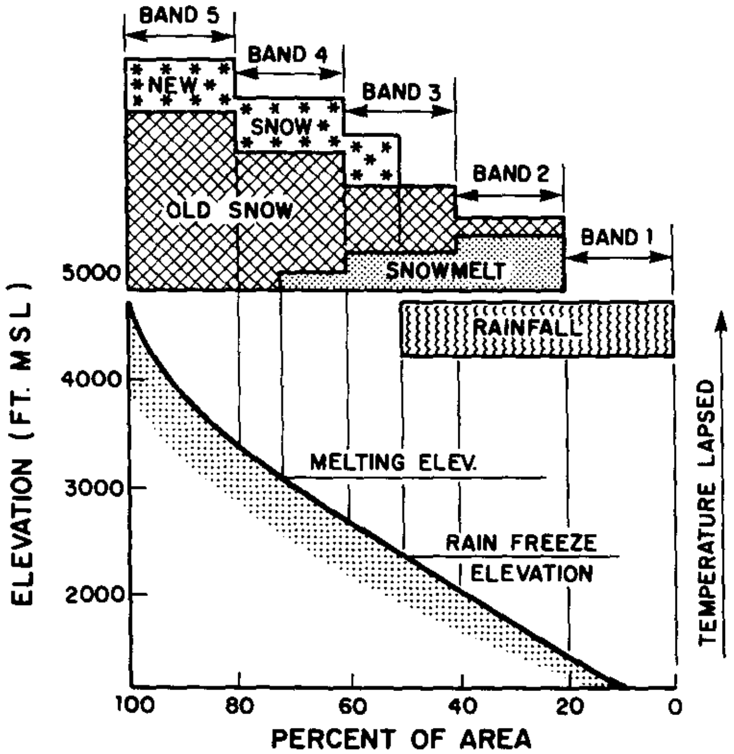
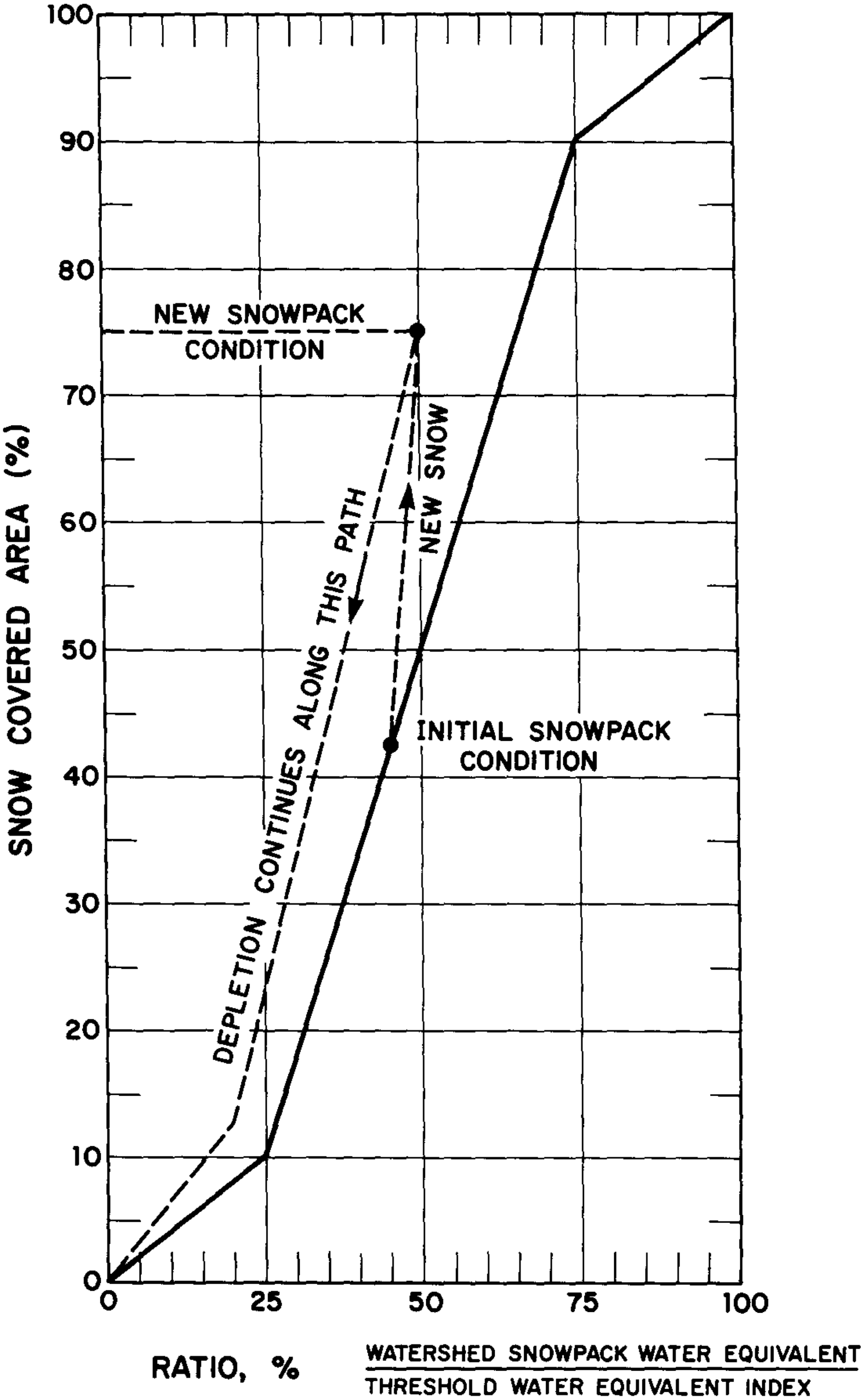


FIG. 3b

SNOW-ELEVATION BAND, EXAMPLE OF RAIN, SNOW AND MELT

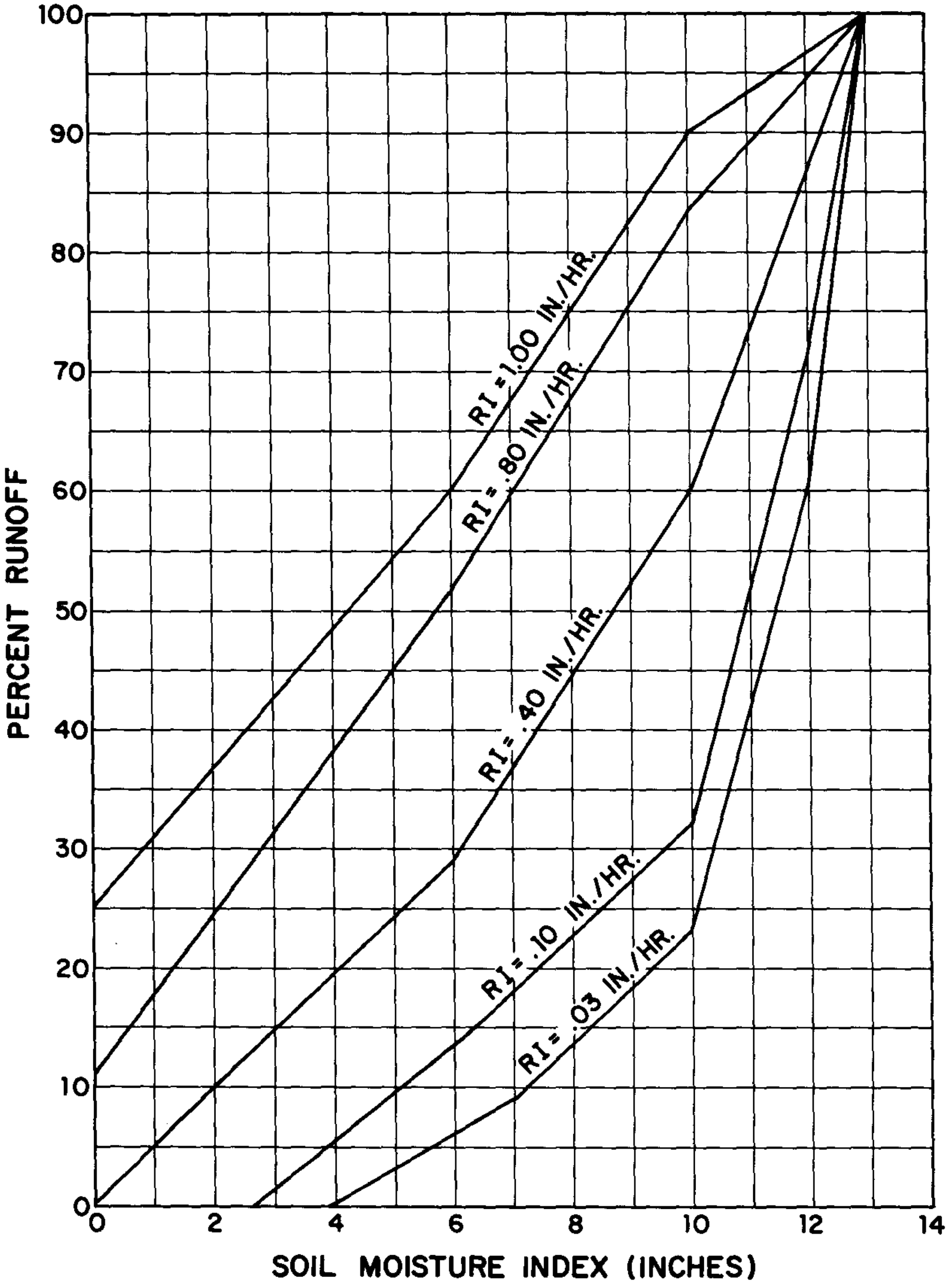
SSARR OPTIONS FOR DETERMINING MOISTURE INPUT

FIG. 3



SNOWPACK CONDITION ADJUSTMENT ROUTINE

FIG. 4



SMI - RI - ROP RELATIONSHIP

FIG. 5

relationship for both the snowcovered and snowfree area (single watershed model), or specifying a different SMI-ROP relationship for each (split watershed model).

3.3 Runoff Balance

The primary function of the runoff balance section is to distribute runoff into three runoff phases (more recent versions of SSARR allows for additional phases if desired). The three phases may be viewed as a slow, intermediate and fast response phases.

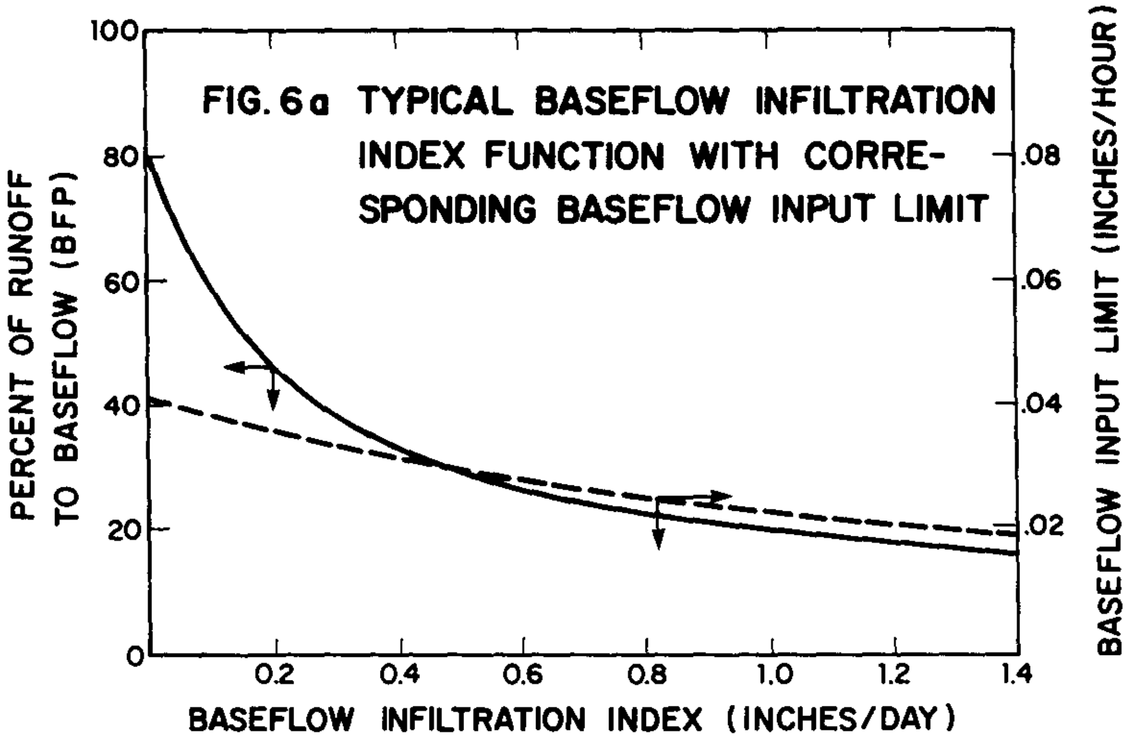
The portion of runoff going to baseflow is a function of the baseflow infiltration index (BII). It is computed by the SSARR model on the basis of a baseflow percent versus BII relationship (figure 6a). An upper limit to the baseflow input rate may also be specified to the model. The baseflow infiltration index, which may be thought of as an index of depression storage which holds runoff for deep percolation, increases with runoff and recovers with time.

Runoff, remaining after the abstraction of baseflow, is separated into a surface and subsurface component on the basis of a surface-subsurface separation relationship, illustrated in figure 6b.

3.4 Hydrograph Shaping

Each component of runoff (surface, subsurface, or baseflow) is computed as an input rate expressed in inches per period. Each period value is converted to the equivalent flow rate, in cubic feet per seconds, based on the drainage area and the length of the period in hours.

The transformation of the runoff, from a pulse to a continuous streamflow hydrograph, is accomplished by routing each phase through a specified number of cascading linear reservoirs ($_{\eta}$), each with a specified "time of storage" (TS) characteristic. Streamflow is computed by adding the final "outflow" from each of the three phase together. This concept is illustrated in figure 7a.



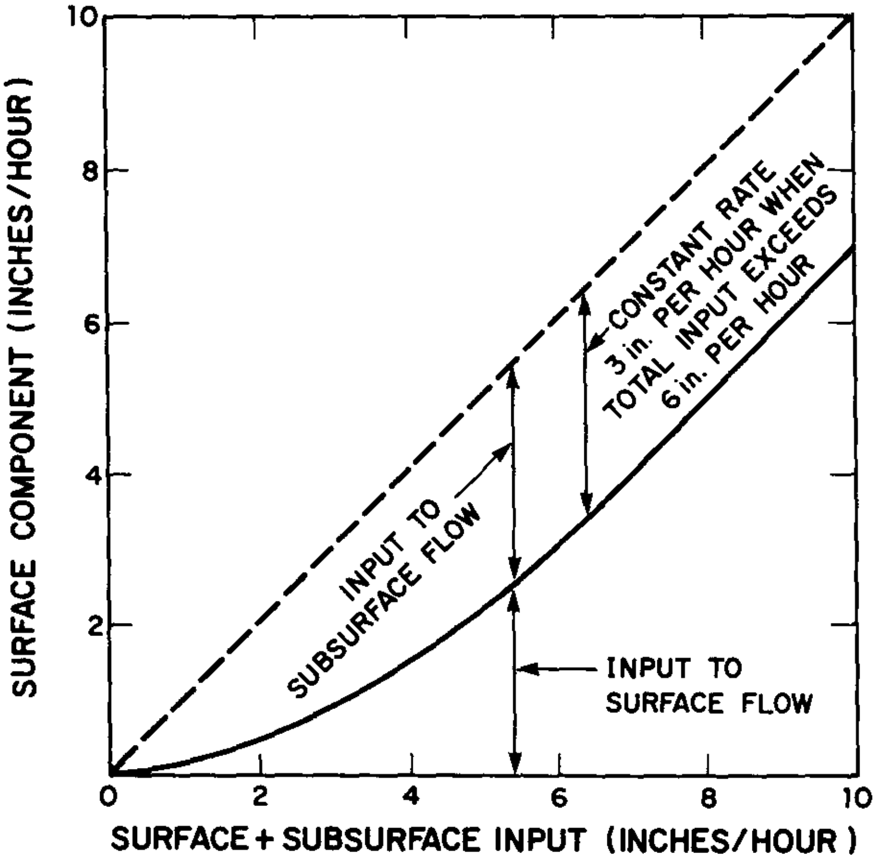


FIG. 6 b SURFACE - SUBSURFACE SEPARATION CURVE

For a desired 'time-to-peak', several combinations of "η" and "TS" will give the same time to peak but the magnitude of the peaks will vary; higher values of ' ' with corresponding lower values of "TS" will produce peakier hydrographs. An example of this point is illustrated in figure 7b.

The number of phases normally used to characterize the various components of runoff are: 3 to 5 for the surface component, 2 to 4 for the sub-surface and 2 or 3 for the baseflow.

The 'time-to-peak' for each of the runoff phases is generally adjusted based on significant runoff events. The peak flow for the surface phase is generally made to coincide with the peak flow of the recorded hydrograph. The peak flow from the sub-surface phase is made to coincide with the point of inflection. The peak for the baseflow is made to coincide with the second portion of the hydrograph recession curve. An example is illustrated in figure 8.

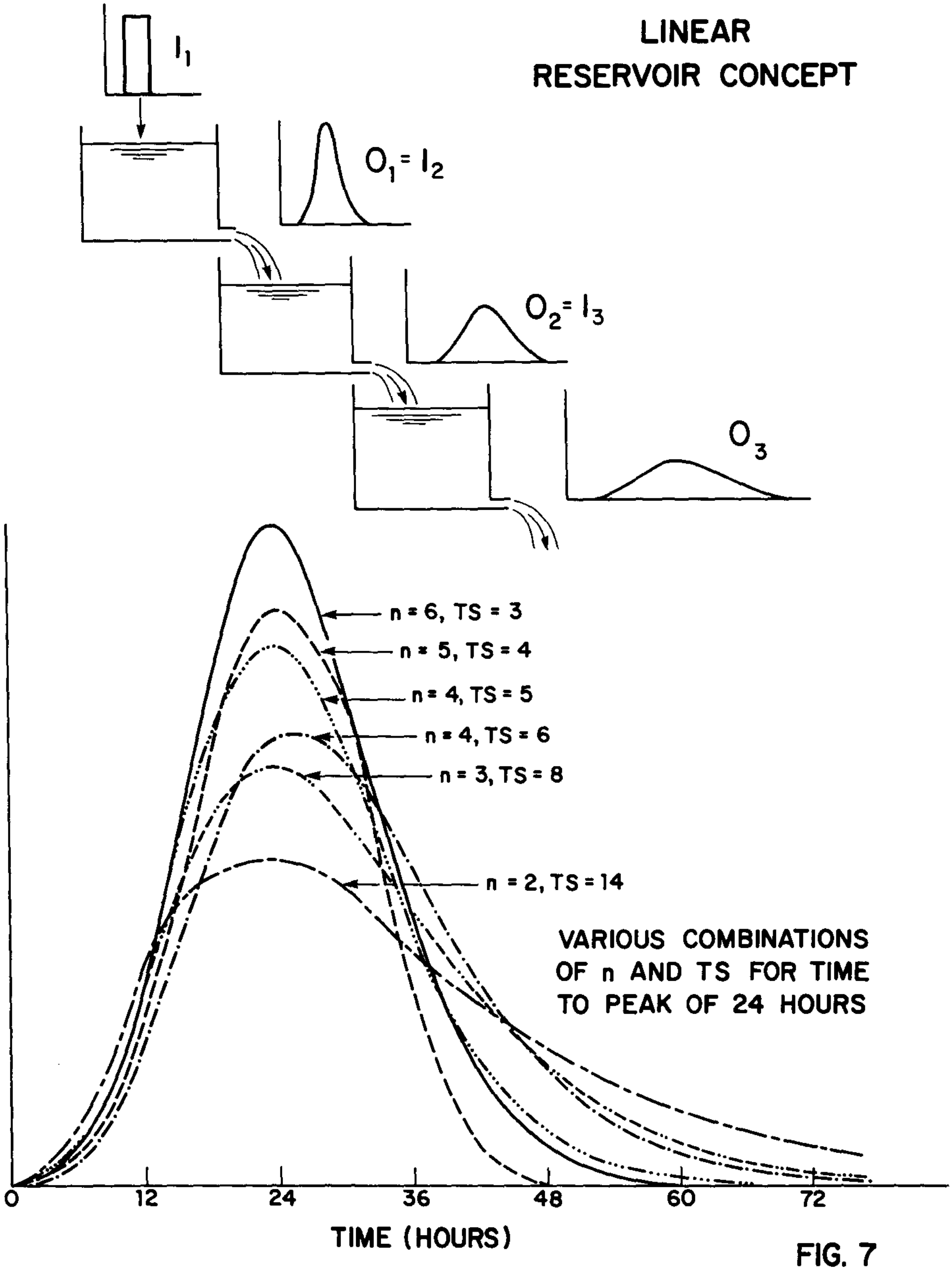
4.0 FUNDAMENTAL ROUTING METHOD

Routing through watershed, river system, and reservoir components of SSARR relies on the Law of Continuity in the storage equation:

$$I(t) = O(t) + ds/dt \qquad (1)$$

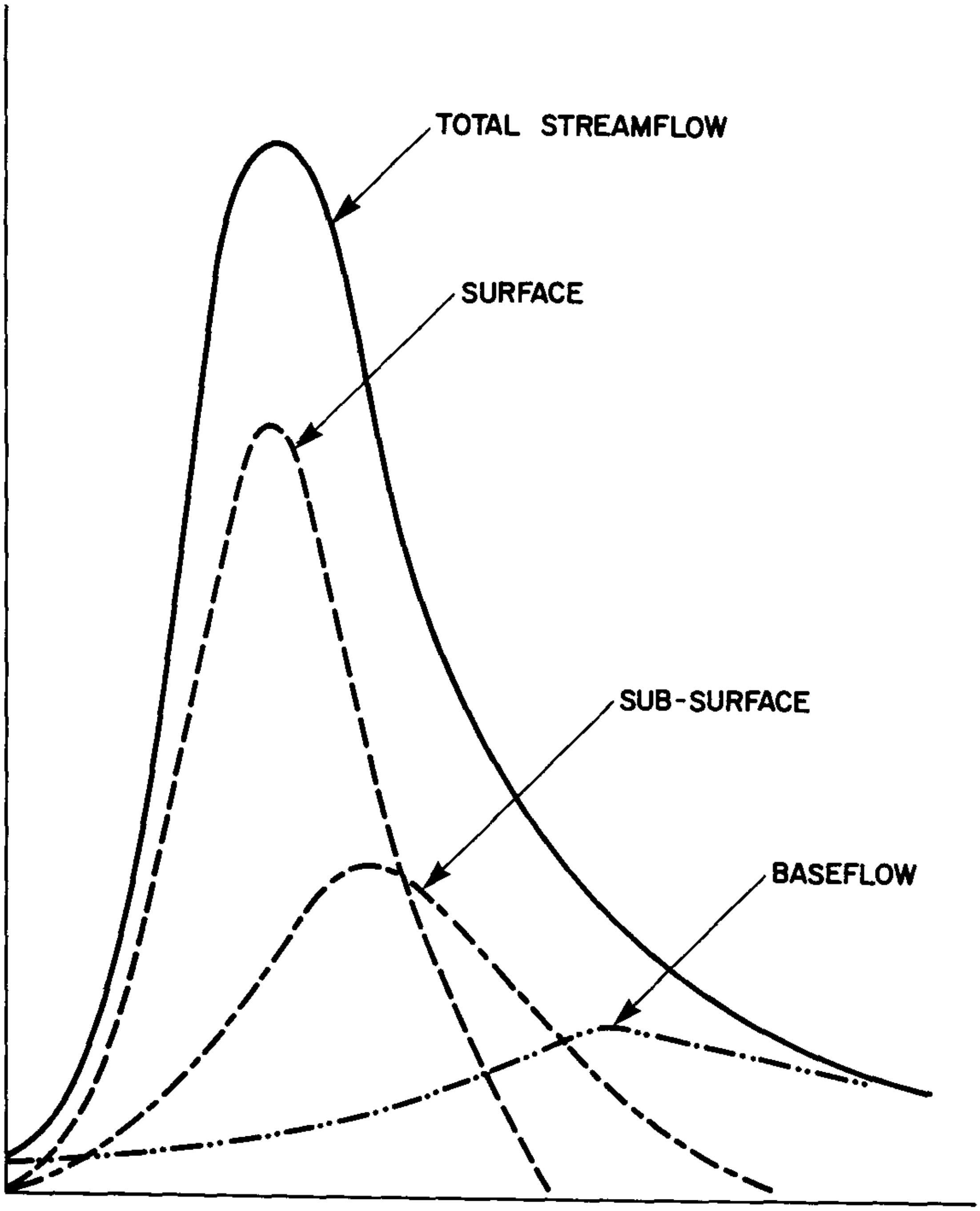
where

$I(t)$ is the instantaneous inflow at time "t"
$O(t)$ is the instantaneous outflow at time "t"
dS/dt is the rate of change of storage with time



LINEAR
RESERVOIR CONCEPT
I_1
$O_1 = I_2$
$O_2 = I_3$
O_3
n = 6, TS = 3
n = 5, TS = 4
n = 4, TS = 5
n = 4, TS = 6
n = 3, TS = 8
n = 2, TS = 14
VARIOUS COMBINATIONS
OF n AND TS FOR TIME
TO PEAK OF 24 HOURS
0
12
24
36
48
60
72
TIME (HOURS)

FIG. 7



WATERSHED ROUTING

FIG. 8

In natural lakes where outflow is a function of lake level (and therefore storage), lake storage is approximately proportional to rate of outflow, and a proportionality constant, "time of storage", factor "TS" can be defined as:

$$TS = S/O \qquad (2)$$

or

$$S = TS * O$$

Differentiating with respect to time yields:

$$dS/dt = TS * dO/dt \qquad (3)$$

Substituting this expression in the Continuity equation, the result is:

$$I(t) = O(t) = (TS * dO/dt)$$

or

$$dO/dt = [(I(t) - O(t)] /TS \qquad (4)$$

For finite time intervals, the following approximation is valid:

$$\frac{(O_2 - O_1}{t} = \frac{\left[\frac{(I_1 + I_2)}{2} - \frac{(O_1 + O_2)}{2}\right]}{TS} \qquad (5)$$

where

- O_2 is outflow at end of time interval
- O_1 is outflow at beginning of time interval
- t is the duration of the time interval
- I_1 is inflow at beginning of time interval
- I_2 is inflow at end of time interval
- TS is the "time of storage" characteristic of the "lake"

The expression can be rearranged as follows to solve for O_2:

$$O_2 = O_1 + \frac{t}{[TS + (t/2)]} (I_{avg} - O_1) \tag{6}$$

This equation permits the calculation of "outflow" at the end of the computational period, as long as the average "inflow" for the period, the "outflow" at the beginning of the period, and the "time of storage" characteristics of the "lake" are known.

In modelling, the calculated outflow at the end of one period becomes the outflow at the beginning of the next period. This method of establishing the time distribution of simulated streamflow is basic to SSARR. It is used in defining the watershed "unit hydrograph", as well as, in channel and reservoir routing.

The case of reservoir routing is simply that of a single lake, although a variable "time of storage" characteristic may be specified as a function of lake level.

4.1 Channel Routing

For channel routing, the river reach is divided up into a number of increments, the increments are then treated as a series of linear reservoirs. The routing equation is then solved for each increment, with the outflow for each increment being used as the inflow to the next downstream increment.

Channel routing can be accomplished by providing to the model either a routing equation, or a table which specifies a time of storage-discharge relationship.

Normally when flows are confined to the channel, the time of storage decreases as discharge increases, and it is convenient to express the relation by means of an equation: For such cases as overbank routing it is more convenient to provide a table of time of storage versus discharge.

The expression can be rearranged as follows to solve for O_2:

$$O_2 = O_1 + \frac{t}{[TS + (t/2)]} (I_{avg} - O_1) \qquad (6)$$

This equation permits the calculation of "outflow" at the end of the computational period, as long as the average "inflow" for the period, the "outflow" at the beginning of the period, and the "time of storage" characteristics of the "lake" are known.

In modelling, the calculated outflow at the end of one period becomes the outflow at the beginning of the next period. This method of establishing the time distribution of simulated streamflow is basic to SSARR. It is used in defining the watershed "unit hydrograph", as well as, in channel and reservoir routing.

The case of reservoir routing is simply that of a single lake, although a variable "time of storage" characteristic may be specified as a function of lake level.

4.1 Channel Routing

For channel routing, the river reach is divided up into a number of increments, the increments are then treated as a series of linear reservoirs. The routing equation is then solved for each increment, with the outflow for each increment being used as the inflow to the next downstream increment.

Channel routing can be accomplished by providing to the model either a routing equation, or a table which specifies a time of storage-discharge relationship.

Normally when flows are confined to the channel, the time of storage decreases as discharge increases, and it is convenient to express the relation by means of an equation: For such cases as overbank routing it is more convenient to provide a table of time of storage versus discharge.

HSPF MODEL

Purpose: The Hydrologic Simulation Program-FORTRAN, known as HSPF is a mathematical model developed under United States Environmental Protection Agency (EPA) sponsorship to simulate hydrologic and water quality processes in natural and man-made water systems. HSPF uses information such as the time series of rainfall, temperature, solar radiation along with parameters related to land use patterns, soil characteristics, and agricultural practices to simulate the processes that occur in a watershed. The initial output from the "lands phase" of an HSPF simulation is a time series of the quantity and quality of water transported over the land surface and through various soil zones down to the ground water aquifiers. Runoff flow rates, sediment loads, and the concentration of nutrients, pesticides, toxic chemicals can be predicted. The model then takes these results and information about the receiving water channels to simulate the water channel processes.

Background: HSPF is an extension and improvement of three previously developed models: 1) The EPA Agricultural Runoff Management Model (ARM), 2) The EPA Nonpoint Source Runoff Model (NPS), and 3) The Hydrologic Simulation Program (HSP, including HSP Quality), a privately- developed proprietary program. The EPA recognized several years ago that the continuous simulation approach contained in these models would be valuable in solving many complex water resource problems. Thus, a fairly large investment was devoted to developing a highly flexible non-proprietary FORTRAN program which contains the capabilities of these three models, plus many extensions. The result of this investment is HSPF.

Application and Use: HSPF is probably the most comprehensive and flexible model of watershed hydrology and water quality available today. The model is unusual in its ability to represent the hydrologic regimes of a wide variety of streams and rivers with reasonable accuracy. Thus, the potential applications and uses of the model are comparitively large including:

- Water Supply Forecasting
- Flood control planning and operations
- Hydropower studies
- River basin and watershed planning
- Storm drainage analyses
- Water quality planning and management
- Point and nonpoint source analyses
- Soil erosion and sediment transport studies
- Evaluation of urban and agricultural best management practices

Data Requirements: Simulation of summer runoff requires hourly precipitation and daily evaporation. The simulation of snow accumulation and melt requires four additional data series wind, air and dewpoint temperature and solar radiation in either a hourly or a six hour timestep. In Canada, wind, air and dewpoint temperature are usually only available from Atmosphere Environment Services (AES) mainline stations. Solar radiation is only available from a limited number of mainline stations.

Special Features: HSPF software is planned around a time-series management system operating on direct access principles. The simulation modules draw input from a Time Series Store (TSS) and are capable of writing output to it. Because these transfers require very few instructions from the user, problems with data handling are minimized.

Constructing an input deck for the HSPF model is made easier due to the modular design incorporated by the program. Comment cards can be freely added throughout the input deck making it more readable. Setting up a run requires merging modules and setting up the linkages between modules. A front-end program called Annie is now available that prompts the user for input and then constructs an input deck.

The model framework evolved from a top down approach emphasizing structured design. This structure allows a user to quickly identify a section of code performing a particular operation and change or extend the system with minimum disruption of the remaining code. For example, we found it relatively simple to modify the snowmelt code by pulling out the snow module from the code, make modifications to various subroutines and then relinking with the main model.

HSPF will run on a wide variety of computer systems. The model will run on IBM PC's, mini computers like VAX/VMS and large mainframe computers such as IBM, Control Data and HP3000 machines.

Limitations: Manitoba's experience with summer and spring simulations showed that HSPF's biggest limitation to more wide-spread use was the availabilty of meterological database that would adequately represent the watersheds studied. Simulation of snowmelt requires climate stations be relatively close to the watershed (within a radius of 50 km). Solar radiation, an important input into HSPF's energy balance snowmelt calculation is only recorded at one location in southern Manitoba. Other researchers have found when data has to be transferred to or synthesized for a watershed there is a loss in accuracy to the point that a degree day melt equation can be just as accurate.

At one point in the Manitoba experience, HSPF's inability to model the effects of frozen ground was a major limitation in simulating spring runoff. This led to a re-examination of methods used to estimate infiltration. Research on water infiltration during snowmelt, developed in Saskatchewan by Granger, Gray and Dyck was incorporated. Also there was a check of the computer code to see how the model's ice lens operated. These changes resulted in significant improvements in the simulation of spring runoff volumes.

With todays downsized workplaces in terms of manpower and study dollars, HSPF may be too expensive to use. It can take three to four man months to calibrate a basin for spring streamflow simulation, given that the user is reasonably proficient in using HSPF and has a good understanding of the hydrology of the basin. This time estimate can easily be doubled due to data errors and modelling errors.

One of the factors, that should be considered before deciding to use HSPF is what accuracy one might expect and is the accuracy gain worth the effort. Our experience in snowmelt modelling was that there was no gain on the three basins we studied. The simulated runoff volume averaged within 15% of the observed but ranged between 0 and 38%, while the the snowmelt peaks on average were within 20% but ranged between 6 and 32% of the observed. General guidelines on HSPF calibration suggest that this would be considered a "good" simulation.

September 15, 1988

HYDRODYNAMIC MODELLING

Introduction

Flows in both natural and man-made channels are usually unsteady. Normally, in flood flow analysis, an assumption of steady state conditions for simplifying the computational procedures produces conservative results from a flood-stage point of view (Perks et al, 1983). The conservatively higher stage values arise from a steady state analysis because of the underlying assumption that given a particular recurrence interval discharge, the flood wave is of infinite length thereby filling all available channel storage and yielding no wedge storage. However, in certain flood studies, a steady-state analysis can lead to overly conservative results and even to erroneous low estimates in particular reaches of complex hydraulic networks. Examples of such studies would be: a dendritic river system where the flows from the tributaries cause backwater effects in the main channel, or the tributaries themselves are even subject to reversals of flow; extensive embayment storage in channel reaches; and estuary reaches experiencing tidal effects. Hydraulic computations for these cases require a more mathematically rigorous approach than the traditional techniques such as the steady-state step method, channel storage routing, and Muskingum routing. The hydrodynamic modelling approach is recommended for these special cases.

The difficulty in formulating the solution of this problem increases with the degree of complexity of the general flow pattern (one, two or three dimensional). For flood study purposes, this discussion is limited to the solution of one-dimensional (1-D) flows.

Saint Venant Equations

Flood waves may generally be considered as 1-D gradually varied unsteady flows. The analytical foundation for the study of this type of flow is based on the work of Saint Venant, 1871. In order to obtain theoretical equations two principles are considered: the conservation of mass, or continuity (difference between inflow and outflow equals the time rate of change of storage) and the conservation of momentum (Newton's second law, stating that the sum of forces acting on a small element of water is equal to the net time rate of momentum leaving the element plus the time rate of accumulation of momentum in the element). Certain flow conditions must also be assumed:

a) the flow is 1-D (velocity is uniform over the cross section and water level across the section is assumed horizontal);

b) the streamline curvature is small, and vertical accelerations are negligible (pressure is hydrostatic);

c) the resistance equations developed for steady flows are applicable for unsteady flows (such as the Manning's or Chézy equation);

d) the average channel bed slope is small;

e) the flow is incompressible and its density is homogeneous;

f) the shear stresses due to wind and changes in atmosphere pressure are negligible.

Applying the laws of conservation of mass and momentum to a non-prismatic channel (Figure 1), the following two equations are obtained:

Conservation of mass (continuity)

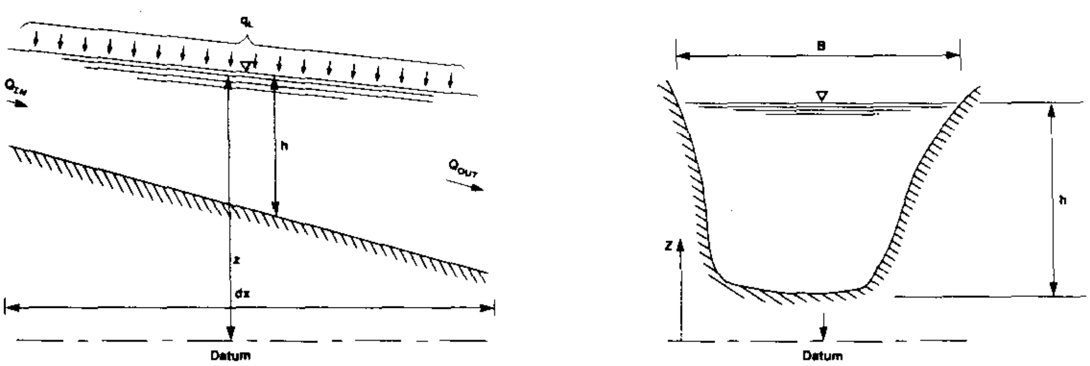
$$B\frac{\partial z}{\partial t} + Bv\frac{\partial z}{\partial x} + A\frac{\partial v}{\partial x} + v\left(\frac{\partial A}{\partial x}\right)_z = q_l \qquad .....(1)$$

Conservation of momentum

$$\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x} + g\frac{\partial h}{\partial x} + g\,(S_f - S_o) = (v_x - v)\,q_l/A \qquad .....(2)$$

Where:
- A = cross-sectional area of the channel
- B = channel top width
- g = acceleration due to gravity
- h = depth of flow
- q_l = lateral inflow per unit width
- S_f = friction slope
- S_o = channel bottom slope
- t = time reference coordinate
- v = mean flow velocity
- v_x = velocity component of lateral inflow, in the direction of the longitudinal axis
- x = space coordinate, in the direction of the longitudinal axis
- z = water surface elevation
- $\left(\frac{\partial A}{\partial x}\right)_z$ = the variation of the cross section area in the direction of the longitudinal axis, for a constant water elevation, to account for the non-prismatic characteristics of the sections

Laws of conservation of mass and momentum applied to non-prismatic channels



(a) PROFILE

(b) CROSS-SECTION

REACH PROFILE AND CROSS-SECTION

Figure 1

The preceding expressions differ slightly from the original Saint Venant equations because lateral inflow was not considered in the earliest formulation. The solution of these equations has become attainable for practical applications during the past two decades with the advent of high-speed computers and the accompanying advanced numerical methods.

Model Availability and Applications

A number of hydrodynamic models are available to analyse the propagation of a flood wave. A model known as the Dynamic Wave Operational (DWOPER) model (Fread, 1978) was developed by the U.S. National Weather Service during the 1970s, and had been continuously updated in order to extend its applicability. It has been widely used in the United States (Perks et al, 1983). This model has, nevertheless, a severe limitation because it handles only first order tributaries (Peters, 1983 and Penn. State University, 1984), and as such, no second or higher order tributaries can be directly considered. In other words, it cannot represent exactly tributaries flowing into other tributaries.

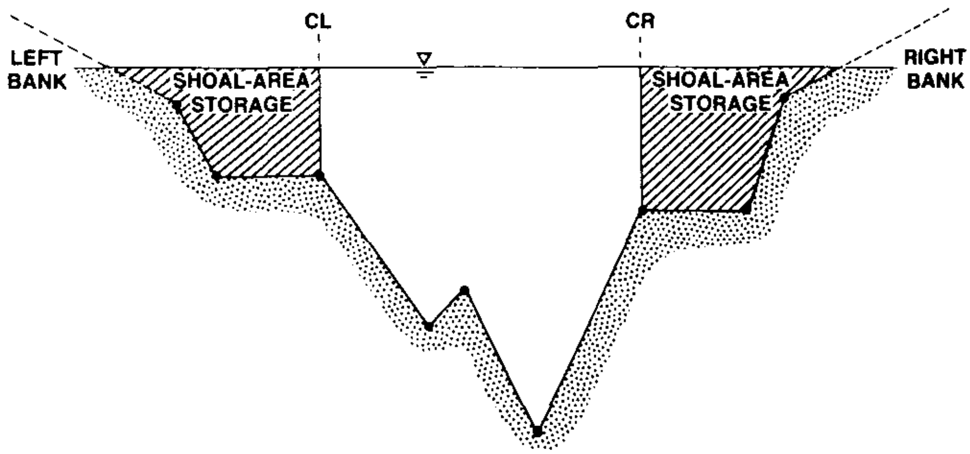
In the early 1980s, the National Weather Service (NWS) of the United States also developed a more complete model, called FLDWAV (Penn. State University, 1984). This model can handle higher order tributaries, and also more complex internal boundaries such as a time dependent dam failure. Up to now few papers have been published which describe the effectiveness or limitations of this latest model.

It is not intended here, to present an exhaustive discussion about the advantages and limitations of the various presently available hydrodynamic models. Instead, a particular model which has been used for several Canadian applications is discussed in detail to give the reader more insight into the use of hydrodynamic modelling techniques. This model, which is referred to as the Environment Canada One-Dimensional Hydrodynamic Model, or simply the ONE-D model, is currently recommended because of its flexibility and demonstrated reliability. The model and its appropriate subroutines are documented and supported by Environment Canada for Canadian users.

The ONE-D model is a FORTRAN coded computer program. Its modular structure permits blocks (subroutines) to be easily added to the main program in order to extend its capabilities. The following features can be presently treated by the model:

- irregular cross-sectional geometry;
- linear interpolation and extrapolation of cross-sections between input cross-sections;
- two types of physical off-channel storage (Figure 2);
  - (i) shoal-area storage, used to differentiate the passive part of the cross-section with the conveyance area. In this case, the core (or conveyance) area is defined by the left side limit (CL) and the right side limit (CR);
  - (ii) embayment storage, used to represent storage in small and shallow bays or lakes;

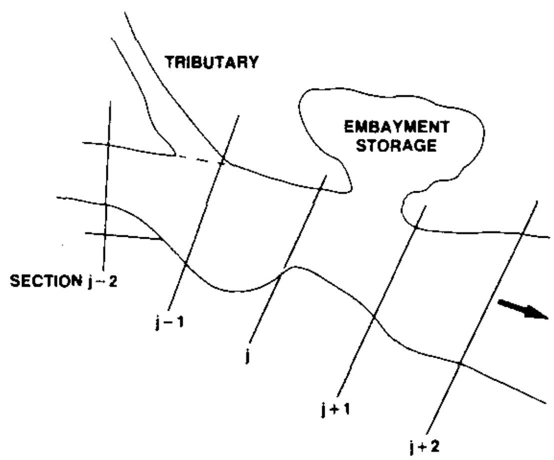
Physical Off-channel Storage



(a) SHOAL-AREA STORAGE

Figure 2(a)

Physical Off-channel Storage



(b) EMBAYMENT STORAGE

Figure 2(b)

- Manning's coefficient of roughness, which may vary with water surface elevation, and from cross-section to cross-section;
- capability to compute steady-state conditions that can be used as initial conditions;
- time-dependent lateral inflow or outflow;
- space increment that can vary from one reach to another;
- any number of reaches connected to a node (the present model setting is for a maximum of 5 reaches per node, but this can be increased to any value by changing the dimensions of arrays and matrices);

- friction variation caused by full or partial ice cover, and by ice cover growth or decay;

- reduction in cross-sectional area caused by ice thickness;

- flow overtopping roads, dykes or embankments adjacent to a reach, and/or through culverts or aboiteaux;

- delineation of flood channels in the floodplain;

- indirect method to respond to channels that occasionally dry up or are subject to very low flows;

- internal boundary conditions, such as small rapids or falls, bridges, weirs, gates and spillways, represented by a relationship or a family of curves between water elevations (upstream and downstream of the structure) and discharges;

- external boundary conditions such as discharge or water surface elevation hydrographs. In the case of downstream boundaries, a relationship between water levels and discharges representing natural controls or hydraulic structures can be used. The model also has the capability to compute an approximate stage-discharge relationship for downstream boundaries, according to the physical characteristics of the reach in the vicinity of the boundary.

In order to apply the model economically to different hydraulic networks, it is necessary to regenerate the dimensions of arrays and matrices in the program. These dimensions are related to the size and complexity of the network. Generating new model sizes is a straight forward task and results in the most efficient representation of the network under study in terms of allocated computer memory space.

General Theory of the ONE-D Model

The governing equations (1 and 2) can be rewritten in terms of discharge and Froude number. It can also be assumed that the lateral inflow enters the system with negligible momentum in the direction of the main flow, or in other words, with a small velocity component v_x. Expressing the friction slope in terms of the Chézy (or Manning) equation, the governing equations are written as:

Conservation of mass (continuity)

$$B\frac{\partial z}{\partial t} + \frac{\partial Q}{\partial x} = q_l \qquad .....(3)$$

Conservation of momentum

$$\left(\frac{\partial Q}{\partial t} + 2v\frac{\partial Q}{\partial x}\right)\frac{1}{Ag} = (Fr^2 - 1)\,\frac{\partial z}{\partial x} + \frac{Fr^2}{B}\left(\frac{\partial A}{\partial x}\right)_z - Q\frac{|Q|}{K^2} \qquad .....(4)$$

Where: C_z = Chézy flow resistance factor

Where: $Fr = (\frac{v^2}{g}\frac{B}{A})^{0.5}$

$$K = AC_z\sqrt{R}$$
$$= \frac{AR^{2/3}}{n} \quad \text{(S.I. units)}$$
$$= 1.486\,\frac{AR^{2/3}}{n} \quad \text{(Imperial units)}$$

n = Manning's roughness coefficient

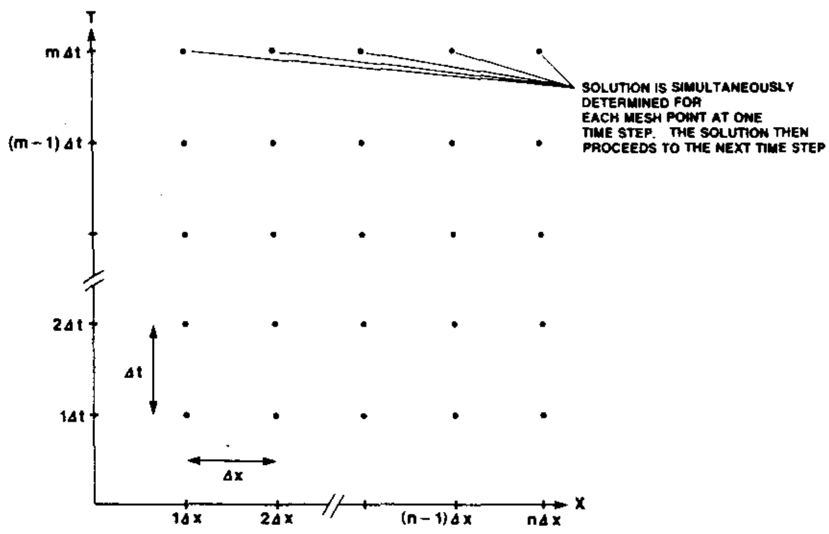
Q = river discharge

R = hydraulic radius

The solution of equations 3 and 4 for practical applications can be achieved through numerical techniques, such as the method of characteristics, the finite element method and the finite difference method. Although the method of characteristics used to be very popular in the 1960s, it is not very efficient for the analysis of fluid transients in open channels, mostly because of the non-prismatic nature of the cross-sections. It is quite accurate, but is difficult to program. The finite element method requires the selection of small time and space increments to describe the system properly, and is not as popular and well developed as the finite difference method. The ONE-D model solves the governing equations by using a finite difference technique.

In a finite difference technique, the differential equations are approximated by discretised equations, also called finite differences. The hydraulic conditions are expressed in terms of time and space increments. For one time step at the time, the solution of the equations is found at each space increment in the system. The solution then proceeds to the next time step (figure 3).

Discretisation of Solution



DISCRETISATION OF SOLUTION

Figure 3

A variety of approximations, called finite differences schemes, can be used to describe the continuous functions. It is far beyond the scope of this paper to review and discuss all the schemes that were developed by different authors (for some examples, see Gunaratnam and Perkins, 1970; Fread, 1976; Mahmood and Yevjevich, 1975). A basic knowledge of the types of scheme and their numerical properties is, however, essential in order to understand the difference between existing hydrodynamic models. The schemes are divided mainly in two different categories: the explicit schemes and the implicit schemes. With the explicit schemes, the unknown hydraulic conditions at the end of a time period and at a certain location are expressed in terms of the known conditions at the beginning of the same time period. In the implicit schemes, the unknowns at the end of a time period and at a specific location are expressed in terms of known conditions at the beginning of the same time period but also in terms of other unknowns at the end of that time period. In the case of the implicit schemes, the solution for hydraulic conditions at the end of a time period must be found for all space increments simultaneously. This requires the simultaneous solution of a system of equations.

When a finite difference scheme is used, numerical properties such as stability, consistency and convergence must be analysed. A scheme is stable when the errors introduced by truncation and round-off do not become large enough to obscure and destroy the solution. The error growth for a stable scheme is bounded and remains small relative to the solution. A scheme is consistent if the discrete equations become the continuous equations, as the time and space increments approach zero. It is convergent if the solution of the discrete equations approaches the solution of the continuous equations as the time and space increments approach zero. This definition of the convergence of a scheme originates from mathematical principles. Since the analytical solution of the governing equations is not known for practical applications, it is not possible from an engineering point of view to compare both analytical and discrete solutions to verify the convergence of the scheme. However, some aspects of the solution can be studied. A scheme will be said to be convergent when the solution is conservative and non-dissipative. It is conservative if it reproduces the conservative property of the governing equations (continuity). It is non-dissipative if the attenuation and celerity of the waves obtained from the discrete solution are similar to the values obtained from the solution of the continuous equations. Explicit schemes, although simpler to apply than implicit schemes, are less appropriate for simulation of long term unsteady flow phenomena such as flood waves in rivers. Because of stability considerations, the explicit schemes are restricted to very small computational time steps. The time step is limited by the Courant-Levy-Friedrich criteria, or by some form of this criteria:

Courant stability criteria

$$\Delta t \leq \frac{\Delta x}{v + |c|} \qquad .....(5)$$

where: $|c|$ = absolute value of the celerity of the wave

Δt = time increment

v = mean flow velocity

Δx = space increment

This criteria is also known as the Courant criteria. In general terms, the observation of the Courant criteria can be understood as a limitation of the computational time period such that a wave will never go through a space increment without being considered in the computation process. The violation of the criteria would mean that a wave enters, goes across and leaves a space increment during the same period of time. The implicit schemes have the advantage to be unconditionally stable, with no restriction on the time step. However, in this case, other considerations such as the convergence of the solution may require some limitations on the computational time period, but those are less severe than the Courant criteria.

The finite difference scheme used in the ONE-D model is an implicit scheme, and is then unconditionally stable. It was developed at the MIT by D.J. Gunaratnam and F.E. Perkins, 1970. The scheme was obtained by applying a weighed residual method of optimization to a simplified linearized version of the governing equations, in order to minimize the difference between the continuous differential equations and the discrete approximations (consistent scheme). Appropriate local and temporal adjustments were used in order to account for the non-linear characteristics of the governing equations. The convergence of the scheme, as defined from an engineering point of view, is found to be a complex function of the Froude number, the Courant condition, the friction and the waves characteristics. The following criteria were proposed for the convergence of the scheme:

Convergence criteria

$$\frac{\lambda}{\Delta x} \geq 100 \qquad .....(6)$$

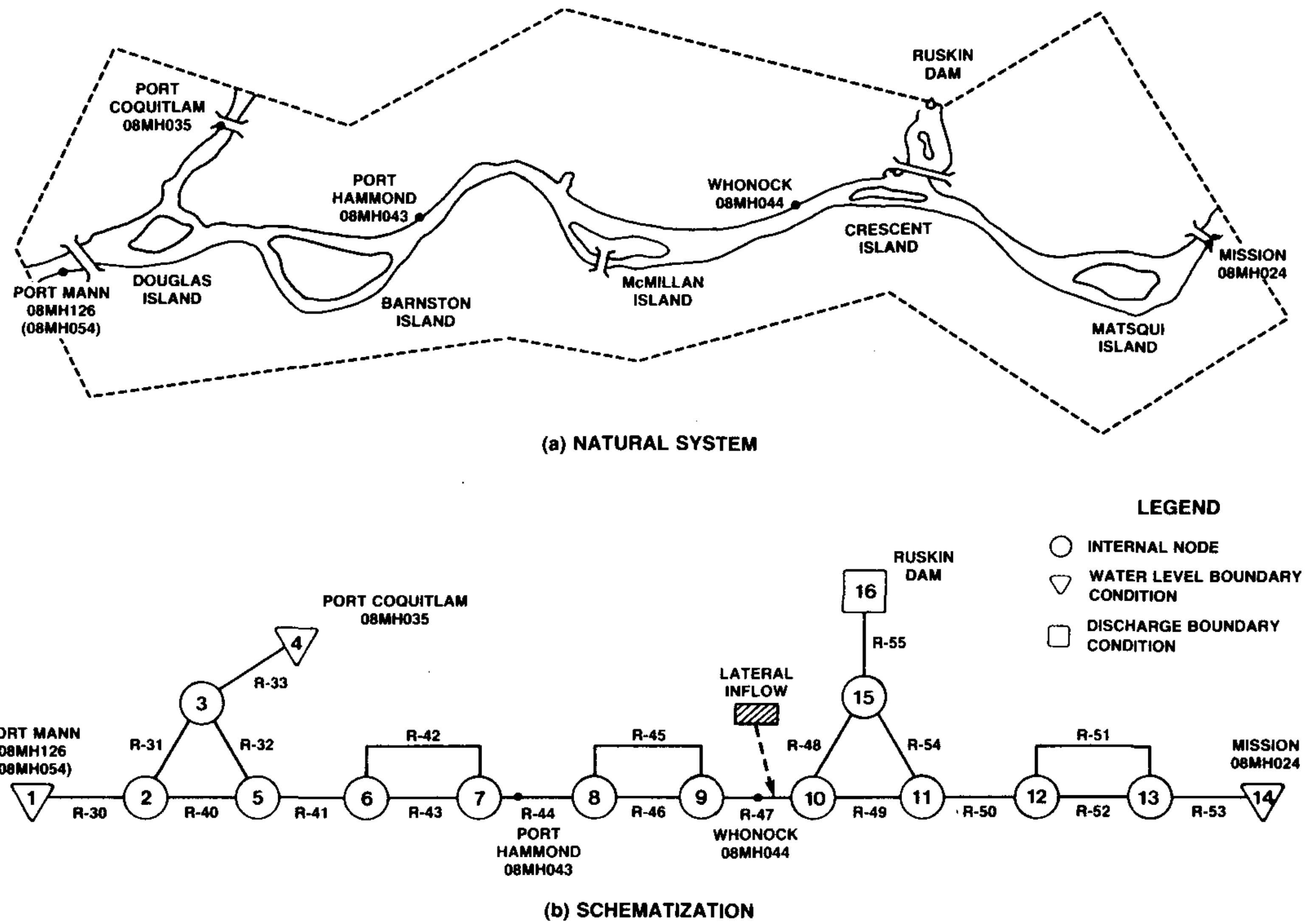
$$\Delta t \leq 5.5 \ \frac{\Delta x}{v + |c|} \qquad .....(7)$$

Where: λ = wave length

The condition expressed by equation 6 shows that the scheme gives an accurate prediction even for the discretisation of waves with relatively high frequencies. A sensitivity analysis has demonstrated that the condition expressed by equation 7 is very conservative for a situation predominated by low frequency waves such as natural flood waves, and the factor 5.5 can be increased depending on the specific characteristics of the system being considered.

Data Input to the ONE-D Model

In the ONE-D model, a river system or delta is represented by a network of reaches inter-connected by nodes (figure 4). The network topology, or schematization, must be defined by the user in order to be representative of the natural system, but with the minimum number of reaches and nodes (Water Planning and Management Branch, 1982).



LOWER FRASER RIVER NATURAL SYSTEM AND SCHEMATIZATION

Figure 4

Each reach is described by a length, one or many Manning's coefficients of roughness, an estimate of the space increment and a table of hydraulic parameters (total top width, core top width, core area, wetted perimeter, and total area, at different water depth or elevation) for each input cross-section. The approximate space increment is used by the model to compute the exact value Δx for the reach, according to the following relationship:

Space increment criteria

$$\Delta x = T_l \,/\, \mathrm{Int}\left(\frac{T_l}{\Delta x_*} + 1\right) \qquad .....(8)$$

Where: Δx = exact space increment

T_l = total length of the reach

Δx_* = approximate space increment

Int = integer value

One constraining feature of the model is that a minimum of 4 space increments (5 mesh points) must be used for each reach. It is recommended to always have an even number of space increments (odd number of mesh points), in order to limit the transfer from reach to reach of numerical solution differences caused by input errors or internal boundary calculations.

The hydraulic parameter tables can be manually generated. However, front-end computer programs such as STAVE, COORD1, COORD2 (Water Planning and Management Branch, Aug 1984), and BLOCKCOORD (Water Planning and Management Branch, June 1984) have been written in order to generate these tables directly from input cross-sectional data. These programs allow the data to be read from processed hydrographic charts, digitized survey field sheets, topographic maps and aerial survey databases.

Topographic data must consist of high quality vertical control free from unspecified datum shifts or surveying blunders. Inertial Survey System (ISS) data have been found to be satisfactory for Mackenzie Delta studies. These data were found to have a root mean square error (rmse) of $\pm.22$ metre for elevation differences between points 10 km apart and a rmse of $\pm.5$ metre for absolute values. Distances between surveyed sections along the reaches can be scaled from aerial photos or large scale maps with no appreciable loss in accuracy in hydraulic computations.

Initial and external boundary conditions must be specified in order to run the model. The external boundary conditions represent the hydraulic behaviour at the extremities (upstream, downstream) of the network, during the simulation period. They are represented by elevations or discharge observations and measurements, and must fulfill the continuity flow requirements. In order to begin a simulation, it is also necessary to estimate the initial conditions (discharge and water level) throughout the entire network, at each input cross-section. In the absence of the more exact knowledge of these conditions, approximate values can be given. However, if they are incompatible with the governing equations, waves will be generated. These disturbances may mask for some time the actual solution, or may even force the run to abort. To avoid this difficulty, it is suggested to start the simulation at a time where the network is close to a steady-state condition. Prior to the simulation, the model can be run for a sufficiently long period of time using the approximate initial conditions to establish proper start-up steady-state conditions while continuing to hold boundary values constant during the "warm-up" period. The hydraulic values obtained at the end of this preliminary simulation are then compatible with the governing equations, and can be used without any difficulty as initial conditions for the transient flow simulation.

Description of ONE-D Model Output

Three different files are created during a simulation by the ONE-D model:

The first file is a printout of the results as requested by the user. These results can include: water level and discharge hydrographs that can be produced at any computational mesh point in the network; water level profiles that can be obtained in any reach and for any time (the time must correspond to an integer number of computational time steps).

A second file, known as the TAPE10 file, includes the water levels and discharges at each computational mesh point and for each time period. This file can be saved to be later used to provide additional information that was not requested in the printed output. For instance this file can be used as an input file for a plot program or to produce daily averages at a specified location.

The last file created by a computer run of the model is a TAPE17 start-up file. It includes the water levels and discharges at each mesh point for the last time-step calculation. This file is used as input initial conditions for the following simulation period.

Special Applications

Figure 5 is the schematic diagram of the network for a flood study conducted with the ONE-D model in the Truro region of Nova Scotia. The North and Salmon rivers, during storm events, carry flood discharges down steep gradients to their confluence in the city of Truro where there is a tendency for high tidal cycles on the Salmon River estuary to occur simultaneously with the storms. There are dykes with aboiteaux along the river which cannot accommodate entirely, 100-year and 20-year storm events. Consequently, simulations of these events must jointly consider the storm hydrographs and tidal cycles; simulate flow through culverts, aboiteaux and over dykes (subroutine "BREACH"); and pass flow through defined flood channels from overtopping points in the river network to downstream points where these flood overflows eventually rejoin the regular river network.

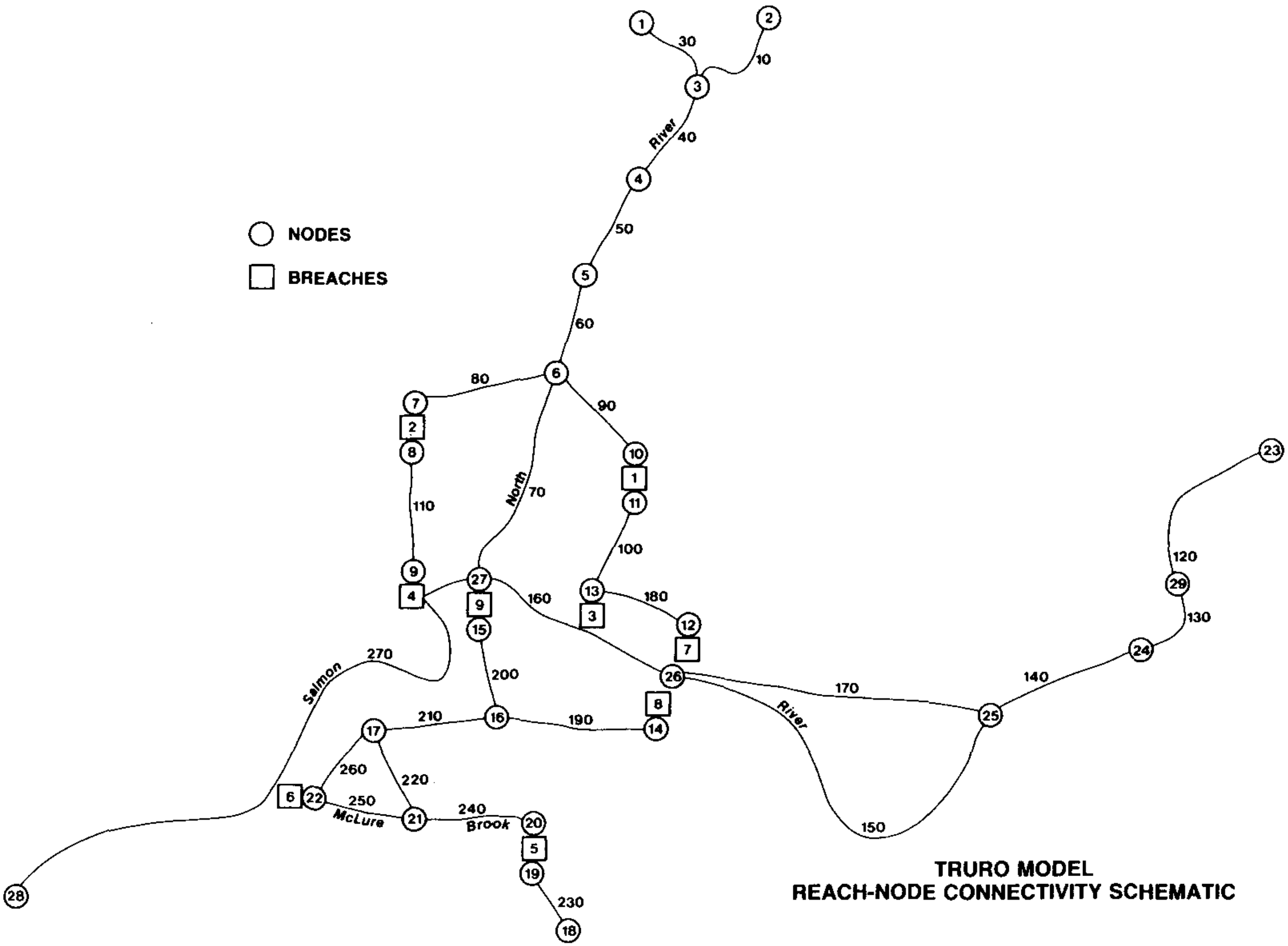
The ONE-D model was also applied to a flood study in the region of Montreal. The problem consisted of highly regulated flows of the St. Lawrence River meeting the partially regulated flows of the Ottawa River in the complex hydraulic network in the Montreal region. Since there is little storage capacity in the Montreal region, a steady-state analysis with the model was performed to determine flood levels for the design of dykes and for establishing the flood level contours.

The ONE-D model has also been used extensively to study the complex hydraulic network of the Peace-Athabasca Delta (Farley and Cheng, 1985). The model was applied: to determine the effects of flow regulation, both upstream and downstream of the delta; to evaluate the mitigating effects of the two overflow-notched weirs constructed in the delta in 1976; and to predict the effects of a proposed control structure on one of the major channels within the delta. Simulations over a 22-year period were conducted in this study as opposed to the normally short simulation periods not exceeding several days required for flood studies.

The ONE-D model is employed operationally on the lower Fraser River (figure 4) for the simulation of flow in a tidal affected region where flow reversals temporarily occur in some reaches (Water Planning and Management Branch, 1983). The computed discharges as a function of time are subsequently used in the suspended sediment loading calculations.

The model is currently being employed for studying the effects of dyke and road construction along the Red River, south of Winnipeg. Culvert and over-dyke flow is also simulated. Present plans call for the model being used in a forecasting mode for the Winnipeg region commencing in 1986. In this case, the ONE-D model will be coupled to a watershed forecasting model.

The ONE-D model is also being adapted to the Mackenzie Delta for assessing the impacts of projected upstream regulation.



TRURO MODEL
REACH-NODE CONNECTIVITY SCHEMATIC

Salmon and North River Model

Figure 5

List of References

Farley, D.W., Cheng, H., May, 1985, *Hydraulic Impact on Peace-Athabasca Delta of Flow Regulation on Peace and Slave Rivers*, Proceedings of National Symposium on Water, Rawson Academy of Science.

Fread, D.L., April 1978. *DWOPER-Operational Dynamic Wave Model*, U.S. National Weather Service.

Fread, D.L., December 1976. *Theoretical Development of Implicit Dynamic Routing Model*, Hydrologic Research Laboratory, Dynamic Routing Seminar, U.S. National Weather Service.

Gunaratnam D.J.; Perkins, F.E., July, 1970. *Numerical Solution for Unsteady Flows in Open Channels*, Report No. 127, Hydrodynamics Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Prepared Under 01-0001-1988 of Water Resources Research, U.S. Department of the Interior.

Mahmood, K., and V. Yevjevich, 1975. *Unsteady Flow in Open Channels*, Vol. 1, Water Resources Publications, Library of Congress Catalog Card Number 75-9251: 484 p.

Pennsylvania State University, February 28-March 4, 1983. *Unsteady Flow in Open Channels* Course notes : Channel Routing, January 1983, D.L.Fread.

Perks, A.R.; Moin, S.; Taylor, S., June 2,3, 1983. *Application of Dynamic Modelling to Flood Hazard Mapping Projects*, 6th Conference, C.S.C.E., Ottawa.

Peters, J.C., December 1983, *Flood Routing Through a Flat, Complex Flood Plain using a One-Dimensional unsteady Flow Computer Program*, Water Resources Bulletin, American Water Resources Association, Vol. 19, No. 6, p. 913-920.

Saint Venant, Barré de, 1871. *Théorie du mouvement non-permanent des eaux avec application aux crues des rivières et à l'introduction des marées dans leur lit*, French Academy of Sciences: p. 148-154 and 237-240.

Water Planning and Management Branch, July 1982, *Environment Canada One-Dimensional Hydrodynamic Model Computer Manual*, Inland Waters Directorate, Environment Canada.

Water Planning and Management Branch, June 1983. *The Application of a One-Dimensional Hydrodynamic Model to the Lower Fraser River-Fraser River Model Study*, Inland Waters Directorate, Environment Canada.

Water Planning and Management Branch, June 1984, *BLOCKCOORD Program*, Inland Waters Directorate, Environment Canada.

Water Planning and Management Branch, August 1984, *COORD1 and COORD2 Computer Manual*-Environment Canada One-Dimensional Hydrodynamic Model Data Preparation Programs, Inland Waters Directorate, Environment Canada.

Water Planning and Management Branch, August 1976, *Summary Report, Ottawa River Regulation.* Inland Waters Directorate, Environment Canada, microfiche file WPM4 4036.

HEC-2 Water Surface Profile Model

by: L. H. Wiens
Environment Canada
September, 1988

Introduction

The model was developed by Bill Eichert for the U.S. Army Corps of Engineers in 1964. Since its development it has undergone almost continuous modification to make it the valuable water resources tool used today. At each step of its evolution features have been added which increase the program's flexibility enabling it to be applied to a wide variety of water surface profile problems. Most recently modifications have been made in the model making it possible to calculate water surface profiles under the influence of ice.

Purpose

The program computes and plots the water surface profile for a river channel of any cross section for either subcritical or supercritical flow, provided that the hydraulic conditions of steady, gradually varied flow are maintained. The effects of various hydraulic structures such as bridges, culverts, weirs, dykes, levees and dams may be included in the computation.

Application

Primarily the model is used for calculating water surface profiles for floods of various frequencies under both natural and modified conditions. The results are used to map the extent of flooding and to design flood mitigation alternatives such as dykes, floodways, and diversions. The versatility of analysis and application has progressed to an almost unlimited degree respecting hydraulic analysis of open channel flow.

Computational Procedure

The reach to be profiled is divided into a series of subreaches by the establishment of cross sections at points of hydraulic change and points of interest as a function of the study being conducted. Computation proceeds, from cross section to cross section, in an upstream direction for subcritical flow and downstream for supercritical flow. The computational approach employed is based on the standard step method for gradually varied, nonuniform, one-dimensional flow, applicable, to both subcritical and supercritical flow conditions.

In the case of subcritical flow the computation begins at the downstream cross section with a given water surface elevation either known or assigned to it. The water surface at the next upstream cross section is computed by a progression of iterations incorporating the Bernoulli and Manning formulae. The iterations proceed until a balanced condition is reached in the one-dimensional energy equation at the two cross sections.

A test is made at the end of each computation to determine if subcritical flow might have entered the supercritical flow regime.

Supercritical flow profile computations begin at an upstream cross section of known, or assumed elevation and proceed downstream. At each cross section, critical elevation is calculated through an iterative process by assuming a water surface elevation and calculating the total energy head until the minimum total energy value is obtained. A parabolic interpolation procedure is followed to expedite the iterative procedure.

Data Requirements

Data requirements for the HEC-2 model fall into the following categories: flow regime, starting elevation, discharge, loss coefficients, cross section geometry, and reach lengths.

The first three items, respectively, require (1) specification whether the flow is supercritical or subcritical, (2) provision of a starting elevation at the appropriate cross section, and (3) establishment of the discharge to be profiled.

There are three types of loss coefficients utilized by the HEC-2 program to establish head losses. They are: Manning's "n" values for friction loss; contraction and expansion coefficients to evaluate transition losses; and bridge loss coefficients to evaluate losses related to weir shape, pier configuration, and pressure flow.

Cross sections are required at representative locations throughout the study reach. This includes locations where changes occur in discharge, slope, shape, or roughness, as well as at locations where levees begin or end and at bridges or control structures such as weirs. Spacing of cross sections varies according to the hydraulic characteristics of the watrcourse as well as the study requirements. Cross sections should be, as much as possible, perpendicular to the anticipated flow and should extend across the entire floodplain.

Cross sections are described by coordinates of distance and elevation starting with the zero station on the left, looking downstream. All points of significant change in elevation should be included in the cross section with the exception of areas such as localized depressions, ponds, and the like, which are ineffective flow areas.

Optional Capabilities

The flexibility of application of the HEC-2 program is enhanced by several optional capabilities in addition to the myriad of, "built in", small discretionary features two numerous to mention. The following is a list of program options:

1. Multiple Profile Analysis - allows the computation of up to 14 profiles in a single run;

2. Critical Depth;

3. Effective Flow Options - used to specify areas of effective and ineffective flow in cross sections;

4. Bridge Losses;

5. Encroachment Options - provides six methods of specifying channel encroachments for floodway studies;

6. Optional Friction Loss Equations - provides a selection of four methods for calculating friction loss;

7. Channel Improvement - automatically modifies cross section data to account for improvements made to natural stream sections;

8. Interpolated Cross Sections - inserts cross sections between those specified by input when the change in velocity head is too great to accurately determine the energy gradient;

9. Tributary Stream Profiles - computes subcritical profiles for tributary stream systems for single or multiple profiles, in a single program run;

10. Solving for Manning's "n" - offers a choice of two ways to calculate Manning's "n";

11. Storage Outflow Data - output from HEC-2 runs are in a format suitable for streamflow routing by the Modified Puls Method using the program HEC-1;

12. Analysis of Flow in Ice Covered Streams; and

13. Split Flow Option - accommodates the computation of profiles in situations when flow leaves the watercourse and doesn't return.

Program Limitations

The four primary assumptions implicit in the analytical approach employed by the program are: (1) Flow is steady, (2) Flow is gradually varied, (3) Flow is one dimensional (in the sense that velocity components in directions other than the direction of flow are not taken into account), and (4) River Channels have slopes less than 1:10. These four basic assumptions also define the limitations imposed on the Model.

An additional restriction is imposed by the limited computational approaches employed by the program in the determination of energy losses. One final confinement stems from the rigid cross section boundaries assumed by the model, making it incapable of dealing with problems such as sediment transport.

STORAGE-EFFECTIVE-DRAINAGE
"SED" MODEL

F. Davies[1]

The Storage-Effective-Drainage, or SED model, relates input to output using a functional relationship of storage. It is capable of accurately modelling basin runoff both in terms of volume and shape.

The SED model was developed by the Hydrology Branch of Alberta Environment during the research activities of the Spring Creek Research Basin. A misunderstanding of storage, and the importance of its hydrologic control on runoff, was a major stumbling block in the research studies. The SED model provides a relatively simple means of analysing the impact of storage (its functional relationship (phi/y)) on runoff. This understanding of basin storage must be considered one of the major contributions of the Spring Creek research to the science of Hydrology.

There are four major points that will be discussed during the workshop:

1. Effective Drainage has a functional relationship between runoff volume and storage, which can be modelled. Figure 1, "Developing the SED Concepts", shows the development of a SED curve from a rudimentary relationship, using traditional hydrologic data (y vs phi), to the actual relationship for Spring Creek. The final relation takes the form of an autocatalytic growth curve, when extended to the full range of storage states of the basin.

1. F. Davies, P. Ag., Hydrologist, Hydrology Branch, Technical Services Division, Alberta Environment.

2. There are three independent techniques that can be used to develop the SED relationship. Figure 2, "Methods to Obtain the SED Relationship", provides a description of each.

3. The SED model not only accurately models the runoff volume, but also determines the shape of the hydrograph. Figure 3, "Developing Hydrograph Shapes from the SED curve", illustrates this technique. It describes how a hydrograph is mapped onto the SED curve by integrating the hydrograph over a SED curve segment. When the rising, and recession limb, segments are related using conservation of mass, and the SED curve, accurate hydrograph shapes are produced.

4. Figure 4, "Hydrograph Results From SED Modelling", provides three examples of the results the model is capable of.

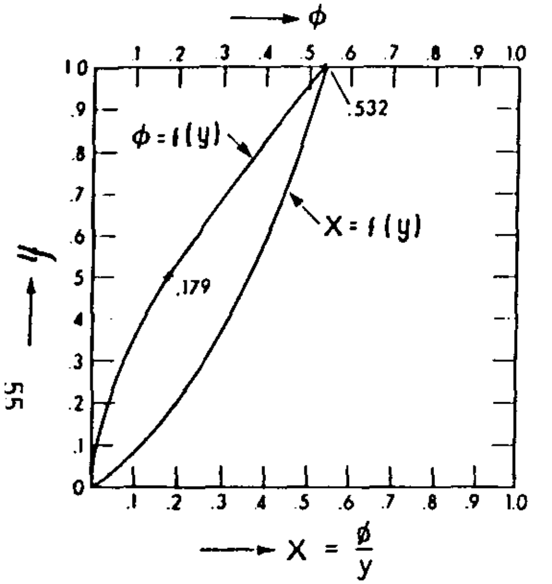
As a result of the Spring Creek research, and the development of the SED model, it is clear that effective drainage is a function of storage. A basin's storage state must be evaluated to determine what a runoff response will be to a given input.

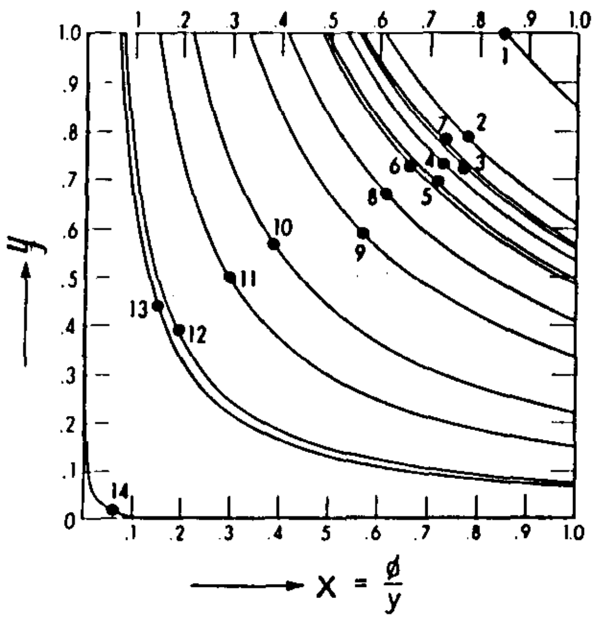
The concepts of the SED model are presented by G. Holecek in a paper entitled; "Storage-Effective-Drainage (SED) Runoff Model", published in the Journal of Hydrology, V-98 (1988), page 294 - 314.

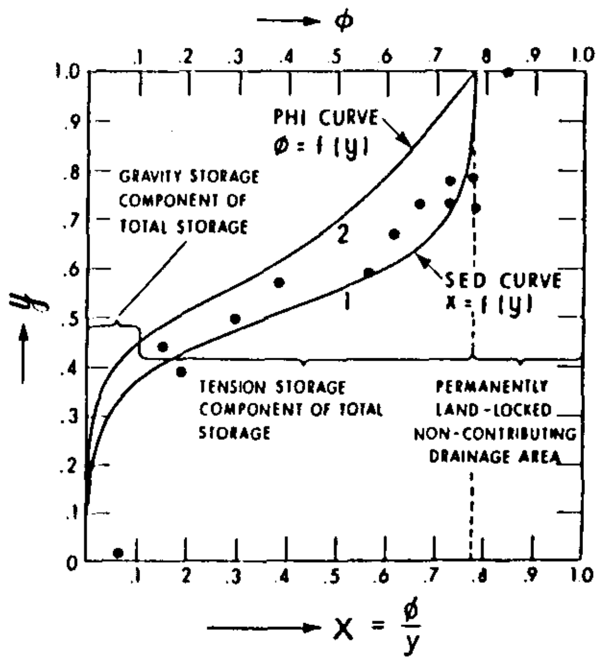
BASIN STORAGE VS EFFECTIVE DRAINAGE

'S E D' MODELLING

DEVELOPING THE 'S E D' CONCEPTS







STEP 1

OBSERVE
1. THE RELATIONSHIP OF STORAGE 'Y' VS EFFECTIVE DRAINAGE 'X' OVER THE FULL RANGE OF BASIN STORAGES
2. USING 3 POINTS (MAX., MEAN, MIN.)
3. PLOT THE RELATIONSHIPS:
   STORAGE 'Y' VS RUNOFF COEFFICIENT 'PHI'
   Y VS STORAGE 'Y' AND
   RUNOFF COEFFICIENT 'PHI'

STEP 2

OBSERVE
1. THE RELATIONSHIP OF STORAGE VS EFFECTIVE DRAINAGE OVER THE FULL RANGE OF BASIN STORAGES
2. USING 14 POINTS
3. PLOT THE RELATIONSHIP:
   RUNOFF COEFFICIENT VS STORAGE
   Y VS STORAGE 'Y' AND
   RUNOFF COEFFICIENT 'PHI'

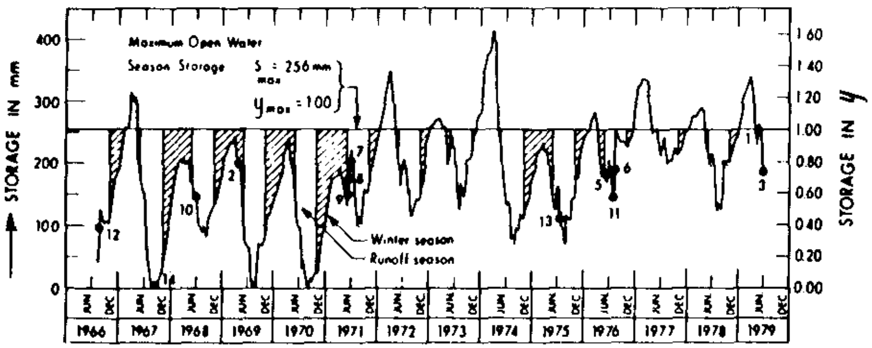
STEP 3

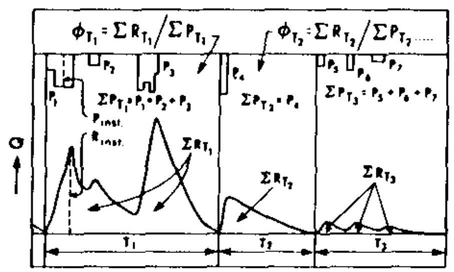
OBSERVE
1. EQUATIONAL FORM OF THE RELATIONSHIP IS AUTOCATALYTIC WHEN VIEWED FROM LOW TO HIGH STORAGE. AUTOCATALYTIC GROWTH IS ALSO REFERRED TO AS LOGISTIC GROWTH CURVE.
2. GROWTH IN EFFECTIVE DRAINAGE OVER THE RANGE OF BASIN STORAGES TAKES AN AUTOCATALYTIC FORM.
3. THE ASYMPTOTIC RELATIONSHIP OF THE BOUNDARY CONDITIONS (UPPER AND LOWER STORAGE LIMITS) ARE OPEN ENDED.

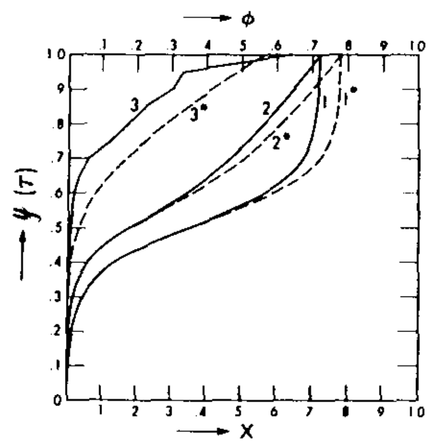
BASIN STORAGE VS EFFECTIVE DRAINAGE

'SED' MODELLING

METHODS TO OBTAIN THE 'SED' RELATIONSHIP







METHOD 1

USE A BASIN WATER BALANCE.
FROM A BASIN WATER BALANCE PLOT IDENTIFY INDIVIDUAL STORM'S RUNOFF VOLUMES AT CORRESPONDING STORAGE CONDITIONS. LET THE MAXIMUM STORAGE AMPLITUDE REPRESENT THE RANGE OF STORAGE CHANGE IN THE BASIN.

METHOD 2

USE SEASONALLY LUMPED INPUT AND OUTPUTS.
FORM THESE RUNOFF COEFFICIENTS ASSUMING THE STORAGE EFFECTIVE RUNOFF RELATIONSHIP MUST PASS THROUGH THE MEAN, MAXIMUM AND MINIMUM OF THESE POINTS. FORCE A SED RELATIONSHIP THAT FITS ALL POINTS.

METHOD 3

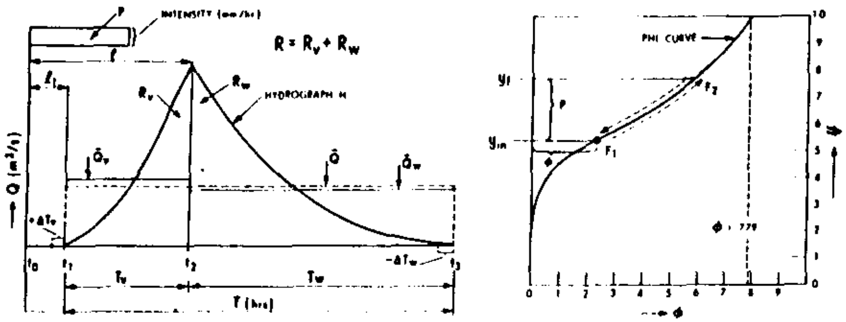
USE FLOW DURATION CURVES.
A SMOOTH PLOT OF THE FLOW DURATION CURVE REPRESENTS THE RANGE OF STORAGE STATES. THE DERIVATIVE OF THIS RELATIONSHIP YIELDS:
STORAGE VS 'PHI' RELATIONSHIP, FROM WHICH THE 'SED' RELATIONSHIP CAN BE DETERMINED.

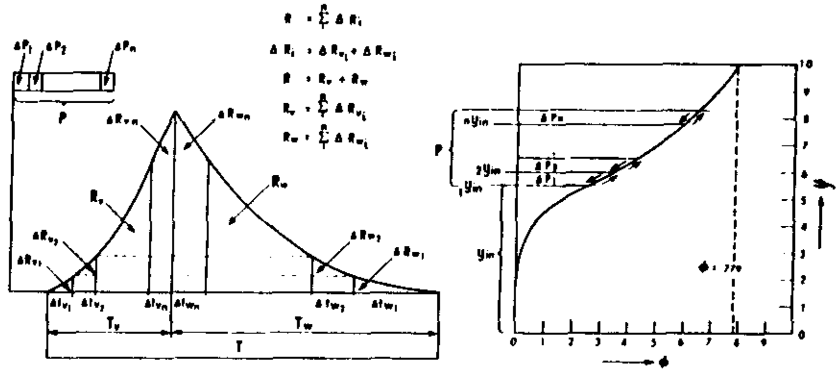
FIG. 2

BASIN STORAGE VS EFFECTIVE DRAINAGE

'SED' MODELLING

DEVELOPING HYDROGRAPH SHAPES FROM THE 'SED' CURVE





POINT 1

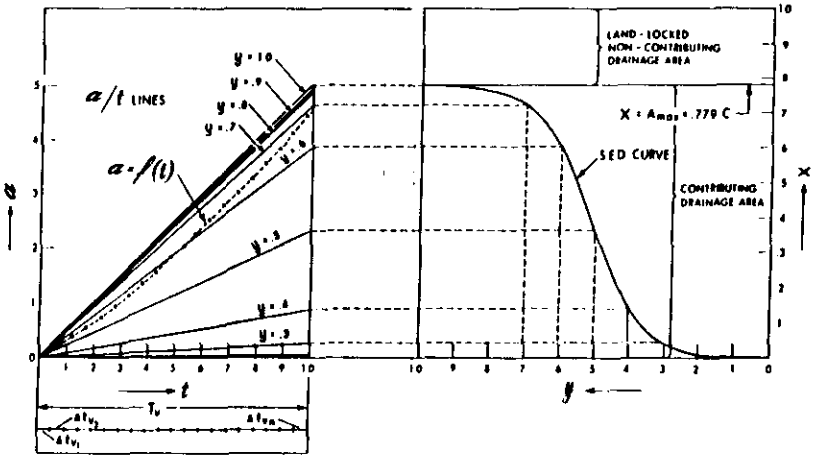
OBSERVE

THE RISING LIMB OF A HYDROGRAPH CHANGES FROM POINT F1 TO F2 (ON A SED CURVE) AS A RESULT OF INPUT P. THE RECESSION LIMB MOVES BACK OVER THE SAME RANGE FROM F2 TO F1 WITHIN SOME UNKNOWN RECESSION TIME PERIOD. THE RECESSION LIMB IS COMPLETE WHEN STORAGE HAS REACHED THE F1 AGAIN.

POINT 2

OBSERVE

INTEGRATION OF THE STORMS INPUTS OVER THE STORAGE RANGE RESULTS IN PAIRED HYDROGRAPH SEGMENTS (A RISING AND RECESSION SEGMENT ALONG THE 'SED' CURVE). THE MEAN Q FOR BOTH SEGMENTS ARE THE SAME.



POINT 3

OBSERVE

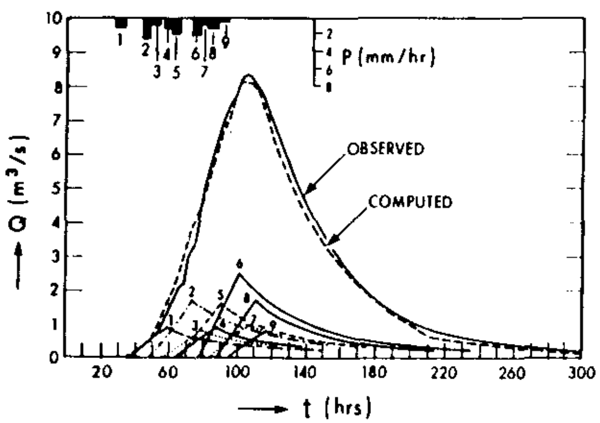
THAT THE VOLUME OF THE PAIRED SEGMENTS ARE NOW KNOWN. USING THE RISING LIMB TIME FORM SEVERAL KNOWN HYDROGRAPHS, AN EQUATION OF TV (RISING LIMB TIME) VS P AND PHI STARTING AT Y INITIAL, TRANSLATES THE 'SED' CURVE INTO A FAMILY OF HYDROGRAPH TIMINGS RELATIVE TO THE SED CURVE'S SLOPE. WITH THE RELATIVE RELATIONSHIP BETWEEN V (VOLUMES), Q (DISCHARGES) AND T (TIME) OVER THE INTEGRATED STORAGE INTERVAL AND THE CONSERVATION OF MASS ASSOCIATED WITH THE INITIAL STORAGE ON THE SED CURVE, HYDROGRAPH SHAPES ARE NOW POSSIBLE.

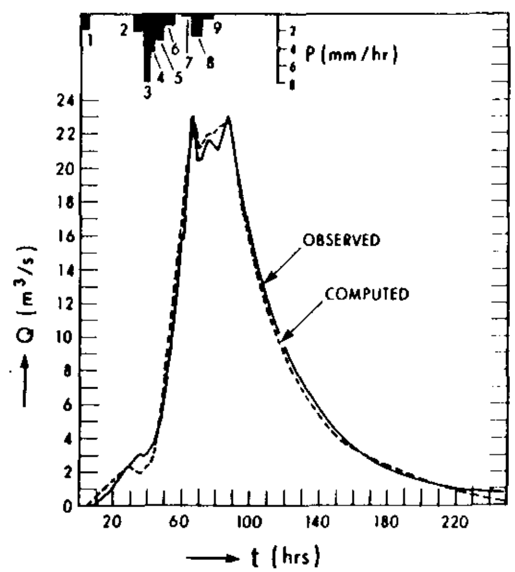
FIG. 3

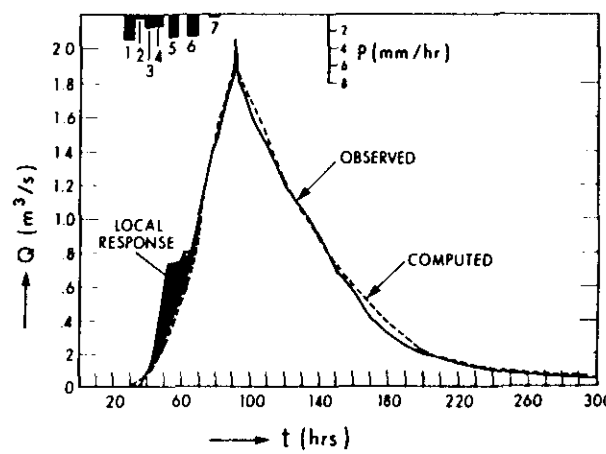
BASIN STORAGE VS EFFECTIVE DRAINAGE

'S E D' MODELLING

HYDROGRAPH RESULTS FROM 'S E D' MODELLING







EXAMPLE 1

INITIAL STORAGE = 0.594
TOTAL PRECIPITATION = 63.700 MM
EFFECTIVE DRAINAGE = 22.200 MM

EXAMPLE 2

INITIAL STORAGE = 0.697
TOTAL PRECIPITATION = 106.000 MM
EFFECTIVE DRAINAGE = 53.200 MM

EXAMPLE 3

INITIAL STORAGE = 0.429
TOTAL PRECIPITATION = 69.400 MM
EFFECTIVE DRAINAGE = 5.700 MM

FIG. 4

HYDRO SYSTEM SIMULATION (HYDSIM) MODEL

by

Ramaiah Divi, Pawan K. Vohra and Dan Ruiu
SaskPower, 2025 Victoria Avenue, Regina, Saskatchewan S4P OS1

Introduction

Optimal management of multi-purpose multi-reservoirs located in several river basins in a hydro-thermal power system is indeed a complex problem. There are several reasons for it. Most notable ones are: Future Electric Load demand and river flows cannot be predicted with certainty (i.e. random variables); each water user group has different objectives to fulfill which are hard to quantify and some of them conflict with each other; and the need to recognize the continuous trade-off between using water for the immediate needs and storing water to maximize the future benefits.

In the case of hydro-thermal power system, yet another problem, namely, "hydro-thermal" co-ordination needs to be addressed in the development of reservoir operating policies. That is, in order to cut down the fuel costs, the hydro power production must be properly co-ordinated (i.e. the timing and quantity of reservoir releases) with the scheduling of available thermal (i.e. coal burning and gas-fired) power plants. Because of fuel costs involved, it is very expensive to run the thermal plants compared to hydro plants.

The problem is to find optimal reservoir operating policies and hydro generation schedules for periods varying from one day to several seasons or years into the future. Typical criterion for optimization include: minimize the production cost (i.e. sum of fuel, operating and maintenance), maximize the hydro power production, etc.

Numerous approaches have been advocated in the literature for the formulation of optimal water resources management problems and the techniques for solving them. Reference [1] presents an integrated approach, which was adopted for the planning and operation of SaskPower's hydro thermal power system. This approach primarily consists of dividing the overall water resources management problem into several sub-problems; solve these sub-problems independently; and interconnect these solutions to obtain the solution to the original problem. Several large scale computer programs were developed for this purpose and the application of these tools for the planning and operation of multi-reservoir systems is shown in Fig. 1. Hydro system simulation (HYDSIM) is one of those programs mentioned above. The purpose of this paper is three-fold. First, to present an overview of the HYDSIM computer model, the formulation of the multi-purpose multi-reservoir simulation problem and briefly discuss the solution techniques. Second, to outline the features of the HYDSIM model and its potential applications. Third, to indicate some of the limitations of the model. In addition, an appendix is attached which includes a short discussion on the input-output system of the program and sample results of the model. The South Saskatchewan River Basin Study (SSRB) Board has selected the HYDSIM program as one of the tools for use in the water management studies.

An Overview of the HYDSIM Model

For production planning (planning horizon ranging from a few months to two years) and long-term planning studies (2 years to 20 years), it is often necessary to develop the future operating policies for both the existing and planned reservoirs and the associated hydro plants as well in advance. Factors to be considered in the determination of such policies include: Forecasts of water demands for irrigation, municipal and industrial, diversions into other basins, recreational needs, hydro power demands; inflows, net evaporation, flood control, reparian flows, environmental considerations and of course the configuration of the river system, operational constraints on reservoir levels and releases, and plant capacities. One of the major issues in the regulation of multi-purpose reservoir systems is the allocation of water to competing users in an adequate manner. Due to the lack of any proper technique for water allocation, a heuristic approach is adopted in this model. It is based upon a priority system. The multi-reservoir problem can be posed as follows:

Statement of the Simulation Problem

A planning horizon is divided into a suitable number of operating periods.

Given:

i) River system description;
ii) Hydraulic models of reservoirs, tailrace, etc.
iii) Hydro plants models;
iv) Start and end reservoir levels.

and for each period:

a) Water demands for irrigation, municipal and industrial use, water diversion to other basins;
b) The priorities for irrigation, municipal and industrial use, water diversion, recreation and power production user groups;
c) Streamflows; evaporation and seepage rates;
d) Hydro unit maintenance schedules;
e) Operational constraints;
f) Plant energy or discharge or lake level (i.e. desired targets).

Find:

Reservoir operating policies and hydro generation schedules that are feasible.

The above simulation problem is a nonlinear because the operating characteristics of reservoirs, tailraces and the hydro unit/plants are intrinsically nonlinear. The basic principle behind a simulation program is to compute the response of the reservoir system for a given set of flow conditions and operating scenarios.

Solution Techniques

To solve the above stated nonlinear multi-purpose reservoir regulation problems, two techniques i.e. i) heuristic, and ii) one-period optimization were utilized in the HYDSIM program. The users, thus, have the choice to select either one of them as solution technique for their applications at hand. These methods are briefly described below:

i) Heuristic Technique

Heuristic technique employed here primarily consists of a set of "built-in" rules for the simulation of reservoir operations. These rules are derived based upon the modern water resources management principles, past operating and computational experience and engineering judgement.

A two-stage algorithm was developed for determining the feasible solution which includes the allocation of water to multiple users. The priority scheme essentially consists of specifying an integer number i.e. 1 to N to the individual water user groups like irrigationalists, recreationalists, power production, etc., where 1 represents the highest priority and 2 represents the second priority, and so on. The analyst or the program user decides upon the order of priorities which can be changed from one period to another for each reservoir and each water user group. The program, therefore, can be used repeatedly to evaluate the impact or sensitivity of these priorities on the operating policies and vice versa.

In Stage 1, the water demands of all off-stream users (e.g. irrigation, municipal and industrial, water diversion) are added together to obtain one equivalent (aggregated) user and a priority is reassigned to it. Thus, the number of user groups are reduced from 6 to 3, namely, off-stream, hydro power production, and recreationalists thereby simplifying the development of computational logic for water allocation. The program allocates water to these three main groups according to the priorities. In the case of water shortages, the least priority group would be most affected while the highest priority group may be least affected. The program is so designed that the analyst can control the severity of shortages experienced by various groups by means of input control parameters.

In the second stage, the total water allocated to the aggregated off-stream user group in Stage 1 (which may be different from the original demand) would be divided to its sub-groups, namely, irrigation, municipal and industrial, and diversion according to their priorities.

Main advantages of the above two-stage approach are: the computational logic for allocating water has been simplified, and it resulted in faster program execution times.

Typically, the reservoir simulation begins with the selection of a river system. Then, using the above stated two-stage algoritm, it computes a feasible operation of the most upstream reservoir in a given period and proceeds downstream along the direction of water flow until all the reservoirs and plants are exhausted on that river system. This process is then repeated for other river valleys. That is, it simulates the operation of one reservoir at a time and does not consider the effects of its decision on the downstream reservoirs.

ii) <u>A One-period Optimization Method</u>

The determination of a feasible operating policy in each period has been formulated as an optimization problem. The objective is to minimize the sum of the squared deviations between the desired policy (user specified) and a feasible policy subject to numerous operational constraints. This constrained nonlinear optimization problem was solved by a penalty function technique [2]. This optimization process is then repeated for all the periods.

The one period optimizer, thus computes a feasible operation of <u>all reservoirs</u> and plants <u>in the system</u> at a time in any given period taking into account their inter-dependencies. Heuristic method, on the other hand, simulates the operation of one reservoir at a time in any given period and does not consider the likely impact of its decision on the downstream plants. The one-period optimization method, therefore, computes a better solution than the heuristic method. It would be very attractive for situations where the total hydro power production of the system is only known and needs to be allocated to individual plants in an efficient manner. The one-period optimizer, however, takes more CPU time (about 1.5 times) than the heuristic method.

The one-period optimization method, in the present form, treats all the off-stream users as a fixed demand and attempts to satisfy it all the time while computing a feasible solution. In the next version, the program will be upgraded to include the individual off-stream uses as independent variables.

Features of HYDSIM

HYDSIM is a general purpose computer program for use in the simulation of operation of multi-purpose multi-reservoir hydro systems. It was written in FORTRAN-77 and has been operational on the IBM 4381 system since 1985. The program can be run on variable (e.g. weekly, 10-day, half-monthly or monthly) time periods. It allows detailed modelling of the operating characteristics of major components e.g. reservoirs, spillways, power plants. The user has three options to select from for the estimation of hydro energy production.

i) The plant output as a function of discharge for a series of net heads (uses one-dimensional polynominal models) (recommended for operations planning).

ii) Calculate the MW output of individual units using their polynominal models, and the unit outputs are added together to obtain the plant output (recommended for feasibility studies).

iii) Use a standard formula. Power output = constant*discharge*net head (recommended for pre-feasibility studies).

Other features include:

- Useful for both operations planning and long-term studies;
- Models the hydro plants with different types and sized units;
- Has special logic to determine the combined operation of plants where the tailrace of an upstream plant is controlled by the forebay of its downstream plant (e.g. Nipawin's tailrace is affected by the Tobin Lake's level).
- The operating decision for a reservoir can be specified in one of four ways (i.e. targets for discharge, reservoir volume or elevation, power production).
- Two options for incorporating the net evaporation rates in the reservoir regulation (i.e. constant quantities or net evaporation adjusted to the lake level in each period).
- Computational requirements are minimal. As an example, it took 37 CPU seconds on IBM 4381 to complete one five-year simulation run of six major reservoirs and associated plants (existing and planned) on the Saskatchewan River basin on monthly time periods.

The program outputs results in nicely formatted tables (see the appendix) and has option to obtain graphical plots as well.

Potential Applications

The computer model could be used in the following areas:

- Hydro site investigations;
- Water resources planning studies;
- Hydro firm capacity or energy projections;
- Flood management studies;
- Hydro power production and fuel budgeting;

- Selection of hydro turbines;
- Evaluation of system improvements (e.g. tailrace channel improvements, plant expansion or upgrading, etc.);
- Hydrological modelling.

Model Limitations

- It is presently dimensioned to solve systems with a maximum of 15 reservoirs and 60 periods. It can be re-dimensioned to solve larger problems. Personal computer (PC) version of it is not available.
- It does not model the irrigation projects in any detail at all.
- It is a deterministic model. That is the inflows and water demands are assumed to be known in advance.

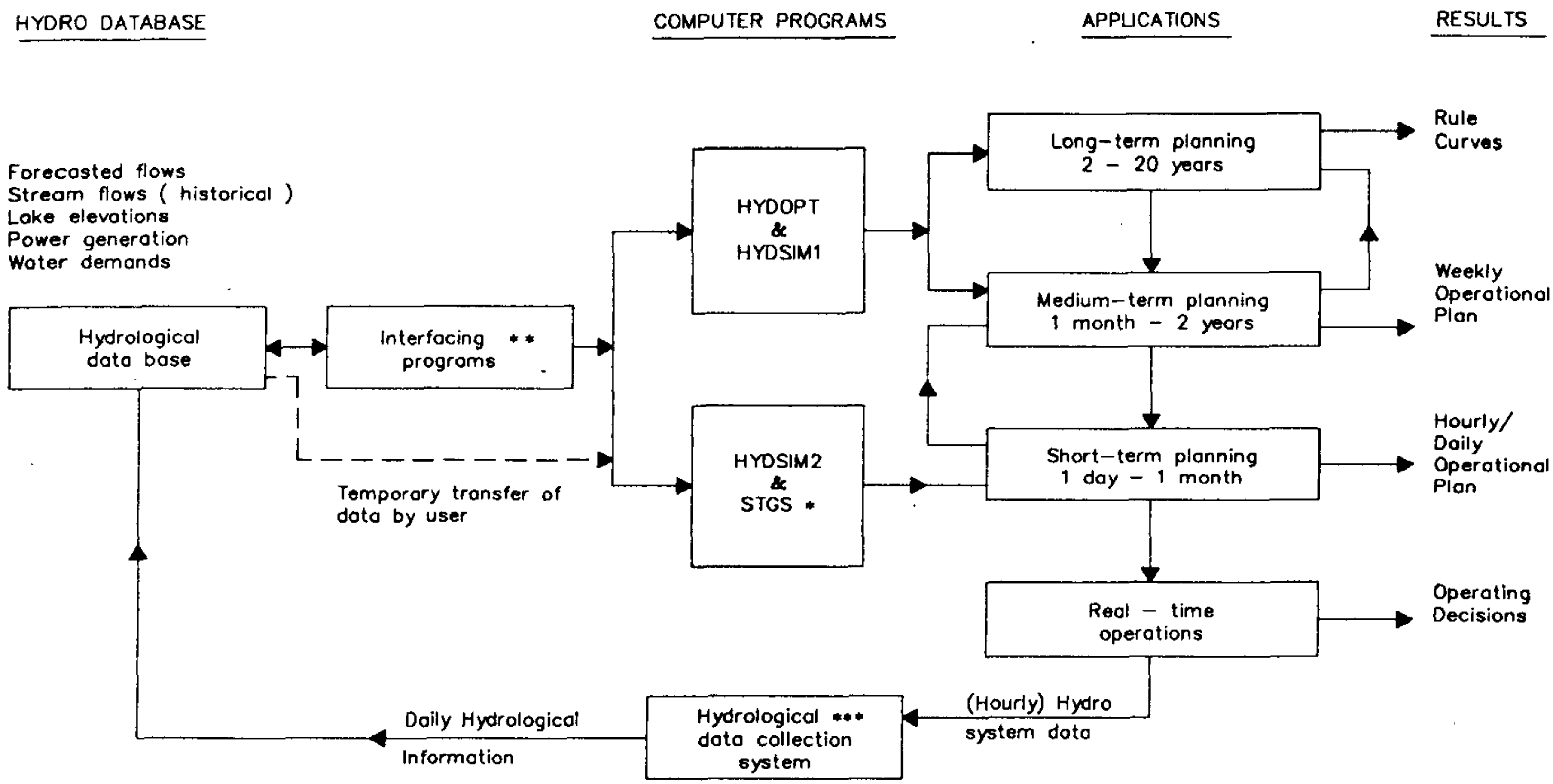
Model Enhancements

A project is underway to develop a set of interface routines to extract the required flow data, water demand data, etc. from a hydrology database and transfer them to the HYDSIM's input data files. This scheme will free the users from the tedious task of entering the huge quantities of data into the files and practically eliminate the data entry errors.

References

1. R. Divi and Dan Ruiu, "Optimal Management of Multi-Purpose Reservoirs In A hydro-Thermal Power System" presented at the Third Water Resources Operations Management Work Shop: Computerized Decision-Support Systems for Water Managers, June 1988, Fort Collins, Colorado, U. S. A.

2. Charlambous, C., "A method to over the ill-conditioning problem of differentiable penalty functions", Technical Report No. 31-0-181077, Dept. of Systems Design, U of Waterloo, October '77.



* Testing Stage
** Under development
*** To be developed

HYDOPT: Hydro optimization (weekly/monthly model – up to 10 yrs.)

HYDSIM1: Hydro system simulation (weekly/monthly model – up to 5 yrs)

HYDSIM2: Hydro system simulation (hourly model – up to one week)

STGS: Short-term generation scheduler (hourly model – up to one week)

FIG. 1 A SCHEMATIC REPRESENTATION OF AN INTEGRATED WATER RESOURCES MANAGEMENT SYSTEM

APPENDIX

Input/Output System

A simplified functional layout of the HYDSIM Program is shown in Fig. 2. The inputs to the program have been classified into three groups, namely, i) control or run parameters, ii) Operational Data, and iii) System Description or Permanent Data.

i) Control Data

This file has been primarily designed to help the user to select the type of run he or she wishes to make. Thus the user has to enter data such as the length of the planning horizon, the number of years in it, the operating time interval (week or month), etc. Also, there are a number of options provided to the user to select the printing of input data and output results for any run.

ii) Operational Data

Those inputs which vary from one run to another are grouped and included in this file. The user can specify the reservoir operating rule in one of the four following ways, namely, the desired discharge, the period end volume or elevation, the energy power production. The environmental concerns and recreational demands can be modeled either in terms constraints on discharge or reservoir elevations. The program allows the specification of forced spillage if required, on a period-by-period basis.

The operational constraints on lake levels, discharges, power production, etc. can be dynamically (i.e. period-by-period) allocated.

For power production, the program expects maintenance schedules for units and it can also handle any deratings on the units.

The user has considerable flexibility in adapting this program for water management applications.

The data to be entered thus consists of:

- Inflows;
- Local inflows;
- Irrigation ; industrial and municipal demands, water diversion;
- Priorities for different water users;
- Evaporation;
- Seepage;
- Forced spill;
- Desired decision;
  (either discharge or volume or period-end elevation or power production i.e. target values);
- Operational constraints on reservoir levels, discharges, etc.;
- Riparian flows;
- Hydro unit maintenance schedules;
- Recreational and/or environmental concerns
  (model them either in terms of discharge or lake level).

iii) System Description or Permanent Data

All the data pertaining to the elements that describe the river or multiple river systems under study is included in this data file. In addition, it contains the mathematical models that represent the operating characteristics of reservoirs, hydro units and plants, tailrace characteristics and spillways.

Thus, the following information is to be supplied.

- Name(s) of river(s) system;
- Number of hydro plants and their names;
- Number of reservoirs and their names;
- Types of hydro plants and reservoirs;
- Mathematical models to compute forebay levels/reservoir volumes;
- Mathematical models to compute tailrace levels;
- Mathematical models to compute head losses;
- Mathematical models to compute power output of plants at different methods.

The above file needs to be set up only once for any given river system and it remains the same as long as the configuration of the system or the characteristics of the components are not altered.

NOTE:

For the Saskatchewan River system and all the existing three reservoirs (Lake Diefenbaker, Codette and Tobin Lake) and hydro plants (Coteau Creek, Nipawin and Squaw Rapids), we have developed the models and prepared the permanent data file and is readily available. No more additional work is needed for the Saskatchewan River system to run the HYDSIM program.

However, if the user wishes to adapt HYDSIM to other river systems, then he has to undertake this work (i.e. develop the system models and preparation of data files).

The output consists of:

- reservoir discharges,
- reservoir elevations,
- inflows,
- power production, etc.

The program prepares a number of reports in nice, tabular format for direct use in technical reports and presentations (see the attached sample).

In addition to the final output, the user can access the intermediate results to learn about how the program performs calculations. Also, the program prints certain messages about the results and input data.

A few sample output reports are attached here to illustrate the type of information that can be obtained from this program. The sample reports pertain to one year simulation results of the Saskatchewan River system (six reservoirs, i.e. Lake Diefenbaker, Dundurn (proposed), St. Louis (proposed), Forks (proposed), Nipawin, and Tobin Lake).

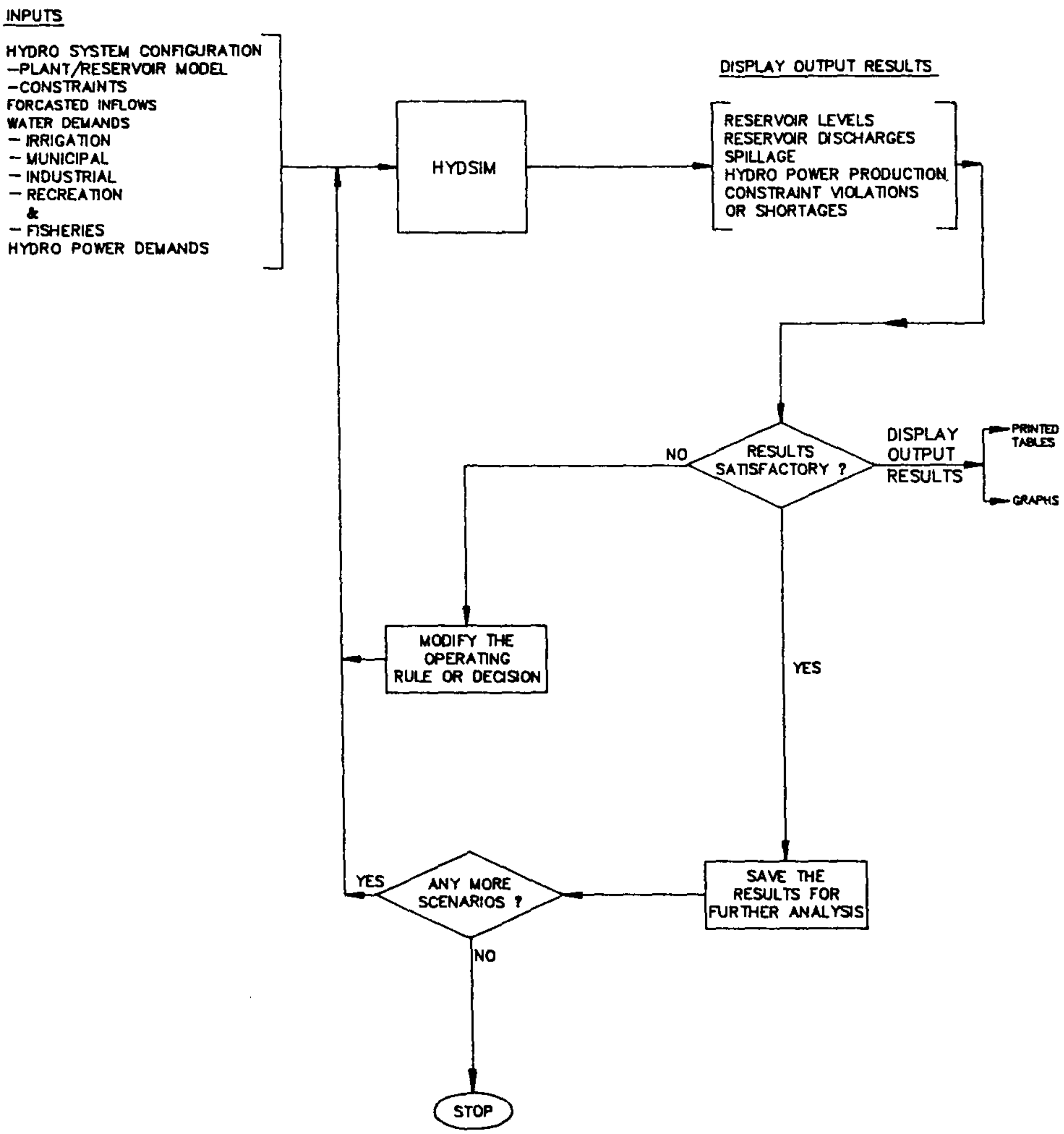


FIG. 2 A SCHEMATIC REPRESENTATION OF THE USE OF HYDRO SYSTEM SIMULATION (HYDSIM) MODEL

0231007

HYSIM1 SIMULATION OUTPUT STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

A SUMMARY REPORT- SCHEDULING OF HYDRO PLANTS

PLANT NAME	UPSTREAM RESERVOIR	ENERGY SCHEDULED (GWH)	ENERGY PRODUCED (GWH)	START FOREBAY ELEVATION (METERS)	END FOREBAY ELEVATION (METERS)	STORAGE LEFT (DAYS)
COTEAU CREEK	DFNBKR	0.0000	834.6459	552.607	554.683	82.7209
DUNDURN	DNDRN	0.0000	165.0902	501.843	501.927	7.3645
ST LOUIS	ST LOU	0.0000	359.4662	453.902	453.926	0.9245
FORKS	FORKS	0.0000	1844.9277	420.013	420.008	0.6100
NIPAWIN	CODETT	0.0000	1325.1898	347.054	347.696	0.2979
SQUAW RAPIDS	TOBIN	0.0000	1235.6883	313.585	313.575	5.6842

TOTAL SCHEDULED ENERGY= 0.57649687E+04 (GWH)

TOTAL ENERGY OUTPUT = 0.57650087E+04 (GWH)

TOTAL WINTER HYDRO ENERGY (SEP-MAR) = 3081.1394 (GWH)

TOTAL SUMMER HYDRO ENERGY (APR-AUG) = 2683.8686 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)*3600.0*24.0}}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

HYSIM1 SIMULATION OUTPUT FOR SASKATCHEWAN RIVER STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

PLANT NAME: COTEAU CREEK UPSTREAM RESERVOIR NAME: LAKE DIEFENBAKER START FOREBAY ELEVATION: 552.61 (M) YEAR-1986

PERIOD NUMBER	STARTING DATE	ENERGY PRODUCED (MW)	ENERGY PRODUCED (GWH)	TOTAL INFLOW (M3/S)	PLANT DISCHARGE (M3/S)	SPILL (M3/S)	END FOREBAY ELEVATION (METERS)	NET HEAD (METERS)	STORAGE LEFT (DAYS)	EFFICIENCY FACTOR (MW/(M3/S))	CAPACITY FACTOR
1	JAN 1	109.90	81.76	115.30	240.34	0.00	551.67	50.38	52.35	0.46	0.59
2	FEB 1	104.40	70.16	93.50	231.49	0.00	550.67	49.59	42.92	0.45	0.56
3	MAR 1	66.25	49.29	294.80	153.58	0.00	551.67	50.92	52.32	0.43	0.35
4	APR 1	31.32	22.55	182.10	71.77	0.00	552.44	52.15	59.76	0.44	0.17
5	MAY 1	59.64	44.37	321.70	137.09	0.00	553.57	50.81	71.07	0.44	0.32
6	JUN 1	103.55	74.56	623.00	212.57	0.00	556.01	53.57	97.17	0.49	0.55
7	JUL 1	92.77	69.02	347.90	185.10	0.00	556.76	55.65	105.65	0.50	0.49
8	AUG 1	84.43	62.81	187.40	169.62	0.00	556.63	56.20	104.20	0.50	0.45
9	SEP 1	66.46	47.85	137.00	139.82	0.00	556.50	54.25	102.70	0.48	0.35
10	OCT 1	150.26	111.80	294.90	299.79	0.00	556.41	55.05	101.73	0.50	0.80
11	NOV 1	142.35	102.49	149.80	289.11	0.00	555.53	54.69	91.94	0.49	0.76
12	DEC 1	131.69	97.98	148.80	275.81	0.00	554.68	53.97	82.72	0.48	0.70

PLANT HYDRO ENERGY FOR YEAR 1986 = 834.6459 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)*3600.0*24.0)}}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

HYSIML SIMULATION OUTPUT FOR SASKATCHEWAN RIVER STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

PLANT NAME: DUNDURN UPSTREAM RESERVOIR NAME: DUNDURN START FOREBAY ELEVATION: 501.84 (M) YEAR-1986

PERIOD NUMBER	STARTING DATE	ENERGY PRODUCED (MW)	ENERGY PRODUCED (GWH)	TOTAL INFLOW (M3/S)	PLANT DISCHARGE (M3/S)	SPILL (M3/S)	END FOREBAY ELEVATION (METERS)	NET HEAD (METERS)	STORAGE LEFT (DAYS)	EFFICIENCY FACTOR (MW/(M3/S))	CAPACITY FACTOR
1	JAN 1	21.09	15.69	240.34	189.00	49.49	501.93	17.01	7.36	0.11	0.46
2	FEB 1	21.15	14.21	231.49	189.00	42.49	501.93	17.06	7.36	0.11	0.47
3	MAR 1	21.48	15.98	153.58	153.55	0.00	501.92	17.24	7.35	0.14	0.47
4	APR 1	0.00	0.00	71.77	66.07	0.00	501.92	17.44	7.35	0.00	0.00
5	MAY 1	15.35	11.42	137.09	131.29	0.00	501.92	17.29	7.35	0.12	0.34
6	JUN 1	21.21	15.27	212.57	189.00	17.80	501.93	17.12	7.36	0.11	0.47
7	JUL 1	20.00	14.88	185.10	179.46	0.00	501.92	17.18	7.35	0.11	0.44
8	AUG 1	25.01	18.61	169.62	169.42	0.00	501.92	17.20	7.35	0.15	0.55
9	SEP 1	17.55	12.63	139.82	139.82	0.00	501.92	17.27	7.35	0.13	0.39
10	OCT 1	20.98	15.61	299.79	189.00	110.72	501.93	16.91	7.36	0.11	0.46
11	NOV 1	21.01	15.13	289.11	189.00	100.11	501.93	16.94	7.36	0.11	0.46
12	DEC 1	21.04	15.65	275.81	189.00	86.81	501.93	16.97	7.36	0.11	0.46

PLANT HYDRO ENERGY FOR YEAR 1986 = 165.0902 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)} * 3600.0 * 24.0}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

HYSIM1 SIMULATION OUTPUT FOR SASKATCHEWAN RIVER STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

PLANT NAME: ST LOUIS UPSTREAM RESERVOIR NAME: ST LOIS LAKE START FOREBAY ELEVATION: 453.90 (M) YEAR-1986

PERIOD NUMBER	STARTING DATE	ENERGY PRODUCED (MW)	ENERGY PRODUCED (GWH)	TOTAL INFLOW (M3/S)	PLANT DISCHARGE (M3/S)	SPILL (M3/S)	END FOREBAY ELEVATION (METERS)	NET HEAD (METERS)	STORAGE LEFT (DAYS)	EFFICIENCY FACTOR (MW/(M3/S))	CAPACITY FACTOR
1	JAN 1	50.69	37.72	237.29	236.77	0.00	453.93	24.69	0.92	0.21	0.29
2	FEB 1	49.46	33.24	230.29	230.29	0.00	453.93	24.72	0.92	0.21	0.29
3	MAR 1	33.26	24.75	152.25	152.25	0.00	453.93	25.08	0.92	0.22	0.19
4	APR 1	0.00	0.00	64.87	64.87	0.00	453.93	25.80	0.92	0.00	0.00
5	MAY 1	28.06	20.88	128.39	128.19	0.00	453.93	25.24	0.92	0.22	0.16
6	JUN 1	44.02	31.70	203.50	203.30	0.00	453.93	24.82	0.92	0.22	0.26
7	JUL 1	38.41	28.57	176.56	176.36	0.00	453.93	24.94	0.92	0.22	0.22
8	AUG 1	36.23	26.96	166.32	166.12	0.00	453.93	25.00	0.92	0.22	0.21
9	SEP 1	30.23	21.77	138.22	138.22	0.00	453.93	25.17	0.92	0.22	0.18
10	OCT 1	63.10	46.95	298.32	298.32	0.00	453.93	24.56	0.92	0.21	0.37
11	NOV 1	60.83	43.80	287.71	287.71	0.00	453.93	24.58	0.92	0.21	0.35
12	DEC 1	57.99	43.15	274.41	274.41	0.00	453.93	24.60	0.92	0.21	0.34

PLANT HYDRO ENERGY FOR YEAR 1986 = 359.4662 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)}*3600.0*24.0)}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

HYSIM1 SIMULATION OUTPUT FOR SASKATCHEWAN RIVER STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

PLANT NAME: FORKS UPSTREAM RESERVOIR NAME: FORKS START FOREBAY ELEVATION: 420.01 (M) YEAR-1986

PERIOD NUMBER	STARTING DATE	ENERGY PRODUCED (MW)	ENERGY PRODUCED (GWH)	TOTAL INFLOW (M3/S)	PLANT DISCHARGE (M3/S)	SPILL (M3/S)	END FOREBAY ELEVATION (METERS)	NET HEAD (METERS)	STORAGE LEFT (DAYS)	EFFICIENCY FACTOR (MW/(M3/S))	CAPACITY FACTOR
1	JAN 1	155.78	115.90	370.87	369.86	0.00	420.01	47.05	0.61	0.42	0.36
2	FEB 1	146.06	98.15	344.89	343.79	0.00	420.01	47.08	0.61	0.42	0.33
3	MAR 1	125.01	93.01	296.65	295.25	0.00	420.01	47.13	0.61	0.42	0.29
4	APR 1	207.83	149.63	496.67	495.47	0.00	420.01	46.91	0.61	0.42	0.48
5	MAY 1	198.57	147.74	470.99	469.79	0.00	420.01	46.94	0.61	0.42	0.45
6	JUN 1	258.66	186.24	619.80	618.50	0.00	420.01	46.78	0.61	0.42	0.59
7	JUL 1	386.44	287.51	959.26	957.66	0.00	420.01	46.41	0.61	0.40	0.88
8	AUG 1	241.34	179.56	572.72	571.32	0.00	420.01	46.83	0.61	0.42	0.55
9	SEP 1	159.03	114.50	380.52	379.52	0.00	420.01	47.04	0.61	0.42	0.36
10	OCT 1	264.53	196.81	637.62	636.42	0.00	420.01	46.76	0.61	0.42	0.60
11	NOV 1	198.81	143.14	471.51	470.41	0.00	420.01	46.94	0.61	0.42	0.45
12	DEC 1	178.42	132.74	421.61	420.61	0.00	420.01	46.99	0.61	0.42	0.41

PLANT HYDRO ENERGY FOR YEAR 1986 = 1844.9277 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)}*3600.0*24.0}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

HYSIM1 SIMULATION OUTPUT FOR SASKATCHEWAN RIVER STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

PLANT NAME: NIPAWIN UPSTREAM RESERVOIR NAME: CODETT START FOREBAY ELEVATION: 347.05 (M) YEAR-1986

PERIOD NUMBER	STARTING DATE	ENERGY PRODUCED (MW)	ENERGY PRODUCED (GWH)	TOTAL INFLOW (M3/S)	PLANT DISCHARGE (M3/S)	SPILL (M3/S)	END FOREBAY ELEVATION (METERS)	NET HEAD (METERS)	STORAGE LEFT (DAYS)	EFFICIENCY FACTOR (MW/(M3/S))	CAPACITY FACTOR
1	JAN 1	109.40	81.40	369.56	359.98	0.00	347.83	33.13	0.35	0.30	0.42
2	FEB 1	106.47	71.55	343.69	343.65	0.00	347.83	33.77	0.35	0.31	0.41
3	MAR 1	90.19	67.10	290.65	288.96	0.00	347.97	34.23	0.41	0.31	0.35
4	APR 1	155.83	112.20	500.67	504.31	0.00	347.68	33.66	0.29	0.31	0.60
5	MAY 1	143.72	106.93	469.69	466.30	0.00	347.94	33.55	0.40	0.31	0.56
6	JUN 1	186.10	133.99	608.10	607.98	0.00	347.90	33.42	0.39	0.31	0.72
7	JUL 1	258.27	192.15	956.86	864.00	89.38	348.09	32.86	0.46	0.30	1.00
8	AUG 1	178.06	132.48	580.32	584.92	0.00	347.59	33.22	0.25	0.30	0.69
9	SEP 1	117.80	84.82	384.52	387.82	0.00	347.28	33.08	0.12	0.30	0.46
10	OCT 1	189.42	140.93	635.02	627.68	0.00	347.85	32.91	0.36	0.30	0.73
11	NOV 1	146.04	105.15	477.41	478.04	0.00	347.80	33.28	0.34	0.31	0.57
12	DEC 1	129.72	96.51	423.91	425.20	0.00	347.70	33.21	0.30	0.31	0.50

PLANT HYDRO ENERGY FOR YEAR 1986 = 1325.1898 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)*3600.0*24.0}}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

HYSIM1 SIMULATION OUTPUT FOR SASKATCHEWAN RIVER STUDY PERIOD: 1986- 1- 1 TO 1986-12- 1

PLANT NAME: SQUAW RAPIDS UPSTREAM RESERVOIR NAME: TOBIN LAKE START FOREBAY ELEVATION: 313.59 (M) YEAR-1986

PERIOD NUMBER	STARTING DATE	ENERGY PRODUCED (MW)	ENERGY PRODUCED (GWH)	TOTAL INFLOW (M3/S)	PLANT DISCHARGE (M3/S)	SPILL (M3/S)	END FOREBAY ELEVATION (METERS)	NET HEAD (METERS)	STORAGE LEFT (DAYS)	EFFICIENCY FACTOR (MW/(M3/S))	CAPACITY FACTOR
1	JAN 1	115.36	85.83	359.88	404.36	0.00	313.15	33.39	4.56	0.29	0.40
2	FEB 1	102.79	69.07	343.65	367.14	0.00	312.94	33.16	4.02	0.28	0.36
3	MAR 1	94.96	70.65	284.86	340.70	0.00	312.33	32.81	2.57	0.28	0.33
4	APR 1	112.02	80.66	504.01	402.01	0.00	313.36	32.87	5.11	0.28	0.39
5	MAY 1	143.70	106.92	465.60	514.09	0.00	312.85	32.86	3.80	0.28	0.50
6	JUN 1	148.52	106.94	599.68	529.76	0.00	313.47	32.88	5.39	0.28	0.52
7	JUL 1	263.97	196.39	952.58	946.17	0.00	313.46	32.39	5.37	0.28	0.92
8	AUG 1	165.26	122.95	592.22	588.49	0.00	313.41	33.03	5.24	0.28	0.58
9	SEP 1	110.71	79.71	392.12	389.16	0.00	313.36	33.44	5.11	0.28	0.39
10	OCT 1	172.86	128.61	626.58	613.60	0.00	313.45	32.95	5.34	0.28	0.60
11	NOV 1	133.61	96.20	483.54	469.63	0.00	313.58	33.37	5.68	0.28	0.47
12	DEC 1	123.36	91.78	428.10	428.00	0.00	313.58	33.53	5.68	0.29	0.43

PLANT HYDRO ENERGY FOR YEAR 1986 = 1235.6883 (GWH)

$$\text{DAYS} = \frac{\text{STORAGE LEFT (CUBIC METERS)}}{\text{(MAXIMUM PLANT DISCHARGE)*3600.0*24.0}}$$

ALL THE RESULTS ARE AVERAGE QUANTITIES

(1420d)

WATER USE ANALYSIS MODEL

J.D. Rogers
Water Planning and Management Branch
Environment Canada
September 27, 1988

Introduction

The Water Use Analysis Model (WUAM) was developed by Acres International Limited under the direction of Inland Waters Directorate (IWD) for the Canada Department of Energy, Mines and Resources (EMR). The objective was to provide a tool to assess changes in water use and to estimate their impacts on surface water supplies. The WUAM uses economic and demographic forecasts to estimate future water use. Water balance calculations, using historical hydrometeorological data points out the imbalances between water demand and water supply.

Model Applications

The Water Use Analysis Model has been designed to answer the "what if" questions relating to multisectoral growth to water availability. WUAM can evaluate the water resource impacts of site-specific projects, of economic and demographic growth, of changing water use coefficients and of various remedial measures such as storage or transfers.

Data Requirements

Unlike other models such as the Water Resources Management Model (WRMM) or the Multi-Reservoir Simulation (HiSpeed) Model, the WUAM must be provided with demographic and economic numbers, water use coefficients, and detailed data on irrigation projects and on hydro and thermal power plants. In general the WUAM needs the data required to calculate the water demands used in water balance calculations. For the WRMM and the HiSpeed Model, water demands are estimated outside the model. The water balance calculations use data similar to the other models. The general data requirements for the WUAM are listed below.

A. Water Use (Demand)

1. Municipal/Rural Domestic/Agriculture (non irrigation)
   - population
   - consumption percentage
   - monthly distribution
   - groundwater fraction

2. Irrigation
   - irrigated area
   - system delivery efficiency
   - water salinity/max & min allowable soil salinity
   - fraction of the water from groundwater
   - area under each crop type
   - soil types and soil parameters
   - irrigation application efficiency
   - crop water use coefficients
   - monthly evapotranspiration rates

3. Industrial
   - water use coefficient (per $ million of output per year)
   - per cent $ contribution to economic region
   - water consumption as a per cent

4. Power production
   A. Thermal Power
      - plant fuel type
      - cooling type
      - condenser types and number
      - intake and consumption coefficients
   B. Hydro-power
      - average hydraulic head
      - turbine efficiency installed capacity
      - flow factors

B. <u>Water Supply</u>

1. Monthly mean natural flows at each study point (node) with a common record length
2. Monthly mean gross potential evaporation
3. Monthly precipitation totals
4. Flows constraints (i.e. minimum required flows)
5. Reservoir description including elevation, flooded area and storage relationships
6. Water transfers into/out of the basin at each study point
7. Reservoir operation rule curves including minimum, maximum and target outflows, and minimum/maximum levels

Special Features

Economic and demographic growth projections are used to forecast demands in each water use category. These scenarios can also include water price/demand interactions, efficiency changes and site-specific projects. Instream water use requirements are met by specifying minimum flows at each study point.

The WUAM is divided into a number of modules which may be called upon to calculate water demands or perform the historical water balance simulation. Of particular interest are the modules which calculate irrigation, thermal power and hydro-power demands.

The irrigation demands are estimated on a monthly basis over the historical period using one year of evpotranspiration rates and monthly precipitation totals over the historical period. Other factors such as delivery efficiency, off stream storage, crop types and irrigation application efficiency are used to estimate irrigation demands. A separate file of gross irrigation diversion and return flows is produced for each irrigation area or district. Irrigation water salinity, soil types and depletion equations are included to point out any water quality or soil productivity problems.

The characteristics of thermal power plants such as plant capacity, intake and consumption coefficients are use to estimate the water requirements of each power plant. The hydro-power station characteristics, such as installed capacity, average head and efficiency, are used in conjunction with reservoir outflows to estimate electric power production. Water intake values have been related to power plant capacity rather than power production.

Limitations

The Water Use Analysis Model's major limitations are its complexity and that it is relatively untested. Minor hinderences relate to its "user friendliness", poor documentation (both internal and user) and lack of standard input formats.

Any model which combines economic growth, population change, irrigation and power plant use and water balance calculations will be very complex. WUAM, unlike many water balance models, needs a multi-disciplinary approach. Although the multi-disciplinary approach appeals to most planners, the model is more difficult to setup, calibrate, and run. Whether the answers are closer to the truth is debateable.

WUAM is still being developed and changed, therefore the model has not been extensively tested in real world situations. Most tests of WUAM have been to show the capabilities of the model rather than to give usable results.

The WUAM was developed to be "user friendly", which involves answering questions during the model setup and operation. This type of input may be advantageous to a new user but batch operation would be less time consuming for the more experienced user.

Poor documentation is a longstanding problem of computer models. WUAM is no different. However internal file documentation (i.e. headers on flow arrays) and standard input formats, to match those of the Sask - Nelson Basin Board Study (SNBB), Water Survey of Canada or PFRA, would make data input and model operation easier and more efficient.

QU'APPELLE RIVER BASIN HYDROTECHNICAL STUDY
Physical Data for the REGUSE Model

T.P.S.SANDHU*

1. INTRODUCTION

Physical data for the REGUSE Model consist of elevation-discharge data for existing outlet structures, tailwater rating curves, desirable maximum and minimum lake levels, lake elevation-storage data, travel times, historical lake levels and flow data for the Qu'Appelle River system. The hydraulic computations for various structures were related to the determination of discharge rating curves for lake outlets including stop-log weirs, vertical sluice gates, radial gates and low-level conduits under variable tailwater conditions. Model data input was also required for tailwater rating curves including backwater effect of existing tributaries and lakes downstream of these structures.

Analysis was undertaken to determine diversions in and out of Last Mountain Lake from the Qu'Appelle River near Lumsden. Information is also presented on maximum and minimum desirable lake levels and time of travel from headreach to the downstream end of the basin. Historical lake levels and flow data were based on Water Survey of Canada data files.

2. PHYSICAL DESCRIPTION OF STRUCTURES

Releases from Lake Diefenbaker into the Qu'Appelle River are made through Qu'Appelle Dam outlet works. The Saskatchewan Water Corporation operates this structure. Water is released during summer to replace evaporation losses and improve the water quality of Eyebrow Lake. Also natural supplies to Buffalo Pound Lake are augmented to meet various demands placed upon it. Thus releases from Lake Diefenbaker are made to maintain a suitable water supply in both lakes.

Craven Dam is the only one river structure without any reservoir with six stop-log bays and two vertical slide gates. Lake control structures exist on Buffalo Pound Lake, Mountain Lake(Valeport), Echo Lake, Katepwa Lake, Crooked Lake and Round Lake. Buffalo Pound Lake has three stop-log bays and a low level gated conduit. Valeport Structure has eight stop-log bays. At Echo Lake, only one bay is controlled by a vertical sluice gate and other seven are stop-log bays. There are two types of structures at the Katepwa Lake outlet, a fixed crest weir andf a structure with two radial gates andf a vertical slide gate. Crooked Lake and Round Lake structures each have nine stop-log bays.

3. HYDRAULIC SUB-ROUTINES FOR DISCHARGE COMPUTATIONS

Stop-log Weirs

There are two modes of operation for stop-log weir structures on the Qu'Appelle River system. For a typical spring or high flood season, structures are fully open to pass flood flows and to save them from damage due to floating ice. During the summer or low flow season, structures are partially or fully closed and flow is either over weir or through partial bay openings. Discharges for these conditions were computed for the REGUSE Model input. For actual operation, these structures are adjustable and it is assumed that gate openings would be adjusted to achieve the required discharge.

---

*Senior Planning Engineer, Water Planning and Management, Environment Canada, Ottawa, Ontario, October 1988.

Open Mode

As shown in Figure 1, when structure is fully open, there is minimum height (P_1) of stop-logs left in place. If these are completely taken out like in the case of Echo Lake, sharp crest is eliminated and broad crest weir hydraulic criteria applies to the rating curve.

$P_1 = WC_1 - WI$

where: WC_1 = weir crest at structure fully open

WI = weir invert level

Effective Head, $H_1 = (D-P_1)$ if $TWL \leqq WC_1$

Effective Head, $H_1 = (E-TWL)$ if $TWL > WC_1$

D = upstream depth
E = upstream level
TWL = tailwater level

For sharp crest, weir discharge coefficient $C_x = 1.782 + 0.24\ (H_1/P_1)$

(Ref. 1, U.S.B.R. formula in metric units)

For broad crest, weir discharge coefficient $C_x = 1.72$

Maximum weir discharge $Q_{wmax} = C_x\ L_e H_1^{3/2}$ L_e = effective weir length

For practical purposes, contraction effect was considered small and actual and effective lengths of weir were assumed to be the same. Approach velocities are small and it is assumed that head correction is not needed for such a small variation. Typical calculations for maximum discharges for different tailwater conditions are given in the attached Table.

Closed Mode

In closed mode, stop-logs are normally in place up to the full supply level.

Weir height, $P_2 = WC_2 - WI$

where: WC_2 = weir crest at structure fully or partially closed.

Effective head, $H_2 = D - P_2$ if $TWL \leqq WC_2$

Effective head, $H_2 = (E-TWL)$ if $TWL > WC_2$

Weir coefficient, $C_y = 1.782 + 0.24\ (H_2/P_2)$

(Ref. 1, U.S.B.R. formula in metric units)

Minimum Weir Discharge, $Q_{wmin} = C_y\ L_e H_2^{3/2}$.

Again approach velocity and contraction effects are assumed to be negligible. Typical values for minimum discharges under variable tailwater conditions are given in the attached Table. At higher levels water can also go around structure and flow over road. This situation was considered as a weir flow and total flow was calculated from addition of flow through structure and road weir flow.

Vertical Sluice Gates

For free flow conditions, when there is no tailwater effect, discharge through vertical gates was calculated as follows:

$Q = CL\ h\ \ [2g\ (y_1)]^{1/2}$

where: C = gate discharge coefficient (Ref. 2, Figure 17-38)
The velocity head was considered negligible as the approach velocity in the lake above structure is low. Also as shown in Figure 2,
L = gate width
h = gate opening
g = gravity constant
y_1 = upstream water depth

The outflow of the gate may be either free or submerged, depending on the tailwater depth. For submerged flow or if the tailwater affects flow, effective head of difference between the upstream and downstream depths was used.

Thus, $Q_S = CLh\ [2g\ (y_1 - y_2)]^{1/2}$ or $Q_S = CLh\ [2g\ (y_1 - y_3)]^{1/2}$

where: y_2 and y_3 = downstream tailwater depths.

For sluice gate with combined overflow and underflow, total flow was based on upstream depth y_1 and downstream depth y_0 (Figure 2).

Calculated values for flows through sluice gates. These values were added on to the weir flow to get total discharge through these structures.

Radial Gates

Radial gates hydraulic concept is shown in Figure 3. Only one structure on Katepwa Lake has two radial gates. Top of the gates when fully open will be at elevation 479.44 m and for lake levels up to this elevation flow will be through gates. Discharge calculations were based on net head acting on gates. Discharge coefficients with values in the range of 0.5 to 0.6 were based on Ref. 2 (Figures 17-39). Discharge was calculated by the following relationship.

$Q = CL\ h\ \ (2gH)^{1/2}$

where: C, L, h and g are the same terms as mentioned before.
H = net head acting on the gate.

For lake levels higher than 479.44 m, water will flow through gates as well as above gates. The flow above gates was based on weir formula and is added on to the radial gate flow to get the total flow through structure.

Low Level Conduits

Only Buffalo Pound Lake and Katepwa Lake structures have low level gated corrugated metal pipes. Discharge through these conduits is small. For Buffalo Pound Lake, there is a maximum of about 2.4 cubic metre per second when gate is fully open at high flow levels. Rating curve for Buffalo Pound has been developed by Saskatchewan Water Corporation from actual field measurements. In the calculation of discharge through structure this small value through conduit was included in the weir flow.

Low level conduit at Katepwa Lake is only of 0.76 m diameter and maximum discharge through it is about one cubic metre per second.

4. DOWNSTREAM TAILWATER RATING CURVES

Tailwater levels downstream of structures are dependent on three flow conditions normally encountered on the Qu'Appelle River System. Firstly, tributaries entering the Qu'Appelle River just downstream of structures influence flow capacity of outlets. Secondly, backwater effect of the existing lakes located immediately downstream of structures raise tailwater levels. In the third situation, natural tailwater levels occur as a result of flow through the structure itself. Tailwater Rating Curves under these three conditions have been developed in the following sections.

Backwater Effect of Downstream Tributaries

Lake structures affected by various tributaries are:

Buffalo Pound Lake	- Moose Jaw River
Last Mountain Lake (Valeport)	- Qu'Appelle River
Katepwa Lake	- Indian Head Creek
Crooked Lake	- Ekapo Creek.

Buffalo Pound Lake Structure tailwater rating curve was developed from Saskatchewan Water Corporation Inflow-Outflow curves for Buffalo Pound Lake including downstream effect of Moose Jaw River flows. Using these curves, effective head under backwater conditions was calculated for lake levels varying from 507.80 m to 509.70 m.

By subtracting the effective head from lake levels, downstream tailwater level was calculated. For levels higher than 509.70 m, gauge data for the Water Survey of Canada Hydrometric Station at Qu'Appelle River below Moose Jaw River (05JG007), was used (Ref. 3). Using the water surface slope from station to the structure, tailwater curve was extended to 510.0 m.

Valeport structure outflows and inflows (structure being used in both directions) are affected by Qu'Appelle River flows. Detailed diversion analysis presented in separate section deals with inflow and outflow magnitudes. However, when there are outflows for tailwater purposes, curves for Qu'Appelle above Craven prepared by Saskatchewan Water Corporation (Ref. 3). Tailwater levels downstream of Katepwa and Crooked Lakes were provided by Saskatchewan Water Corporation.

Katepwa Lake Backwater Effect on Echo Lake

A regression analysis of the Echo Lake and Katepwa Lake levels was carried out by using the SPSS computing program. A period of lake level records from 1972-85 is available for both lakes. Lake levels at WSC gauging station for Echo Lake at Fish Hatchery (Stn. O5JK005) and Katepwa Lake at outlet weir (Stn. 05JL004) were used only for open water conditions where there was no effect of ice. Both linear and non-linear regressions were performed using the SPSS Computer Program (Ref. 4). The linear regression using level differences between the two lakes as dependent variable and upstream Echo Lake as independent variable gave better correlation results (R=0.83265). The following regression equation was obtained from this analysis.

$$K_L = 289.09364 + 0.39518375\ E_L$$

This equation was used to develop the corresponding lake levels for two lakes. To evaluate the tailwater rating curve for Echo Lake structure, effective head acting on the Echo structure was computed from level differences between two lakes. It was determined that, when the downstream Katepwa Lake level rises above 478.87 m, Echo Lake weir becomes submerged and outflows are reduced.

Tailwater as a Result of Flow through Structure

When there is no downstream tributary or lake effecting onflows from a structure, tailwater is dependent on the downstream flow area and magnitude of flow. Craven and Round Lake Structures are two examples of this situation. Rating curve for Qu'Appelle River below Craven Dam is available for Water Survey of Canada Hydrometric Station No. 05JK002. Tailwater Rating curve for Round Lake was provided by Saskatchewan Water Corporation (Ref. 5).

5. LAST MOUNTAIN LAKE DIVERSION ANALYSIS

The Valeport Structure on Last Mountain Creek is being used for diverting water into Last Mountain Lake from the Qu'Appelle River during high flows. This water is released to bring the lake levels down to desirable limits or augment flows downstream of Craven during summer months. Thus the structure is being used in both directions.

Diversion into Last Mountain Lake

With structural control at Craven, Qu'Appelle River flows could be directed into Last Mountain Lake. However, during spring flooding when Craven structure is left completely open, a natural split of flows occurs at such times. Valeport structure is opened to allow diversion into Last Mountain Lake. Water Survey of Canada Station on Last Mountain Creek near Craven (Stn. No. 05JH006) has records from 1968 to 71 and 1973 to 77. Long-term records (1944-85) are also available for Qu'Appelle River near Lumsden (Stn. No. 05JF001). Inflows into Last Mountain Lake are recorded as negative flows in WSC records. These were separated from outflows and regression analysis of the Qu'Appelle vs Last Mountain Creek inflows was carried out using SPSS Computing Program (Ref. 4). Both records indicated high degree of correlation (R = 0.95789) and the following equation was obtained in log form.

$$\ln[Q_M] = -0.53176389 + 0.96343188 \ln[Q_L]$$

where: Q_M = Last Mountain Creek inflow near Craven

Q_L = Qu'Appelle River flow at Lumsden.

This equation was used to compute Qu'Appelle split flows or diversion into Last Mountain Lake.

Outflows from Last Mountain Lake

Outflows from Last Mountain Lake via Valeport Structure occur mostly during the months of March to September. The Regression Analysis was undertaken for both the linear and non-linear variations between Qu'Appelle River flows at Lumsden and outflows from the lake. Both regressions indicated practically low correlation (R = 0.12401 for linear and 0.31566 for non-linear). So the regression equations obtained cannot be applied to achieve meaningful results. One main reason for this kind of variation may be the fact that in the past there was no set control pattern for releasing water from Last Mountain Lake. A plot of flows at two stations without any effect of ice conditions indicated releases from Last Mountain Lake were made when Qu'Appelle River flows at Lumsden dropped to about 14.0 m^3/s.

6. LAKE REGULATION AND TRAVEL TIMES DATA

Lake Regulation

In the past and for that matter even the current operating practice for structures is aimed at keeping the normal operating range for Qu'Appelle Lakes within a desirable range. Elevation-Storage data and information on maximum and minimum normal operating ranges were provided by Saskatchewan Water Corporation. These levels reflect an acceptable compromise where maximum flooding levels are tolerated and minimum low levels are allowed to pertain on these lakes. Lakes can only be lowered to the point where the physical characteristics of the structure allow water to continue to flow out of them.

Travel Times

The basic information available on travel time was originally gathered by Water Survey of Canada in 1973. This was subsequently reproduced in graphic form by Saskatchewan Water Corporation (Figures III-47,48,49 Reference 3)

For other reaches where travel times were not available, HEC-1 Computing Model was used to estimate the travel times by reservoir routing. Total travel time through the Qu'Appelle River system from headreach to the downstream end of the basin is about 23 days.

7. LAKE LEVELS AND FLOW DATA

Historical lake levels and flow records were obtained from Water Survey of Canada hydrometric data files. In those instances where flows for small ungauged tributaries and local areas were not available drainage area prorating techniques were used for a gauging station close to the area. Ungauged flows are taken as percentage of the gauged flow.

REFERENCES

1. Handbook of Hydraulics, H.W. King and E.F. Brater, McGraw-Hill Book Company, New York, 1963.

2. Open-Channel Hydraulics, V.T. Chow, McGraw-Hill Book Company, New York, 1959.

3. Qu'Appelle Conveyance Study Handbook of Hydraulics, Appendix 1, Saskatchewan Environment, 1975.

4. Statistical Package for the Social Sciences (SPSS), N.H. Nie, C.H. Hull, J.G. Jenkins, K. Steinbrennet and D.H. Bent, McGraw-Hill Book Company, New York, 1970.

5. Operation of the Qu'Appelle River System, Saskatchewan Water Corporation, Hydrology Branch, Moose Jaw, Saskatchewan, 1974.

ELEVATION-DISCHARGE DATA FOR
BUFFALO POUND STRUCTURE

CODE	STAGE	STORAGE	DISCHARGE (m3/s)		TWL	
	(m)	(m3)	MAX	MIN	(m)	
1115	507.80	49462.E3	0.00	0.00	507.80	BUFFALO
1115	508.11	53273.E3	5.17	0.00	507.80	POUND
1115	508.72	70930.E3	27.54	0.00	507.80	LAKE
1115	509.32	87558.E3	60.75	0.00	507.80	
1115	509.41	90215.E3	66.59	0.80	507.80	
1115	509.94	105379.E3	105.59	14.67	507.80	
1115	510.55	123655.E3	159.14	41.94	507.80	
1115	510.68	127601.E3	178.21	55.38	507.80	
1115	511.00	137313.E3	244.55	109.16	507.80	
1115	511.45	149756.E3	372.22	214.98	507.80	
7777						
1115	507.80	49462.E3	0.00	0.00	508.00	BUFFALO
1115	508.11	53273.E3	1.08	0.00	508.00	POUND
1115	508.72	70930.E3	18.82	0.00	508.00	LAKE
1115	509.32	87558.E3	48.55	0.00	508.00	
1115	509.41	90215.E3	53.90	0.80	508.00	
1115	509.94	105379.E3	89.88	14.67	508.00	
1115	510.55	123655.E3	140.45	41.94	508.00	
1115	510.68	127601.E3	158.86	55.38	508.00	
1115	511.00	137313.E3	223.55	109.16	508.00	
1115	511.45	149756.E3	348.88	214.98	508.00	
7777						
1115	507.80	49462.E3	0.00	0.00	508.50	BUFFALO
1115	508.11	53273.E3	0.00	0.00	508.50	POUND
1115	508.72	70930.E3	3.07	0.00	508.50	LAKE
1115	509.32	87558.E3	23.02	0.00	508.50	
1115	509.41	90215.E3	27.07	0.80	508.50	
1115	509.94	105379.E3	55.73	14.67	508.50	
1115	510.55	123655.E3	98.28	41.94	508.50	
1115	510.68	127601.E3	115.00	55.38	508.50	
1115	511.00	137313.E3	175.55	109.16	508.50	
1115	511.45	149756.E3	295.72	214.98	508.50	
7777						
1115	507.80	49462.E3	0.00	0.00	509.00	BUFFALO
1115	508.11	53273.E3	0.00	0.00	509.00	POUND
1115	508.72	70930.E3	0.00	0.00	509.00	LAKE
1115	509.32	87558.E3	5.43	0.00	509.00	
1115	509.41	90215.E3	7.92	0.80	509.00	
1115	509.94	105379.E3	28.48	14.67	509.00	
1115	510.55	123655.E3	62.67	41.94	509.00	
1115	510.68	127601.E3	77.67	55.38	509.00	
1115	511.00	137313.E3	134.03	109.16	509.00	
1115	511.45	149756.E3	247.72	214.98	509.00	
7777						
1115	507.80	49462.E3	0.00	0.00	509.50	BUFFALO
1115	508.11	53273.E3	0.00	0.00	509.50	POUND
1115	508.72	70930.E3	0.00	0.00	509.50	LAKE
1115	509.32	87558.E3	0.00	0.00	509.50	
1115	509.41	90215.E3	0.00	0.00	509.50	
1115	509.94	105379.E3	8.83	8.71	509.50	
1115	510.55	123655.E3	33.86	32.86	509.50	
1115	510.68	127601.E3	47.05	45.72	509.50	
1115	511.00	137313.E3	99.08	98.14	509.50	
1115	511.45	149756.E3	206.85	202.17	509.50	
7777						
1115	507.80	49462.E3	0.00	0.00	510.00	BUFFALO
1115	508.11	53273.E3	0.00	0.00	510.00	POUND
1115	508.72	70930.E3	0.00	0.00	510.00	LAKE
1115	509.32	87558.E3	0.00	0.00	510.00	
1115	509.41	90215.E3	0.00	0.00	510.00	
1115	509.94	105379.E3	0.00	0.00	510.00	
1115	510.55	123655.E3	12.42	12.23	510.00	
1115	510.68	127601.E3	23.61	23.27	510.00	
1115	511.00	137313.E3	70.97	71.58	510.00	
1115	511.45	149756.E3	172.57	170.34	510.00	
8888						

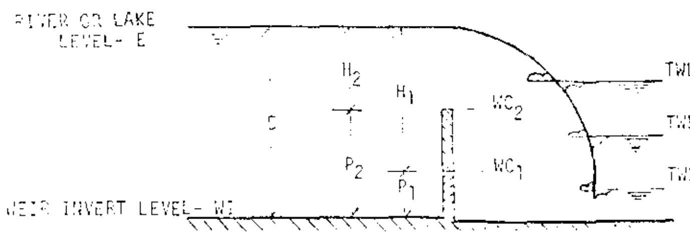


FIGURE 1 : STOP-LOG WEIR CONCEPT

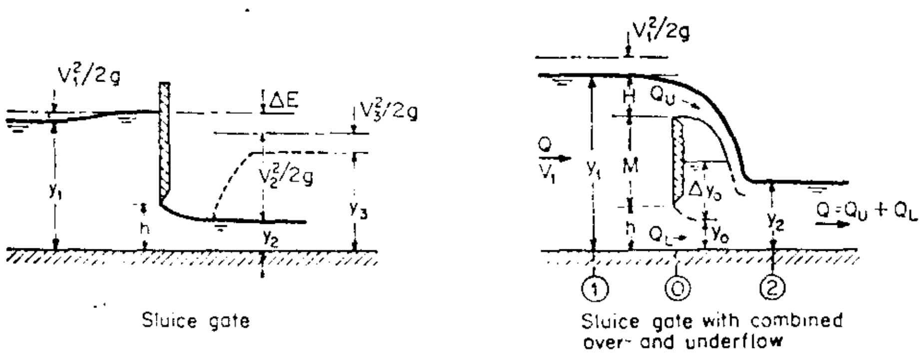


FIGURE 2 : VERTICAL SLUICE GATE HYDRAULIC CONCEPT

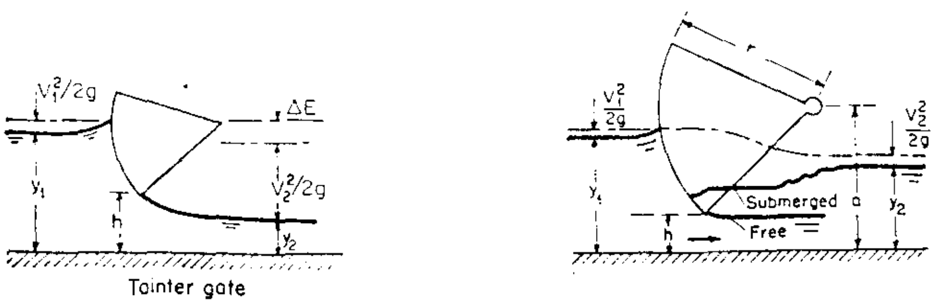


FIGURE 3 : RADIAL GATE HYDRAULIC CONCEPT

HEURISTICS AND NETWORK FLOW ALGORITHMS FOR MULTI-RESERVOIR SYSTEM REGULATION

Donald W. Farley[1]; Maurice Sydor[2]; Gerald E. Brown[3]

Abstract

Effective modelling of a multi-reservoir/channel network must simulate a rational operating policy which recognizes several often conflicting needs such as flood control, navigation, flow augmentation, recreational requirements, environmental constraints, energy production, and identified water uses. A generalized model employing a network flows algorithm and a heuristic database is being tested on the Qu'Appelle River basin in Canada. The heuristic database consisting of knowledge-based information relating to the various identified water resource needs in a basin is refined through runs of the model on supercomputer facilities for extensive historical periods of record. The basis of the model's solution is the simultaneous consideration of the hydrological events (runoff/tributary inflows and evaporation loss), hydraulic characteristics (stage/storage data, routing, etc.) and the heuristic data for a specified time horizon. The time horizon being defined as being the entire period for which a simultaneous solution is generated, can be divided into a variable number of equal unit-time periods (presently limited to 10). The selected unit-time period can vary from one day to one month. Real-time applications using either short- or long-term time horizons can be run with hydrological forecasts and climatological data. The use of daily data and extended time horizons permits channel routing to be included in the solution process. Strategies are recommended for running the model on supercomputers, scalar mainframes and microcomputers.

Heuristics and Reservoir Regulation

In the multi-reservoir regulation modelling problem, the runoff and channel flow components can, with varying degrees of difficulty, be simulated through the use of regression, conceptual and physically based models. The control structure operations component of the

[1]Chief, Water Management Systems Division, Water Planning and Management Branch, Inland Waters Directorate, Environment Canada, Ottawa, Ontario K1A 0H3
[2]Head, Water Management Modelling Section
[3]Senior Econometrician

problem involves choosing among a set of alternative actions. One, or several of these alternatives will be more desirable than the others from the standpoint of some criterion. Thus, flow regulation modelling, in both the planning and operational modes, must include a rational decision making technique which takes into account the temporal and spatial distribution of the basin water supplies, the water use demands, and the hydraulic characteristics of the control structures.

Rational operating policy must deal with several often conflicting needs such as flood control, navigation, flow augmentation, recreational requirements, environmental constraints, energy production and identified water uses. All of these needs require subjective judgement on such issues as: what identified needs have priority, and does the priority vary over the annual cycle? What is likely to be the energy demand several months hence? What is the best compromise policy between low flow augmentation and flood protection in coping with uncertain future water supply forecasts? Do moral obligations overrule economic gain? An interesting example of this latter issue occurs in the Ottawa River Regulation Modelling System (Courbu, Lau 1984) where the objective function in the linear programming solution incorporates variable coefficients for energy value and flood damages. This weighting technique provides the decision makers with a trade-off curve to be used in a team approach to identify the best compromise solution.

The above issues and many other similar ones which pervade water resources problems can be conveniently assigned to the obscure domain of heuristics. Fordyce et al, 1987 refer to heuristic reasoning as meaning that one brings to bear as much intuition, and as many plausible arguments, as possible on problems which are either computationally intractable, or for which inadequate theory exists. The application of heuristics to operational research (OR) models is unappealing to the purer mathematicians. Heuristics deal with the messy details that do not fit into our mathematical models but are very helpful when these purely mathematical models are inadequate. Heuristics are usually associated with approximate rather than exact answers. They are rules derived from experience with manual methods. In dealing with water resources problems we are compelled to resort to heuristics in order to get the answer within an acceptable period of time when algorithmic methods do not work. However, heuristics are awkward to report in mathematical modelling papers. Like movies from someone else's vacation, the accounting of a bird watching expedition in the Gatineau Hills, and recitations of other people's surgical operations, heuristic details are very interesting to the people immediately concerned, but they are a crashing bore to everyone else. However, Tingley, 1987 reports that the acceptance of heuristics in management science/operational research (MS/OR) has grown with increasing theoretical justification. Heuristics has become a significant aspect of artificial intelligence (AI).

Convergence of OR and AI Techniques

Most experts (Tingley, 1987; Simon, 1987 and Phelps, 1987) agree that there is a growing similarity between OR and AI, both in the problems they face and in the techniques they use: both the OR and AI approaches build models; both use heuristic procedures in the absence of optimal ones; both use computer implementations; both employ interdisciplinary teams. Intelligent systems also have strong roots in the OR field. Fordyce et al, 1987 refer to the text "Principles of Operations Research" by Harvey Wagner, 1969, which discusses the concepts of goal-seeking simulations and synthetic intelligence.

Streamflow regulation problems involve significant heuristic elements, but in most cases the major portion of each problem can be appropriately and conveniently defined in terms of the physical laws of continuity and conservation of mass. Linear programming and network flows algorithms can be used to accurately represent these laws with the heuristic elements represented in the knowledge base through target bounds and arbitrary cost coefficients. In fact, Mark Houck, at the 1987 Round Table Discussion on New Trends in Water Resources Systems Analysis (University of Manitoba, 1987) states that linear programming and network flows optimization techniques can perform excellently in replicating the informed judgement of experts for water resource studies.

Network Flow Models

Network flow algorithms have been developed to handle a special class of linear programming problems. These problems may each be described as a network whose links carry flow. Network algorithms take advantage of this special structure to produce a least cost optimal solution much more quickly with less core storage required, and virtually no round-off error in comparison with general linear programming codes. The particular algorithm used in the application described in this paper is referred to as the "out-of-kilter" algorithm and was obtained from Woolsey and Swanson (1975). A more powerful out-of-kilter algorithm referred to as SUPERK (Barr et al, 1974) is currently being tested in the subsequent applications discussed in this paper, and is recommended for large network problems.

In river basins where regulation and water use problems exist, the interconnecting links or arcs represent channel discharge and reservoir storage at various locations within the basin. Flow movement is from node to node and is restricted by a maximum channel or reservoir capacity (upper bound), and minimum discharge or level requirements (lower bound). Flow along each arc is the flow per unit-time period. For example, if a unit-time period of one week was selected, the flow level would be the total volume of flow in m^3 for one week. Auxiliary routines are required for converting flow volumes to m^3/s and storage volumes to reservoir stage. The notion of time can be introduced into the flow network by considering storage as flow from a reservoir node in one unit-time period to the corresponding node in the next period. The network flow diagram (Figure 1) equates vertical flow with channel flow from one river station to another, and horizontal flow with reservoir storage flow from one time period to the next. The additional arcs represent initial storage and flow conditions. The return path (not shown in diagram) between the sink and the source node ensures that flow continuity is achieved at all nodes throughout the network.

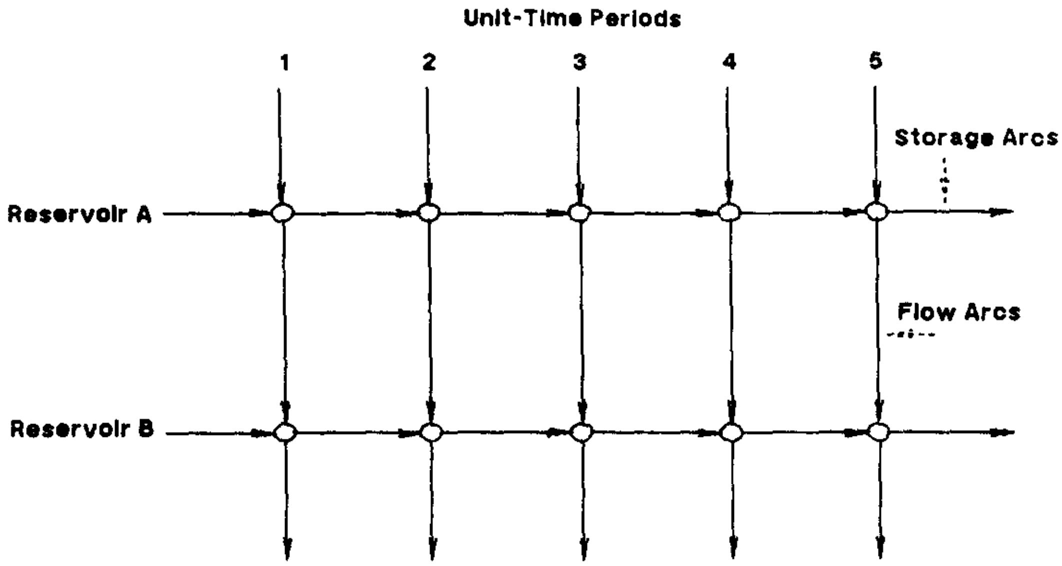


Figure 1: Space/Time Network Diagram for a Simple Basin
with Storage and Channel Flow

Network Flow Routing

The introduction of space and time allows the important concept of hydraulic routing to be incorporated into the network. Channel travel time is needed to allow flow to move from one river location to another. In a space/time flow network (Figure 2), routing is accomplished by means of a diagonal arc which allows vertical flow from an upstream station to a downstream one, but delays the flow for one or more horizontal unit-time periods. For the purpose of clarity, Figure 2 shows a fan radiating from only a single node, whereas there is actually a similar fan radiating from each node.

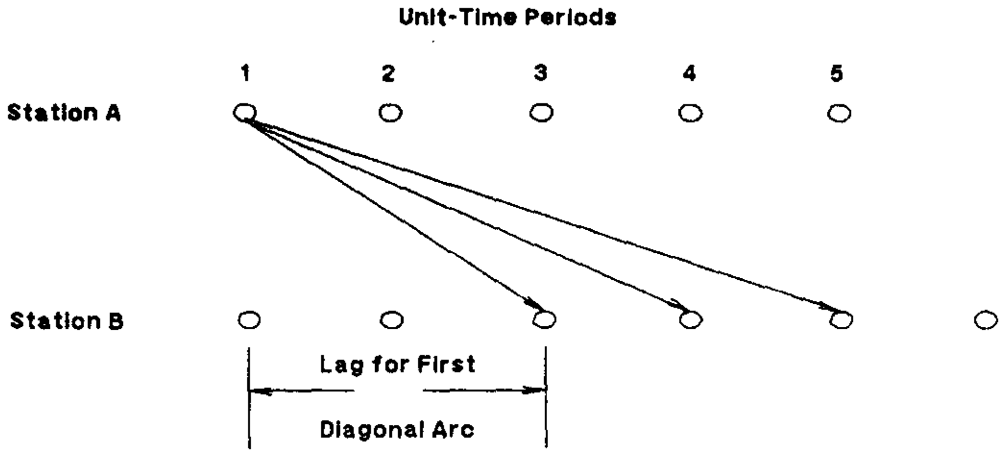


Figure 2: Simplified Diagram Showing Fan of Routing Arcs
for the First Time Period

In its simplest form, the concept of routing can be reduced to a single travel time (diagonal) arc which carries all the unit-time period flow between two channel stations. The values of the upper and lower bounds for this single arc are set to allow the total range of discharges from the upstream station to move downstream. Hydraulic routing can be represented in the network by a fan of arcs as shown in Figure 2. The flow leaving the upstream station arrives at the second station through several arcs, each delayed by successive discrete unit-time periods. The flow in each arc is calculated to be a function of the total flow leaving the upstream station. The upper and lower bounds associated with each arc in the flow network must be set before the network flow problem is presented to the out-of-kilter optimizer. A routine external to the optimizer determines the amount of flow leaving the upstream node of the reach. The routine calculates, based on the hydraulic routing characteristics of the channel, the proportion of this flow to be allocated to each of the routing arcs. The upper and lower bounds associated with the routing arcs are then adjusted in such a way that the required flow levels are achieved when the network flow problem is again presented to the optimizer. An iterative loop is used to first: obtain a flow solution from the optimizer using an initial flow network, second: calculate the differences between the current solution and the previous one, third: calculate new bound values if this difference is outside a specified tolerance. In the last step, the new bound conditions are imposed on the flow network and the iterative loop is closed by returning the network to the optimizer so that a new flow solution can be found. The REGUSE model, described in the next section, uses this iterative method to generally impose a number of hydraulic conditions on the solution.

The hydraulic relationships which govern streamflow in a channel, require that an empirical routing equation based on calibration coefficients derived from actual flow data be used to calculate the flow in each of the routing arcs. In the REGUSE model, the present routing routine calculates the flow in each arc based on three routing parameters. These are the number of arcs in the arc fan, the time shift, in discrete numbers of unit-time periods, of the first arc and the flow to be allocated to each arc as a simple fixed percentage of total flow leaving the upstream station. Figure 3 shows a typical calibration result for the Qu'Appelle River Basin in Saskatchewan using this method. In this calibration the first of the three routing arcs used is lagged one unit-time period and the fixed flow percentages are 35%, 35% and 30%. The calibration is based on two-day flow data. These data consisted of measured channel and tributary flow plus estimated local inflows, evaporation losses and water demands.

The REGUSE Model - A Network Flow Application

REGUSE is a mathematical model designed to assist basin managers in real-time regulation of rivers and reservoirs and in carrying out various types of basin planning studies. The basis of the model's solution is the simultaneous consideration of all hydrologic events (runoff/tributary inflows and evaporation loss), hydraulic characteristics (stage/storage data, routing, etc.) and economic water use data for a specified time horizon. A time horizon is defined as the entire period in which a simultaneous solution is generated. The time

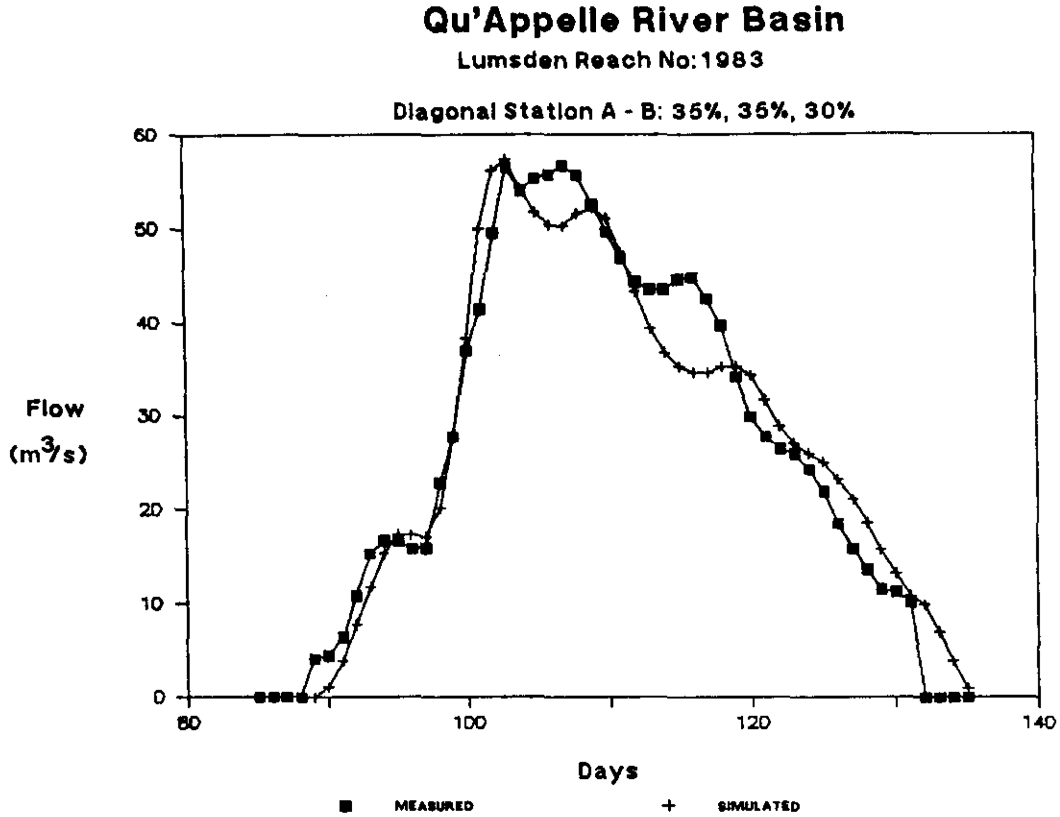


Figure 3: Calibration of the Lumsden Reach for Network Flow Routing

horizon can be divided into a specific number of equal unit-time periods. The model has been tested with 10 unit-time periods in the time horizon. The unit-time period can vary from one day to a month or more (see Figure 4). The model can be employed for planning studies in the traditional simulation fashion of unit-time period by unit-time period over the total specified simulation time, or it can be run as a series of chained short-time horizons with each time horizon's duration spanning hydrological forecasting capabilities. Real-time applications using either short- or long-time horizons can be run with hydrological forecasts and climatological data.

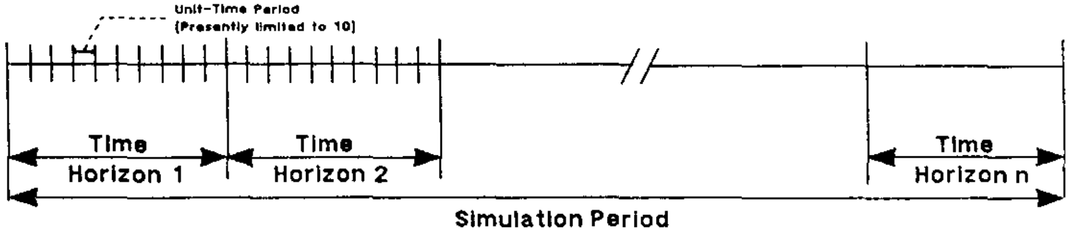


Figure 4: Time Definitions

The model can automatically generate a complete flow network for the basin by combining the physical hydraulic and hydrologic characteristics of the basin with flow data supplied by the user. The out-of-kilter optimization algorithm is used to compute the final solution. The network flow approach is ideally suited for basin management applications because the linkages between each station in the basin network can be assigned upper and lower bounds which relate directly with the flow and storage capacities of an actual river or reservoir system. Penalty coefficients are used to discourage undesirable high or low flows or levels in channels and reservoirs respectively by the use of auxiliary arcs (Figure 5).

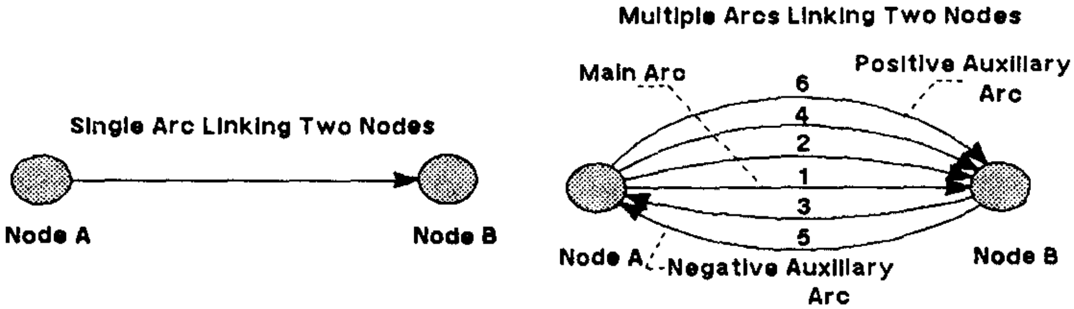


Figure 5: Arc Definitions

In Figure 5, additional arcs, added in the positive sense relative to the main arc, increase the maximum flow between nodes A and B. If the upper bound of the second positive auxiliary arc is greater than zero, then the maximum flow between the nodes is the sum of the upper bound for the main arc plus the upper bound for auxiliary arc 2. If the lower bound of arc 2 is zero, the minimum flow is equal to the lower bound of the main arc. For flow to be less than the main arc's lower bound, the minimum flow level set by the lower bound of the main arc must first be met and then that amount of the flow which is above the value required by the optimizer, returns from node B to A through one or more of the negative auxiliary arcs 3 and 5. The maximum flow from A to B is set by the sum of all the upper bounds of the arcs in the A to B direction and the minimum flow from A to B is set by the minimum lower bound of the main arc in the A to B direction minus the sum of the upper bounds of the negative arcs in the B to A direction, assuming that the lower bounds of all the auxiliary arcs are set to zero.

Figure 6 illustrates multiple operating zones for a reservoir corresponding to the preceding multiple arc flow diagram (Figure 5). Because the zone levels in Figure 6 vary with time, the upper and lower bound of the main arc and the upper bounds of all the auxiliary arcs change for each period. The lower bounds for the auxiliary arcs must always be zero.

Figure 7 shows typical penalty coefficients assigned to six rather arbitrary reservoir zones for a specific time of the annual operating year. These heuristic data for a multi-reservoir system must be developed through a combination of factual information and model testing over a representative period of historical records. In the examples done to date, a small negative coefficient is often applied to

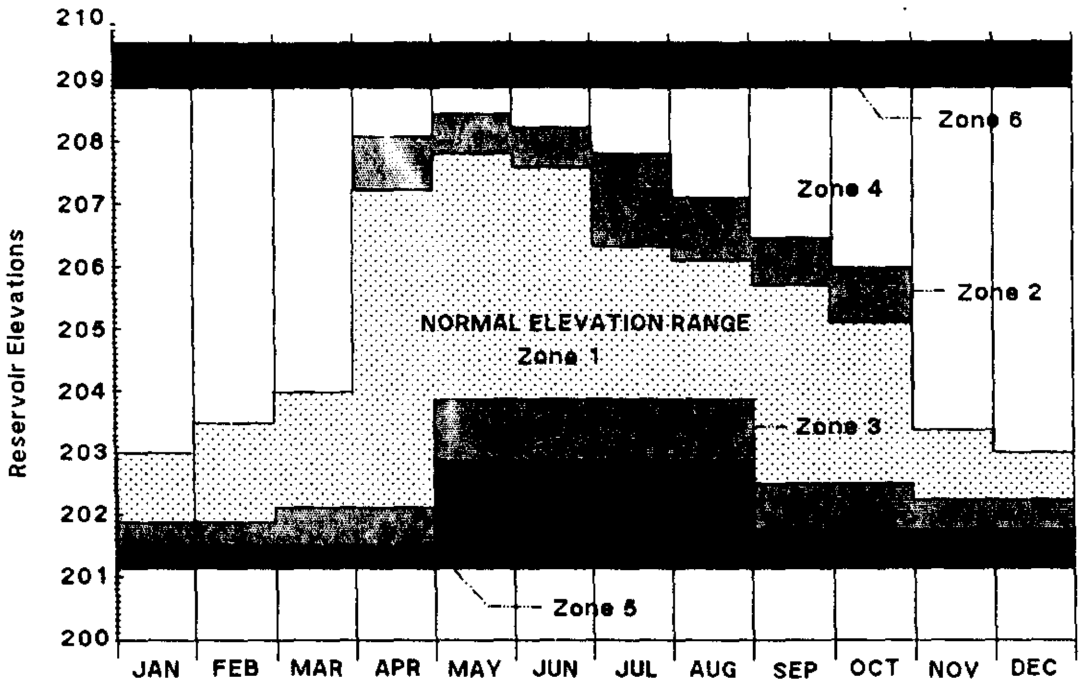


Figure 6: Representation of Multiple Zones for Reservoir Regulation

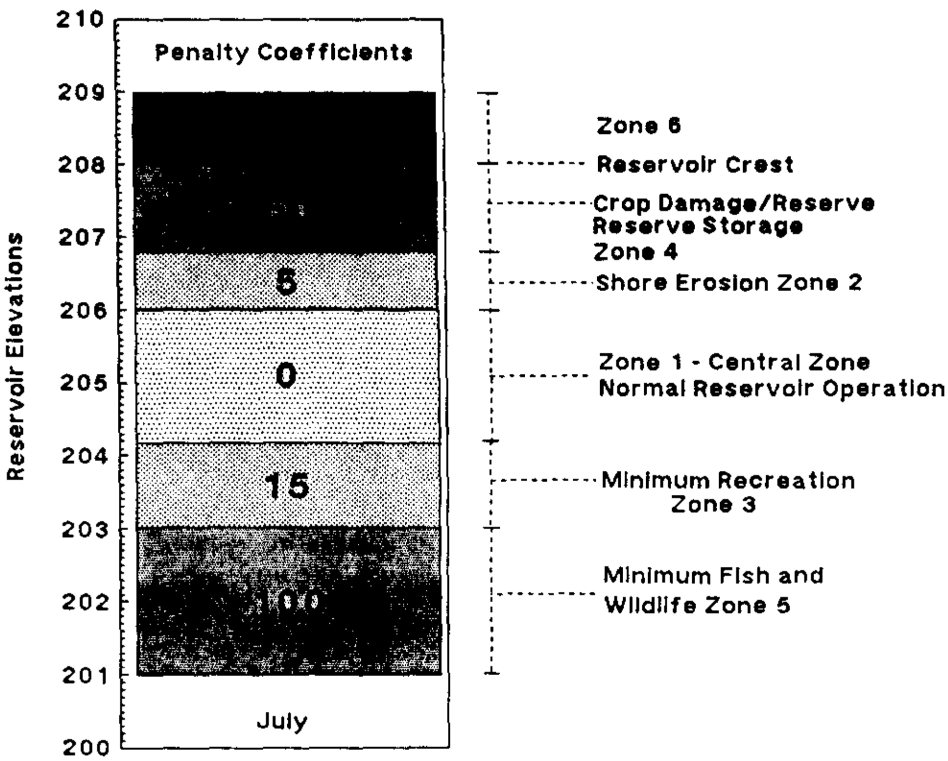


Figure 7: Penalty Coefficients

the storage arc for zone 1 which has the effect of storing water upstream in cases when most, if not all, the auxiliary arcs with high cost coefficients can maintain zero flow values.

REGUSE Model Structure

The model consists of three main programs (Figure 8). The first is the Generator which has two functional blocks. The user data files are read by the first block and the second creates the Main List of vectors which form the input to the optimizer. The user data file includes the hydraulic characteristics of the basin and the heuristic data. The hydraulic data consists of stage/discharge/storage tables, backwater/elevation data, and other data describing the physical characteristics of the basin. The heuristic data include lake and channel regulation curves, penalty coefficients and other decision variables supplied by the basin management team. The daily hydrometric data files are created and updated by a secondary program and include water use data, evaporation loss, channel flow, point inflows and lake elevation data. Initial conditions and target values are also stored in this file. The Main List vectors describe the configuration of the arcs in the flow network and give their upper and lower bounds and

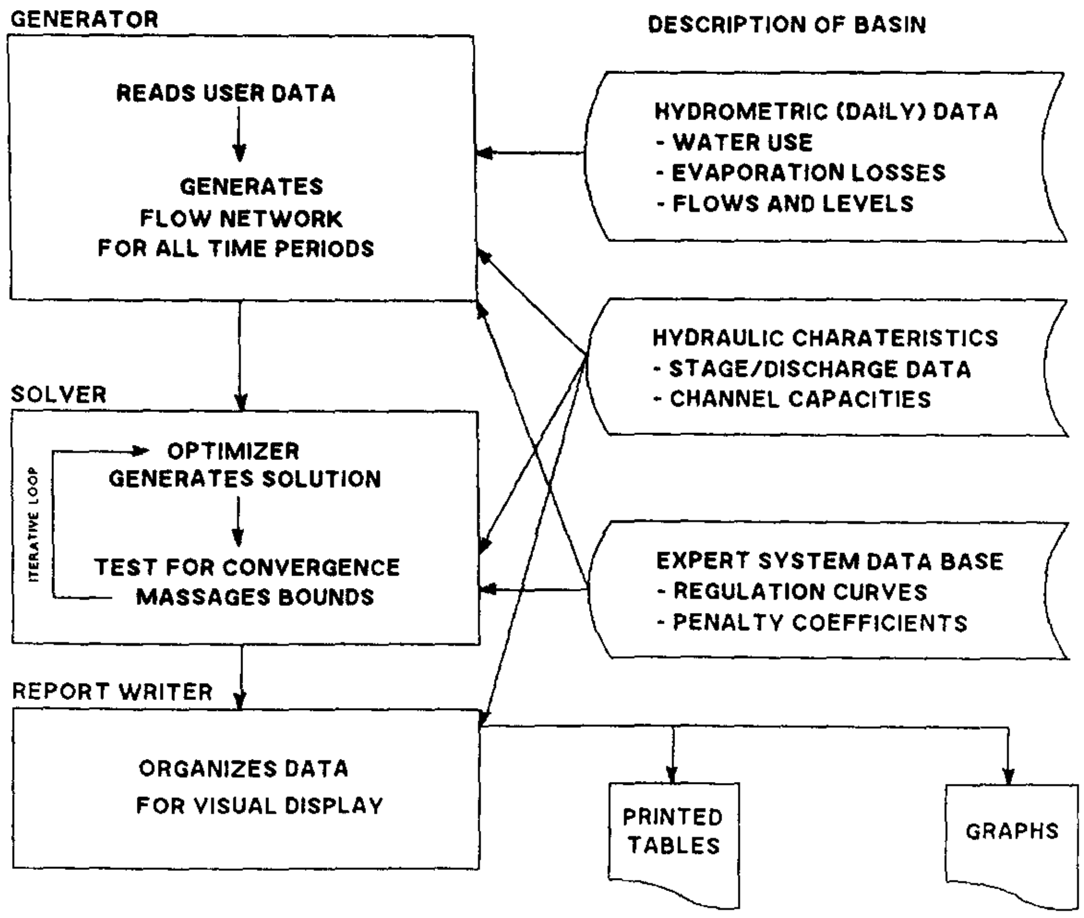


Figure 8: Overall REGUSE Model Flow

their penalty coefficients. To simplify the creation of the flow network, nine model features, identified as path types, have been defined. The path types allow the model user to construct the schematic diagram of the basin, identifying hydraulic and hydrometric features such as routing, multiple zone storage and flows, water use and control structures. The path types have subroutines which automatically generate a portion of the arcs in the Main List. These arcs represent the particular path types in the flow network. The data files provide the information needed by this block of the generator program.

The second program in the model is the solver. The arcs in the flow network are described entirely by their position in the network and their bounds and penalty coefficients. In some path types the flow in an arc is a function of the flow in other arcs. A weir is an example, where discharge is a function of lake elevation. This functional relationship is contained in the stage/discharge tables. However, before the discharge bounds can be calculated, the lake and tail water stages must be known. These values are only available after a solution has been obtained from the optimizer. An iterative loop uses the optimizer to find a solution, calculates new discharge bounds, and then recalculates the solution based on these new bounds. This loop continues until the solution differences between iterations is within defined tolerances. There are seven path types which require bound or penalty adjustment in the iterative loop. These include: a gate path which calculates weir discharges and insures that the discharge through a gate structure is within acceptable limits; the split flow table; the switched flow path which selects either of two sets of bound and penalty coefficients based on total channel flow; the routing path which adjusts the bounds on multiple arcs as a function of channel flow and the multiple zone storage and flow paths where the penalty coefficients can be increased if the flow in a regulation zone persists for too many time periods. In summary, the solver program consists of the optimizer, a subprogram which tests whether the flow or level differences between one iteration and the next are within a specified tolerance and a bound massager subprogram which adjusts the bounds or penalty coefficients based on the last solution from the optimizer.

The third program, the Report Writer, organizes the solution data so that visual tables and graphs are available to the user.

Path Types Available in REGUSE

1. Single Zone: This is the simplest of the path types. With it the user can specify a single flow channel having an upper and lower flow bound and a penalty coefficient. These values remain constant for all the unit-time periods in the time horizon. This path type has a number of uses. For example, a channel reach requiring flood routing might have its main channel identified using one single zone path, with a limiting upper flow bound and a parallel overbank channel identified with a second single zone path. These single zone paths would continue into separate routing paths, each having routing characteristics matching the type of flow they represent.

2. Split Flow: This path type forces the flow in a channel to split into two channels according to a flow data table. It can also be used to separate main channel flow and overbank flow.

3. Travel Time: This path type provides a simple form of routing where only travel time is a consideration.

4. Routing: This path type provides not only travel time, but also modifies the flow hydrograph as it moves down the channel. Although approximate, net flow routing appears to meet most operational needs, given the uncertainty of forecasted runoff.

5. Switched Flow: This path type allows the bound and penalty coefficients to be switched back and forth between two sets of values based upon a flag vector generated by the split flow path. The flags represent flow levels above or below a specific total channel flow. This path type can be used to modify the flood routing characteristics at a specific channel flow or it can eliminate sections of the schematic diagram under specified flow conditions.

6. Storage: This path type provides for multiple storage zones. The zone levels and penalty coefficients can change from unit-time period to unit-time period over the entire time horizon.

7. Flow: This path type is similar to the storage path above except that it provides multiple-flow zones in a channel. The penalty option is also available for this path.

8. Water Use: This path incorporates all the daily flow data such as water use demands, tributary and runoff inflows and evaporation losses into the flow network. This path allows the agriculture, municipal, industrial, etc. water demands over the entire time horizon to be included in the solution. Although entered as daily data, the data are automatically aggregated to match the unit-time period used by the model.

9. GATE1: This path has two modes. If the lake level is above a specified crest elevation, the structure is considered to be a weir. Discharge is calculated from a stage/discharge/tail water table. If the lake elevation is below the crest, flow is considered to be through a gate/stop-log structure. In this mode it is assumed that the discharge calculated by the optimizer can be achieved at the structure. The model calculates a maximum and minimum discharge based on the reservoir and tail water stages, and during the model's iteration cycle the discharge from the structure is constrained to these limits. The tail water stage is calculated as a function of total downstream discharge, including any downstream inflow, or it can be set equal to the stage of a downstream lake.

Steps in Applying REGUSE

The first step in applying REGUSE is to prepare a schematic diagram for a single unit time-period including all the hydraulic features of the basin expressed in terms of the path types previously described. The model then creates a network for the entire specified time horizon. Secondly, the user prepares a user schematic list identifying the from/to nodes, the path type, a data reference number, and a print/plot option code. Thirdly, the heuristic derived data, the hydraulic channel and structure data and the hydrological data are read into a random access file. Finally, the solution is generated, and the tables and graphs for the specified path types are printed.

The Qu'Appelle Basin - A Demonstration Project

The Qu'Appelle River in Saskatchewan is a prairie drainage basin (Figure 9) extending 400 km from its head waters at the Qu'Appelle Dam at Lake Diefenbaker to the Assiniboine River near the Manitoba/Saskatchewan border. The river is important because its drainage area of 52,000 km^2 is inhabited by approximately one-third of the province's population and much of the basin's agricultural and recreational industry is dependent on it. The Qu'Appelle valley varies in depth from 30 to 90 m and in width from 1.6 to 3.2 km. There are seven major lakes in its system which are regulated by six control structures.

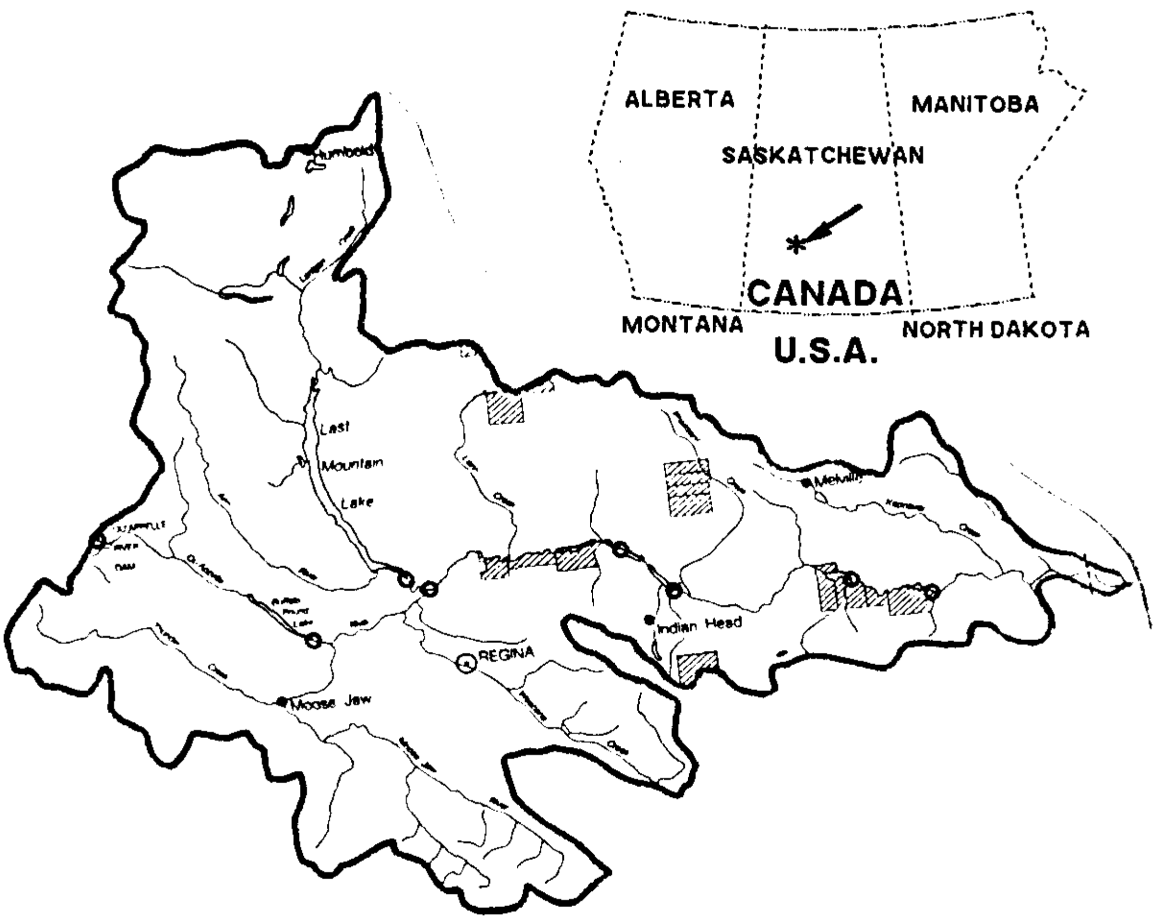


Figure 9: Qu'Appelle Demonstration Basin

Although evaporation is the largest single loss from the system, local agriculture, the two cities of Moose Jaw and Regina and the potash industry located within the basin are significant water users. Channel flows must be maintained at levels which will meet the water demands of the licenced pump/sprinkler irrigation systems of the wheat and market garden farmers within the valley over the summer growing periods. During the spring freshet, flooding of the agriculture lands is an important first step in the cultivation cycle. In years of low runoff, many farmers induce flood irrigation by actually plugging the river.

The basin has a number of hydraulic features which require the full range of capabilities built into the REGUSE model. Long meandering sections of channel mean that travel times of four to five days for a reach are common (extending to 28 days for the entire basin). The flow solution generated by REGUSE is based on the simultaneous consideration of all hydraulic features for all unit-time periods within the time horizon. The routing function incorporates travel time into the network for flow from one unit-time period to another. Consequently, the routing becomes an integral part of the solution. The need for changing the routing coefficients to deal with the main channel and overbank flow routing is accommodated by using the switched flow feature which selects the appropriate set of coefficients when a specified discharge has been reached.

A second feature is the reversing flow in the channel linking the main river channel with Last Mountain Lake. During periods of high flow, water from the main channel moves into Last Mountain Lake. Under normal flow conditions, discharge from the lake is controlled by a stop-log structure. Strong winds influence the amount and direction of flow. In REGUSE, the split flow feature allows flow from the main channel to be diverted into Last Mountain Lake, based on a flow table derived from historical flow data. Return flow is controlled by the model's gate/weir function plus a switched flow path which prevents outflow from the lake when inflow is occurring.

A third, important feature of the river system is the manner in which the control structures are operated. Under normal low flow conditions, discharge through the structure is over the stop-logs or through the control gates. During the spring freshet, when the lake level is above a specified level, the structure is opened and water passes through and over it, uncontrolled. When flood levels are very high, flow over the adjacent approach roads can also occur. The gate function in REGUSE handles both modes by testing the lake level against a specified level. When below this crest, the structure is considered to have gates or stop-logs and be fully controllable. Above, it is treated as a weir. In the weir mode, the discharge is determined by a stage/discharge table reflecting a fixed crest structure and the adjacent approach roads. If tail water levels are to be considered, then an expanded table is used. In the gate/stop-log mode, the model calculates a solution based on the assumption that whatever discharge is calculated, it can, in fact, be achieved by the work crews adjusting the structure. The model insures that this discharge is within practical upper and lower limits set by the model user for each lake elevation.

A backwater condition and reversal of flow occurs at Buffalo Pound Lake during high spring flows downstream of Moose Jaw Creek. This situation is handled in REGUSE by the split flow function which directs secondary flow from the creek into the lake according to a flow splitting table.

An additional basin need handled by REGUSE, is the requirement to specify time dependent regulation curves for each of the lakes. The model's multi-zone storage feature allows up to nine target limit curves to be specified for each lake. The levels and penalty coefficients associated with each of these curves can vary with time. Penalty coefficients play an important role in REGUSE and when greater than zero, place a cost against the lake elevation falling within the zone between two level curves. A further property of the model's multi-zone storage feature is the ability to automatically increase a penalty coefficient if the level calculated persists above a specified level curve for more than one unit-time period. The multi-zone feature can also be applied to channels where the regulation curves are specified in terms of flow rather than elevation.

The next step in implementing the model for operational use involves running the model in a planning mode over the period 1972 to 1983. The heuristic database defining the multi-zones and associated cost penalties will be refined by an interdisciplinary team fully familiar with the basin. A vector computer (Cray 1-S) has been used in the model testing stage and will be employed in the refinement of the heuristics.

Conclusions

(1) Heuristics cannot be avoided in streamflow regulation for basins with multiple water uses.

(2) Network flow algorithms appear to be ideally suited, at this time, for performing the task of the inference engine common to expert systems. Available expert systems software, referred to as shells, does not conveniently handle the hydraulic portion of the regulation problem covered by physical laws.

(3) The increasing availability of vector computing facilities is rapidly decreasing electronic data processing costs and the development time for applying computationally intensive river/reservoir regulation models to particular basins.

Acknowledgement

All hydraulic data for structures and channels, the major portion of the hydrologic data and the initial heuristic data were supplied by the Saskatchewan Water Corporation for the Qu'Appelle basin demonstration project.

Appendix 1 - References

Barr, R.S.; Glover, F.; Klingman, D. 1974. An Improved Version of the Out-of-Kilter Method and a Comparative Study of Computer Codes. Mathematical Programming 7 (1974). North Holland Publishing Co. pp 60-86.

Courbu, I.; Lau, P.C.K. 1984. Trade-Off Decisions in Streamflow Forecasting for Multiobjective River Basins - The Ottawa River Case. Canadian Water Resources Journal, Volume 9, No. 4, 1984.

Fordyce, K.; Norden, P.; Sullivan, G. 1987. Review of Expert Systems for the Management Science Practitioner. INTERFACES 17:2 March to April 1987 (pp 64-67).

Phelps, R.I. 1986. Artificial Intelligence - An Overview of Similarities with O.R. Journal of the Operational Research Society Vol. 37, No. 1 (pp 13-20).

Simon, H.A. 1987. Two Heads are Better than One: The Collaboration Between AI and OR. INTERFACES 17:4 July to August 1987 (pp 8-15).

Tingley, G.A. 1987. Can MS/OR Sell Itself Well Enough? INTERFACES 17:4 July-August 1987 (pp 41-52).

University of Manitoba, Civil Engineering Department, Round Table Discussion on New Trends in Water Resources Systems Analysis, personal notes by Farley on round table discussion held on May 14, 15, 1987.

Woolsey, R.E.D.; Swanson, H.S. 1975. Operations Research for Immediate Application, a Quick and Dirty Manual. Harper and Row Publishers Inc.

Heuristics and Network Flow Algorithms
for Multi-Reservoir System Regulation
by Farley, D.W.; Sydor, M.; Brown, G.E.

List of Figures

MULTIRESERVOIR SIMULATION MODEL (MRSM)

by
Brian Bell
PFRA Hydrology Division

General

The Multireservoir Simulation Model was developed in the late 1960's by the Saskatchewan-Nelson Basin Board (SNBB) for use in the Saskatchewan-Nelson Basin Study. It was subsequently acquired by the Prairie Farm Rehabilitation Administration (PFRA) in 1976 and extensively modified for use in the Souris River Basin Study. Various components of the model have been modified since the Souris River Basin Study to provide a more versatile and applicable model.

The model simulates the flow of water through a multireservoir system on a monthly basis. The model first examines the entire period of record and computes volumes, known as reservoir rules, that must be maintained in order to meet demands immediately below any reservoir or other study point, regardless of flow requirements at downstream points. Reservoir releases from upstream subsystems to meet downstream demands are checked to ensure that the volume of water in the subsystems exceeds the computed reservoir rules.

In the simulation process, all study points and projects (collectively called study points) are scanned in an upstream-to-downstream order for the purpose of redefining minimum outflows at each study point. If a reservoir is able to store all of the inflow (i.e. no spills), the minimum outflows are not redefined. If a study point cannot store all the inflow in excess of water demands, the excess water must be passed and the defined "minimum" outflow is increased to the excess quantity for that month. All study points are then scanned in a downstream-to-upstream order for the purpose of redefining outflows at each study point. If inflows are not sufficient to meet the demands at a study point, the inflows are supplemented by a release from an upstream reservoir. This simulation process causes downstream reservoirs to empty first (unless desirable storage is specified) and keeps system spills to a minimum. If the outflows are in excess of downstream requirements, the outflows remain set to either the minimum or redefined minimum flows. All study points are then scanned in an upstream-to-downstream order to provide a final indication of all outlet outflows.

In the simulation process, the user has the capability of defining various parameters such as water demands, outlet details, inflow, return flow, conveyance loss, net evaporation, ice formation, dead storage, desirable storage, full supply level and an elevation-storage-area relationship. Some of these project parameters (e.g. outlet details, ice formation, dead storage, full supply level and the elevation-storage-area relationship) are loaded onto computer files as project data, some parameters (e.g. net evaporation, inflow, water demands) are loaded as arrays in the data files prior to the simulation process and are retrieved during the simulation process, while other parameters (e.g. return flow, conveyance loss) are loaded onto computer files as part of the project combination data. Most of the parameters can also be redefined during the multireservoir simulation execution data setup.

Data Structure

The Multireservoir Simulation Model requires three different types of data:

1) Project data,
2) Project combination data, and
3) Multireservoir simulation data.

Project data are prepared for all selected study points (including all possible projects) considered in the system. These data define the physical characteristics of each study point.

Project combination data are used to define study point linkages and other information dependent upon the specific combination of study points under consideration.

Multireservoir simulation data are required prior to the execution of a simulation run. The data are used to set up variable dimensions, to override some of the project data and project combination data if required, and to provide other necessary information for the simulation run. Multireservoir simulations also require large amounts of input data. These data are organized for operating efficiency and quality control. All input data, except multireservoir simulation data, are catalogued and stored in direct access data banks. The output from multireservoir simulation runs may also be stored in data banks for future reference and subsequent studies.

Project Parameters

Water demands can be satisfied by water obtained either directly from a reservoir via an outlet or a water conveyance channel (termed as "onstream" demand), or from water derived within the effective drainage area between study points (termed "offstream" demand). The model is able to distinguish between these two types of demands by the form of the input data and system specifications.

Natural monthly mean flow arrays are required for each study point. These arrays are accessed by the model and manipulated in accordance with instructions specified during the project combination data setup. Inflow to the study sites are determined within the model by combining the indicated flow arrays to obtain local inflow, subtracting "on-stream" and "off-stream" uses from local inflow to determine net inflow, and adding residual flows (if any) from upstream study sites to the net local inflow. However, the resultant net inflow is not permitted to fall below zero.

Residual flows from each upstream study point are determined within the model by first adjusting the upstream study point outflow by the "onstream" demand. The resultant value is then decreased by the appropriate base conveyance loss and further decreased by the percentage conveyance loss. Conveyance losses are "lost" from the system as water moves to the downstream study point. Normally, conveyance losses are associated only with artificial channels (i.e. irrigation canals) or intermittent streams because any conveyance losses incurred during periods of flow are inherently incorporated in the flow record and the reconstructed natural flow arrays.

Monthly net evaporation arrays are required for each study point having a specified capacity greater than zero. These arrays are accessed by the model and used in the simulation in accordance with specified indicators. Evaporation losses from each project are computed by the model as a function of the reservoir area. Net evaporation is normally accumulated during winter months (starting in November). The accumulated value is multiplied by the reservoir area based on "average" monthly storage (or daily storage as appropriate) and incorporated in the water balance in March or April depending upon runoff conditions.

Monthly ice formation for each project is computed by the model as a function of reservoir area. The reservoir area corresponding to the "average" monthly storage (or daily storage, as appropriate) is multiplied by the appropriate value of ice thickness (a nominal total value of 0.90 metres;

0.15 metres in November and February and 0.30 metres in December and January). The volume of water stored in the form of ice is returned to a liquid form using the same criteria as used for net evaporation.

Dead storage, desirable storage, capacity and an elevation-storage-area relationship are specified (when applicable) for each project.

Limitations

The model has several limitations which may or may not be important in the simulation of specific systems. These limitations are briefly discussed in the following items.

1) The model is designed to make the most efficient use of the water in the system (i.e. minimize spills). Thus, projects are drained in a downstream-to-upstream progression in satisfying specified demands or minimum flow criteria. Water is completely utilized at a project (i.e. water level drops to dead storage level) before water is released from an upstream project to meet the specified demands. However, a more realistic system operation plan with respect to the distribution of stored water within the system can be achieved through the judicious specification of desirable storage.

2) The model does not consider channel storage or travel time. The effect of releasing water at an upstream project is felt instantaneously at the downstream project. This assumption is reasonable if the time of travel is much less than the time increment (monthly) used in the model.

3) The search technique for obtaining a maximum guaranteed flow at a selected study point is somewhat restrictive. An initial estimate must be made of guaranteed flow, the range of the guaranteed flow must be specified and the accuracy of search must be indicated. The specification of these constraints will depend on the required accuracy, how confident the user is in the initial estimate and cost considerations. However, a guaranteed flow can be determined even if there are shortages somewhere in the system, as long as no shortages are incurred at the specified study point outlet. Alternatively, a guaranteed flow can be determined at the specified study point outlet under a constraint of no shortages in the system. Furthermore, the magnitude and significance of shortages that are permitted at the maximization point can be varied to provide additional flexibility.

HY03 - SINGLE RESERVOIR SIMULATION MODEL

by
David Kiely
PFRA Hydrology Division

General

The HY03 computer program was developed in the late 1960's by the Hydrology Division of the Prairie Farm Rehabilitation Administration (PFRA) to assess the water supply potential of existing or proposed water storage projects. Since then, the program has been extensively modified to meet the changing needs of the Hydrology Division.

HY03 is a monthly water balance computer program which simulates the flow of water through a single water storage reservoir in accordance with reservoir characteristics and water demands. The simulation is based on the following water balance equation where the net inflow minus the outflow, which includes any direct reservoir uses, is equivalent to the change in storage.

$$\Delta S = [(F - U) + (F_i - U_i) + F_s] - [D_r + D_l + Evap + Ice + Spills]$$

where: ΔS = change in storage in dam^3
F = inflow volume in dam^3
U = inflow water demand in dam^3
F_i = imported water volume in dam^3
U_i = import water demand in dam^3
F_s = supplementary inflow volume in dam^3
D_r = riparian (downstream) water demand in dam^3
D_l = local demand in dam^3
Evap = evaporation volume in dam^3
Ice = volume of water, in dam^3, formed into ice during November to February and returned to a liquid form in the appropriate spring month
Spills = volume of water, in dam^3, in excess of the reservoir capacity

The net monthly inflow (less supplementary inflow) is determined by subtracting the monthly water demands from the corresponding monthly inflows and imported water volumes. The monthly consumption of water from the reservoir is composed of the local and riparian demands, evaporation and ice (if applicable). Excess water is spilled. The monthly volumes of water consumed by evaporation and temporarily stored as ice are calculated by an iterative process based on the assumption that all inputs and withdrawals occur simultaneously. However, priority is given to satisfying evaporation and ice formation components before meeting the requirements of the local and riparian outlet water demands in shortage conditions. The order of priority in which the model meets the local and riparian demands may be specified by the user. In the model, when the demands cannot be met (i.e. the reservoir falls below a given storage limit), supplemented water can be added to the system in accordance with specified constraints to meet the demands. The supplementary inflow specified by the user is the upper limit to this process.

Special Features

If special reservoir or project operating conditions or constraints are to be simulated, the HY03 model is able to simulate the reservoir on a pseudo-daily basis. Pseudo-daily calculations are made primarily for cases when the local or riparian demand (e.g. irrigation) is for a portion of certain months or when an elevation-discharge relationship is used for either of the demand outlets. In the pseudo-daily calculations, the model distributes the evaporation rate uniformly over the entire month but allows the monthly inflow, supplementary inflow and imported flow volumes to be distributed over entire months or specified daily periods.

The HY03 reservoir model also has the ability to simulate the water quality aspect of the reservoir in terms of total dissolved solids (TDS). The TDS values for the inflow, import and supplementary flows can be input into the model directly or through a TDS-discharge relationship. Mixing within the reservoir is assumed to be complete and instantaneous, and the quality of the withdrawn water is determined on the basis of this mixing assumption. As an option, supplementary and/or imported inflow may be routed directly to the local outlet in order to avoid the mixing effect of the reservoir in calculating water quality.

Another special feature of the HY03 program is the ability of the model to allow for rationing of the reservoir water based on prespecified storage limits. Reduction factors can be applied to the local demand at specified storage thresholds in order to model varying reservoir operations.

Data Requirements

The user has the capability of defining various reservoir or basin characteristics during a reservoir simulation. These characteristics are:

1. inflow (including imported water and supplementary flow)
2. net evaporation
3. water demands (including upstream, local and riparian demands)
4. ice formation
5. dead storage
6. reservoir capacity
7. elevation-storage-area relationships
8. elevation-discharge relationships
9. initial reservoir storage
10. water quality parameters
11. local demand reduction factors and thresholds

Some of these parameters (inflow, net evaporation, dead storage and water demands) can be loaded as arrays in the data files prior to the simulation process and can be retrieved during the simulation process, or they can be read as part of the data setup. The other parameters are input in the data setup.

Limitations

The program has several limitations which may or may not be important in the simulation of a specific reservoir. These limitations are briefly discussed in the following items.

1. The program is designed to simulate a reservoir on a monthly basis. Inflow, net evaporation, upstream water demands, etc., are utilized as monthly volume values. Normal daily or weekly fluctuations in peak inflow, upstream water demands or net evaporation are not considered. However, daily operations and conditions of the reservoir can be simulated if the pseudo-daily option is specified.

2. The program does not generally consider routing effects or outflow capacities. The project is assumed to have sufficient capacity to release water to meet specified demands or to spill excess water as required. However, elevation-discharge relationships for the local and riparian outlets can be used if the pseudo-daily option is specified.

3. A unique dead storage value cannot be specified for the local and riparian demands if the simulation is made on a monthly basis. The dead storage value can be varied for each month over the study period; however, it applies to all demands (i.e. outlets). A unique dead storage value can be used for each outlet if the pseudo-daily option, in conjunction with a corresponding elevation-discharge relationship, is specified.

HY01
PFRA WATER SUPPLY POTENTIAL PROGRAM

prepared by G. W. Bell
PFRA Hydrology Division

The HY01 computer program is used to assess the water supply potential of existing or proposed water storage projects. The program was developed in the late 1960's by the Hydrology Division of the Prairie Farm Rehabilitation Administration. Since its inception, the program has undergone extensive modifications to improve its flexibility and to make it compatible with other Hydrology Division computer programs.

HY01 is a monthly water balance computer program which can be used to estimate the water supply potential (i.e. excess water availability referred to as draft) for a single water storage reservoir. The program simulates the flow of water, on a monthly basis, through a reservoir in accordance with reservoir characteristics, water demands and user-specified draft criteria. The simulation is based on the water balance equation presented in equation 1 where the net inflow minus the outflow, which includes any direct reservoir uses, is equivalent to the change in storage.

$$S = (F-Du)-(Dr+R+Dp+Evap+Ice+Spill) \quad \text{Equation 1}$$

where:
- S = change in storage in dam^3
- F = inflow volume in dam^3
- Du = upstream water demand in dam^3
- Dr = draft in dam^3
- R = riparian volume in dam^3
- Dp = prior water demand in dam^3
- Evap = evaporation volume in dam^3
- Ice = volume of water, in dam^3, formed into ice during November to February and returned to a liquid form in the appropriate spring month
- Spill = volume of water, in dam^3, in excess of the reservoir capacity

The available draft is determined through an iterative process. An initial estimate of the available annual draft is made by dividing the total net volume of water available at the reservoir by the number of months in the study period that the annual draft is required, as indicated in equation 2.

$$ADV = \frac{\sum_{N=1}^{N=Nm} [(F-Du) - R - Dp] + (IR - Ds)}{[(NQ - NSHAL) * (M_l - M_f + 1)]} \quad \ldots\ldots\ldots\ldots \quad \text{Equation 2}$$

where:

ADV = annual draft volume in dam^3

$\sum$ = summation of monthly data; N = consecutive number of month and Nm = total number of months in the study period.

F = monthly flow volume in dam^3

Du = monthly upstream water demand in dam^3

R = monthly riparian volume in dam^3

Dp = monthly prior water demand volume in dam^3

IR = initial reservoir volume in dam^3

Ds = dead storage volume in dam^3

NQ = number of years in study period

NSHAL = number of years that shortages are permitted

M_f = first month draft is to be withdrawn from the reservoir

M_l = last month draft is to be withdrawn from the reservoir

Once an initial estimate of the annual draft volume is made, it is distributed to the appropriate months in accordance with the specified draft distribution pattern and incorporated into the water balance equation. After the entire period has been simulated, a check is made to see if all draft criteria are met. If the criteria are not met, the initial annual draft is adjusted and the simulation is repeated for the entire study period. This process is repeated (to a maximum of 30 trials) until the trial annual draft meets the user-specified criteria.

The draft can be determined based on six user-specified parameters which establish the draft criteria. These parameters are:

1. draft period,
2. draft distribution,
3. percent of years in which shortages are permitted,
4. maximum allowable shortage as a percent of draft,
5. significant shortage as a percent of draft, and
6. reduction of draft at a specified storage.

The annual draft period can be designated by specifying the first and last months of a consecutive period that the draft is to be withdrawn. For example, withdrawals would likely occur during the January to December period in each year of the study period for municipal demands, or the May to August period for irrigation demands.

The monthly draft can also be varied within the annual draft period in accordance with a specified draft distribution. For example, 60% of the annual draft can be taken out in May, 20% in June and 20% in July. The draft distribution can be totally flexible, as long as the monthly percentages over the specified draft period total 100%.

The percent of years (based on the number of years within the study period) in which the annual draft is obtainable can be specified by the user in terms of the percent of years in which shortages are permitted (e.g. 0%, 10%, 20% of the years) within the study period. As the percent of years in which shortages can occur is increased, the magnitude of the annual draft increases because a greater number of shortages are permitted in the low flow years.

The maximum allowable shortage as a percent of the draft can also be specified. If the user allows shortages in the potential draft to occur, the largest allowable shortage can be set as a percent of the draft. This criteria restricts the maximum shortage that can occur as a function of the trial draft. This feature may be useful in determining an available draft for a town water supply where a partial shortage could be tolerated.

Significant shortage as a percent of the draft (i.e. a shortage is only recognized as a 'shortage' if its magnitude exceeds a user-specified percentage of the draft) can be specified. This feature may be useful in cases where small shortages are acceptable and would not affect the viability of the project.

Reduction of draft at a specified storage (i.e. water rationing) can be incorporated into the calculation of the annual draft. Rationing reduces the occurrence of severe shortages during low flow years and allows for a greater utilization of water during high flow years. The magnitude of the draft is reduced during low-flow periods by a user-specified percentage of the estimated trial draft based on the volume of water stored in the reservoir prior to each month during the draft period. Up to six different reservoir volumes and corresponding draft reduction factors can be specified.

The program has several limitations which may or may not be important in the simulation of a specific reservoir. These limitations are briefly discussed in the following items.

1. The program is designed to simulate a reservoir on a monthly basis. Inflow, net evaporation, water demand, etc., are utilized as monthly volume values. Normal daily or weekly fluctuations in peak inflow, water demand or net evaporation are not considered.

2. The program does not consider routing effects or outflow capacities. The project is assumed to have sufficient capacity to release water to meet specified demands or to spill excess water as required.

3. A unique dead storage value cannot be specified for each demand (i.e. prior water uses, riparian flow and draft). The dead storage value can be varied for each month over the study period; however, it applies to all demands (i.e. outlets).

Natyield Model

J.H Taggart[1]

The Natyield Model was developed by the Hydrology Branch of Alberta Environment, to compute natural yields in heavily licenced watersheds within Alberta. These yields are then used to assess the availability of water for licencing.

In the past, a number of techniques were used to estimate natural yields in licenced watersheds. These techniques included: using recorded near-natural hydrometric data; using variable drainage areas based on a straight line representation of drainage area for extremely wet and dry years; and by use of the project depletion method. These methods have numerous problems and at best provide inconsistent and questionable results.

The objective of the Natyield Model, is to provide reliable natural yield estimates for licenced watersheds, using recorded hydrometric data within the watersheds. The technique accounts for all consumptive uses, storage, and net evaporation losses. The model computes the natural yield, which is required to produce the recorded volume for the month being modelled.

The model was first developed as an annual model, which accounted for the licences in operation in a given year, using composite reservoirs, (modelled reservoirs used to represent a large number of reservoirs). Later, the model was expanded into a continuous monthly model, which represents all the licences, waterbodies, and points of interest, within the watershed for the period of record. The Natyield Model configures the watershed by representing each licence, waterbody, or point of interest, by a node within the model. A water balance is

1. J.H. Taggart, P. Eng., Hydrologist, Hydrology Branch, Technical Services Divsion, Alberta Environment

then computed at each node, starting at the upper reaches of the watershed and extending down to the hydrometric station. Using a Regula-Falsi technique the model iterates between the computed and the recorded monthly volumes. This process is repeated until a balance is established between the computed and recorded output volumes. The natural yield estimate that produced this balance is assumded to be the natural yield for the basin. The model then proceeds to the next month.

The data required for the use of the Natyield Model include:

1. Drainage Area to each node.
2. Hydrometric Data.
3. Net Evaporation estimates.
4. Estimates of Surface Area and Capacity of each waterbody.
5. Years of Operation of each licence and reservoir.
6. Consumption use (Demand) of each licence.

The application of the model to watersheds, in close proximity to each other, indicates similar natural yields in each basin. The hydrometric data however, indicated major differences in the basin's output. The use of the Natyield model, thereby provides the expected consistency between basins that was absent when using the other techniques.

The Natyield model can be modified for other uses, by inputting the natural yields back into the model. Natural flows in a licenced basin can then be computed by disabling the iteration sub-routine and leaving only the natural waterbodies in the model. Similarly, the model can also be used as an operational model, or a planning model, by introducing the existing and/or proposed development into the model. In the future, with further modification including a shorter time-step, the impacts of drainage could be simulated.

APPENDICES

PRAIRIE PROVINCES WATER BOARD

COMMITTEE ON HYDROLOGY
PRAIRIE HYDROLOGY WORKSHOP NO. 3
SASKATOON, SASKATCHEWAN
OCTOBER 18-19, 1988

TUESDAY - OCTOBER 18, 1988

8:30 - 9:00	Registration
9:00 - 9:15	Welcome and Opening Remarks - A. B. Banga (Sask.) - R. L. Kellow (PPWB)

PRAIRIE PROVINCES WATER BOARD

9:15 - 10:00	PPWB Activities - R. L. Kellow (PPWB)
10:00 - 10:15	Master Agreement on Apportionment - History and Terms - A. B. Banga (Sask.)
10:15 - 10:30	Coffee
10:30 - 11:00	COH Activities - F. Martin (PFRA)

SIGNIFICANT HYDROLOGY EVENTS

11:00 - 11:30	Meteorologic Events - R. F. Hopkinson (AES)
11:30 - 12:00	Runoff Events - L. Warner (Environment Canada)
12:00 - 1:15	Lunch

HYDROLOGIC AND HYDRAULIC MODELS

1:15 - 2:00	HYMO - B. Kallenbach (Sask.)
2:00 - 2:45	HEC-1 - J. Yarotski (PFRA)
2:45 - 3:30	SSARR - S. Figliuzzi (Alta)
3:30 - 3:45	Coffee
3:45 - 4:30	HSPF - G. Mohr (Manitoba)
4:30 - 5:00	1-D - M. Sydor (Environment Canada)
5:00 - 5:30	HEC-2 - L. Wiens (Environment Canada)

6:00 - 7:00	Happy Hour
7:00 - 8:00	Banquet
8:00 - 8:30	Guest Speaker: W. Nicholaichuk (NHRI)

WEDNESDAY, OCTOBER 19, 1988

WATER MANAGEMENT MODELS

8:45 - 9:45	SED - F. Davies (Alberta)
9:45 - 10:20	HYDSIM - R. Divi (SPC)
10:20 - 10:35	Coffee
10:35 - 11:15	WUAM - J. Rogers (Environment Canada)
11:15 - 12:00	Reguse - P. Sandhu and G. Brown (Environment Canada)
12:00 - 1:15	Lunch
1:15 - 2:15	MRSM - B. Bell (PFRA)
2:15 - 2:45	HY03 - D. Kiely (PFRA)
2:45 - 3:00	Coffee
3:00 - 3:15	HY01 - G. Bell (PFRA)
3:15 - 4:00	Natural Yield Model - J. Taggert (Alberta)
4:00 - 4:30	Wrap-up - F. Martin (PFRA)

L I S T O F P A R T I C I P A N T S

SASKATCHEWAN WATER CORPORATION

NAME	AFFILIATION	ADDRESS
A.B. Banga Phone: (306) 694-3959	Manager Hydrology Service Sask. Water Corporation	Victoria Place 111 Fairford St. E. Moose Jaw, Sask. S6H 7X9
Dalvin Euteneier Phone: (306) 694-3966	Hydrologist Hydrology Service Sask. Water Corporation	Same as above.
Garnet Gobert Phone: (306) 862-1763	Regional Hydrologist Sask. Water Corporation	P.O. Box 2133 Nipawin, Sask. SOE 1EO
Martin Grajczyk Phone: (306) 694-3958	Hydrologist Hydrology Service Sask. Water Corporation	Victoria Place 111 Fairford St. E. Moose Jaw, Sask. S6H 7X9
Bernie Kallenbach Phone: (306) 694-3960	Hydrology Service Sask. Water Corporation	Same as above.
Lloyd Thiele Phone: (306) 694-3876	Sask. Water Corporation	Same as above.
Jon Vittachi Phone: (306) 694-3962	Assistant Hydrologist Hydrology Service Sask. Water Corporation	Same as above.
Nico Wyngaarden Phone: (306) 694-3883	Regional Hydrologist Sask. Water Corporation	Same as above.

PRAIRIE FARM REHABILITATION ADMINISTRATION

NAME	AFFILIATION	ADDRESS
B.J. Bell Phone: (306) 780-6516	Hydrologic Engineer Hydrology Division Prairie Farm Rehabilitation Administration	4th Floor 1901 Victoria Ave. Regina, Sask. S4P 0R5
Gord Bell Phone: (306) 780-5167	Prairie Farm Rehabilitation Administration	Same as above.
Duane Kelln Phone: (306) 780-6510	Hydrology Division Prairie Farm Rehabilitation Administration	Same as above.
Dave Kiely Phone: (306) 780-5167	Hydrology Division Prairie Farm Rehabilitation Administration	Same as above.
P.J. Kujawa Phone: (306) 780-6653	Design Engineer Design Division Prairie Farm Rehabilitation Administration	6th Floor 1901 Victoria Ave. Regina, Sask. S4P 0R5
Fred R.J. Martin Phone: (306) 780-5165	Manager, Hydrology Division Prairie Farm Rehabilitation Administration	4th Floor 1901 Victoria Ave. Regina, Sask. S4P 0R5
Mike Mowchenko Phone: (306) 780-5168	Prairie Farm Rehabilitation Administration	Same as above.
R.J. Woodvine Phone: (306) 780-6518	Head, Flood Studies Hydrology Division Prairie Farm Rehabilitation Administration	Same as above.
Jim Yarotski Phone: (306) 780-6516	Prairie Farm Rehabilitation Administration	Same as above.

ENVIRONMENT CANADA

NAME	AFFILIATION	ADDRESS
Bill Aitken Phone: (306) 780-6357	Project Engineer Water Planning & Management Branch Inland Waters Directorate Environment Canada	1st Floor 1901 Victoria Ave. Regina, Sask. S4P 0R5
G.E. Brown Phone: (819) 997-1953	Environment Canada	Inland Waters Directorate Environment Canada Place Vincent Massey Ottawa, Ontario K1A 0E7
R.F. Hopkinson Phone: (306) 780-5739	Atmospheric Environment Service Environment Canada	P.O. Box 4800 Regina, Sask. S4S 2G1
Brian Johnson Phone: (306) 780-5343	Water Resources Branch Environment Canada	210 - 1102 - 8th Ave. Regina, Sask. S4R 1C9
Ken Jones Phone: (306) 780-5739	Scientific Services Meteorologist Atmospheric Environment Service	P.O. Box 4800 Regina, Sask. S4S 2G1
Leslie Lamers Phone: (306) 780-6425	Hydrologic Engineer Water Planning & Management Branch Environment Canada	1901 Victoria Ave. Regina, Sask. S4P 0R5
Greg MacCulloch Phone: (403) 292-5409	Water Resources Branch Environment Canada	854-220-4th Ave. S.E. Calgary, Alberta T2P 2M7
Mike Renouf Phone: (306) 780-5040	Studies & Systems Engineer Environment Canada Water Resources Branch	#210-1102 - 8th Ave. Regina, Sask. S4R 1C9

ENVIRONMENT CANADA - CONT'D.

NAME	AFFILIATION	ADDRESS
Jim Rodgers Phone: (306) 780-5320	Drought Study Officer Water Planning & Management Branch, C & P Inland Waters Directorate Environment Canada	1st Floor 1901 Victoria Ave. Regina, Sask. S4P OR5
T.P. Sandhu Phone: (819) 953-1526	Senior Planning Engineer Engineering & Development Division WPM, Environment Canada	351 St. Joseph Boulevard Hull, Quebec
M. Syder Phone: (819) 997-2319	Water Planning & Management Branch Environment Canada	Inland Waters Directorate Environment Canada Place Vincent Massey Ottawa, Ontario K1A OE7
Lyman Warner Phone: (403) 231-5406	Water Resources Branch Hydrology Division Environment Canada	Bag 2909 Postal Station "M" Calgary, Alberta T2P 2M7
Larry H. Wiens Phone: (306) 780-5329	Environment Canada	1st Floor 1901 Victoria Ave. Regina, Sask. S4P OR5
Brian Yee Phone: (306) 780-5342	Boundary Waters Engineer Water Resources Branch Environment Canada	#210-1102 - 8th Ave. Regina, Sask. S4R 1C9

MANITOBA WATER RESOURCES BRANCH

NAME	AFFILIATION	ADDRESS
Eric Blais Phone: (204) 945-7414	Manitoba Water Resources Branch	1577 Dublin Avenue Winnipeg, Manitoba R3E 3J5
Rick Bowering Phone: (204) 945-6397	Manitoba Water Resources	Same as above.
Neil Harden Phone: (204) 945-6118	Hydrologist Manitoba Water Resources Branch	Same as above.
Bob Harrison Phone: (204) 945-6118	Hydrologist Manitoba Water Resources Branch	Same as above.
G.E. Mohr Phone: (204) 945-6695	Flow Forecaster Manitoba Water Resources Branch	Same as above.
Dave Walker Phone: (204) 945-7413	Manitoba Water Resources	Same as above.
A.A. Warkentin Phone: (204) 945-6698	River Forecast Centre Manitoba Water Resources	Same as above.

ALBERTA ENVIRONMENT

NAME	AFFILIATION	ADDRESS
Ron Bothe Phone: (403) 427-6277	Senior Hydrologist Northern Basins Hydrology Branch Alberta Environment	10th Floor, Oxbridge Place 9820 - 106 Street Edmonton, Alberta T5K 2J6
Frank Davies Phone: (403) 427-6277	Hydrology Branch Alberta Environment	Same as above.
Andy DeBoer Phone: (403) 427-6277	Alberta Environment	Same as above.
S.J. Figliuzzi Phone: (403) 427-6277	Hydrology Branch Alberta Environment	Same as above.
A. Majeed Mustapha Phone: (403) 427-6277	Hydrology Branch Alberta Environment	Same as above.
John Taggart Phone: (403) 427-6277	Hydrology Branch Alberta Environment	Same as above.

PRAIRIE PROVINCES WATER BOARD

NAME	AFFILIATION	ADDRESS
A.J. Chen Phone: (306) 522-6671	Operations Engineer Prairie Provinces Water Board	201-2050 Cornwall St. Regina, Sask. S4P 2K5
R.L. Kellow Phone: (306) 522-6671	Executive Director Prairie Provinces Water Board	Same as above.