

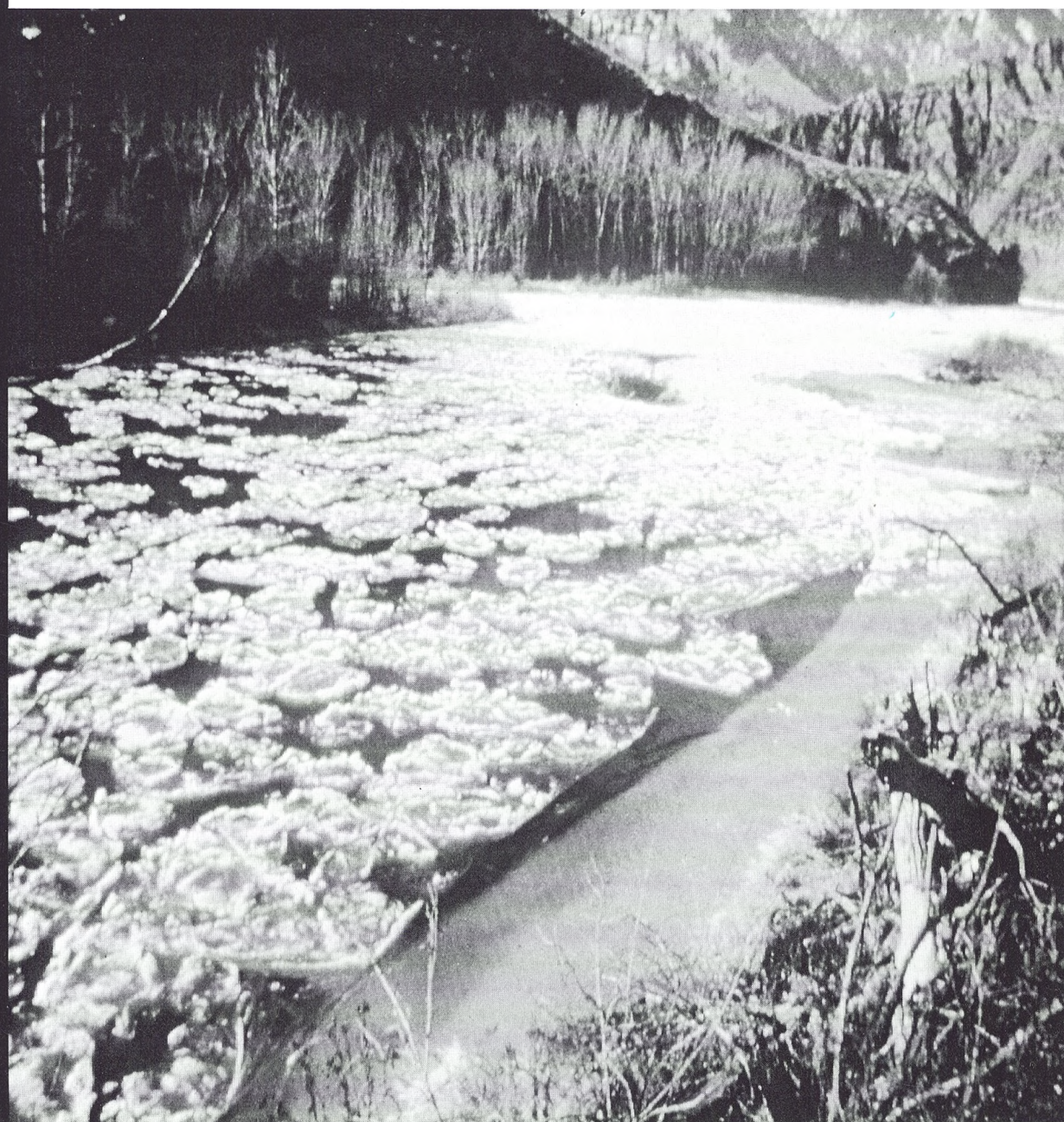
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# A Mathematical Model for River Ice Processes

A.M. Wasantha Lal and Hung Tao Shen

May 1993



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### **Abstract**

River ice processes are complex phenomena that are affected by many factors, including meteorological conditions, thermal inputs, hydraulic conditions and channel geometry. In this study a one-dimensional model called RICE is developed for simulating ice processes in rivers. In the river hydraulics component, the flow condition is determined by an implicit finite-difference solution of one-dimensional unsteady flow equations. In the thermal component, distributions of water temperature and ice concentration are determined by a Lagrangian-Eulerian solution scheme for equations of transport of thermal energy and ice. A two-layer formulation is introduced to model the ice transport. In this formulation the total ice discharge is considered to consist of the surface ice discharge of suspended ice distributed over the depth of the flow. The effect of surface ice on ice production, as well as the formation of skim ice and border ice, is included. The dynamic formation and stability of the ice cover is formulated according to existing equilibrium ice jam theories with due consideration to the interaction between the ice cover and the flow. The undercover ice accumulation is formulated according to the critical velocity criterion. The growth and decay of the ice cover is simulated using a finite-difference formulation applicable to composite ice covers consisting of snow, ice and frazil layers. The model has been applied to the St. Lawrence River and the Ohio River system, with simulated results comparing favorably with field observations. Future improvements on the mathematical model as well as theoretical formulations on various ice processes are discussed.

*Cover: Ice floes on the Salmon River, Idaho.*

For conversion of SI metric units to U.S./British customary units of measurement consult *Standard Practice for Use of the International System of Units (SI)*, ASTM Standard E380-89a, published by the American Society for Testing and Materials, 1916 Race St., Philadelphia, Pa. 19103.

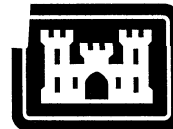
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# A Mathematical Model for River Ice Processes

A.M. Wasantha Lal and Hung Tao Shen

May 1993

## **PREFACE**

This report was prepared by A.M. Wasantha Lal, Research Engineer, Department of Civil and Environmental Engineering, and Hung Tao Shen, Professor of Civil and Environmental Engineering, Clarkson University. The study was supported by the U.S. Army Corps of Engineers under Contract No. DACA 89-84-K-0008, under the Clarkson–CRREL Joint Graduate Program in Ice Engineering. The report was technically reviewed by Dr. George Ashton and Steven Daly, both of CRREL.

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# **A Mathematical Model for River Ice Processes**

A. M. WASANTHA LAL AND HUNG TAO SHEN

## **INTRODUCTION**

Mathematical modeling techniques are increasingly being used to analyze river hydraulics. The numerical simulation of river hydraulics under open water conditions using one- or two-dimensional models is at an advanced stage due to the developments in the last few decades (Mahmood and Yevjevich 1975, Cunge et al. 1980, Fread 1985). When a river is subjected to ice conditions, the modeling of ice processes becomes important, not only because of the need to understand river ice phenomena, but also because of their effect on the hydraulic condition.

The river ice phenomena include the formation, evolution, transport, dissipation and deterioration of various forms of ice. These processes are not only affected by the ambient atmospheric conditions and the river geometry, they also interact in a complex manner with the flow condition in the river. Many river ice processes are not completely understood. However, by using existing theories, a comprehensive mathematical model can be developed. Such a model can be used to provide a continuous description of the river ice phenomena based on a limited amount of field data. From the engineering point of view, the model will assist engineers in evaluating the possible beneficial and detrimental consequences of the design and operation of river control works. A mathematical model can also be used by researchers to identify crucial gaps and weaknesses in the current knowledge of river ice. In addition, by interpreting the field data in a comprehensive manner, mathematical models can also be used to test new hypothetical formulations to advance our understanding of river ice processes. Since field data are often incomplete and the quantitative extrapolation from laboratory physical models to field conditions is still uncertain, this last aspect is particularly useful in river ice research.

Because of the complexity of river ice phenomena and the limited current understanding (Shen 1985), the development of a river ice model requires the maximum use of existing theories and mathematical techniques. In recent years a number of river ice models have been developed. Most of these models use backwater analysis in the hydraulic computation. These models include ones developed by Simonsen and Carson (1977), Petryk and Boisvert (1978), Michel and Drouin (1981), Petryk et al. (1981a) and Calkins (1984). Ice concentrations and water temperature distributions along the river are not considered in these models. Shen and Yapa (1984) developed a model that uses unsteady flow analysis in the hydraulic computation. The model also simulates water temperature and ice concentration distributions along the river by considering the longitudinal transport of thermal energy and frazil ice. A simulation model has been developed recently by Houkuna (1988) based on the model of Shen and Yapa and the unsteady flow model developed by Fread (1985). This model, however, does not consider mechanical thickening of the ice cover. Almost all of these models simulate river ice processes with various degrees of simplifications in the analytical formulation of physical phenomena.

### **River ice processes**

At the beginning of winter, heat loss from the water surface due to the low ambient air temperature will exceed heat gain due mainly to the short-wave radiation. As a result the water temperature can drop to the freezing point. Further cooling will lead to supercooling of the river water and frazil ice formation. In slow flow areas,

where the turbulence intensity is not strong enough to mix the water or the frazil crystals over the depth, skim ice can form in the river even before the cross-sectional averaged water temperature drops to the freezing point. Frazil ice crystals, when mixed over the flow depth due to turbulence, can grow in size, multiply in number and agglomerate to form porous flocs. As the frazil particles and flocs grow in size, the increase in buoyancy can lead to the formation of a moving surface ice layer with a much higher concentration than the ice remaining in suspension.

The continuous increase in the surface ice concentration may lead to the formation of ice pans. Ice pans grow in size and strength when they travel along the river due to the freezing of interstitial water and the further accumulation of frazil ice, both around the circumference and on the bottom of pans. Ice pans may either sinter into still larger ice floes if they travel over a long distance, or they may break into fragments when passing through rapid sections. Partial coverage of the water surface by the moving surface ice layer results in a reduction of the net ice production due to the insulating effect. The downstream transport of the surface ice will cease when it reaches an artificial obstacle or a river section where an ice bridge across the river is formed by the congestion of the surface ice run. Once an obstacle is reached, the incoming surface ice will accumulate at its upstream side and extend the ice cover upstream. Frazil ice remaining in the suspension underneath the surface layer will be transported downstream and deposited on the underside of the ice cover to form frazil ice accumulations or hanging dams.

The phenomenon of ice bridging is not well understood, even though ice bridges usually form at the same location each winter. The formation of an ice bridge at a river section is related to the ice transport capacity of the section and the rate of ice discharge coming from upstream. The maximum rate of ice discharge that can pass through a river without forming an ice bridge depends on the flow velocity, the channel top width between banks or border ice boundaries, the surface slope, and the size, concentration and material properties of the ice in the surface layer (Ackermann and Shen 1983, Matousek 1984b, Shen et al. 1988).

Once an ice cover is initiated, it may progress upstream through the accumulation of incoming surface ice floes and slush. The rate of progression of the leading edge of an ice cover depends on the rate of surface ice supply and the thickness of the new ice cover, which is governed by the flow conditions at the leading edge. When the flow velocity is relatively low, incoming ice floes will form a smooth ice cover accumulated by a single layer of ice floes. This process is often called juxtaposition. During the freeze-up period, when the surface ice consists mainly of highly deformable frazil slush or loose frazil pans, the juxtaposition mode may not be clearly identifiable due to the compression of the surface ice elements. At a higher velocity range, surface ice elements can overturn or submerge at the leading edge to form a thicker ice cover. This mode of ice cover formation is often termed narrow jam or hydraulic thickening (Pariset and Hausser 1961). In this mode the ice cover thickness is limited by a critical under-cover entrainment velocity. When this velocity is exceeded, the ice particles that are swept under at the leading edge will be washed downstream, leading to the cessation of the leading edge progression, until flow velocity at the leading edge is reduced. Such a reduction in velocity can be caused either by the change in river discharge or the backwater effect induced by the increase in the size of the under-cover accumulation of ice.

The cover formed under any hydraulic condition has to be thick enough so that it is capable of withstanding the net force acting on it. Forces acting on an ice cover include current drag, wind drag, weight of the ice cover, and bank shear. The ice cover can collapse any time during the winter when the strength of the ice cover, together with the bank shear, cannot support the external forces acting along the direction of flow. During freeze-up, mechanical thickening or “shoving” will occur until the cover reaches a thickness that is capable of withstanding the external forces. The freeze-up of a thin solid ice cover in the granular surface ice accumulation can significantly increase the strength of the initial ice cover.

The surface ice that is swept under the ice cover can travel along the underside of the ice cover and be deposited downstream. This deposition, together with the similar deposition of frazil ice that remained in suspension when entering the ice-covered region, can lead to the formation of frazil hanging dams or frazil ice jams. Frazil hanging dams or frazil ice jams often occur under ice covers downstream of rapids. If the flow velocity increases at a later period, loosely accumulated frazil ice dams can be eroded and transported farther downstream.

These surface and under-cover ice accumulation processes have been presented in the context of ice cover



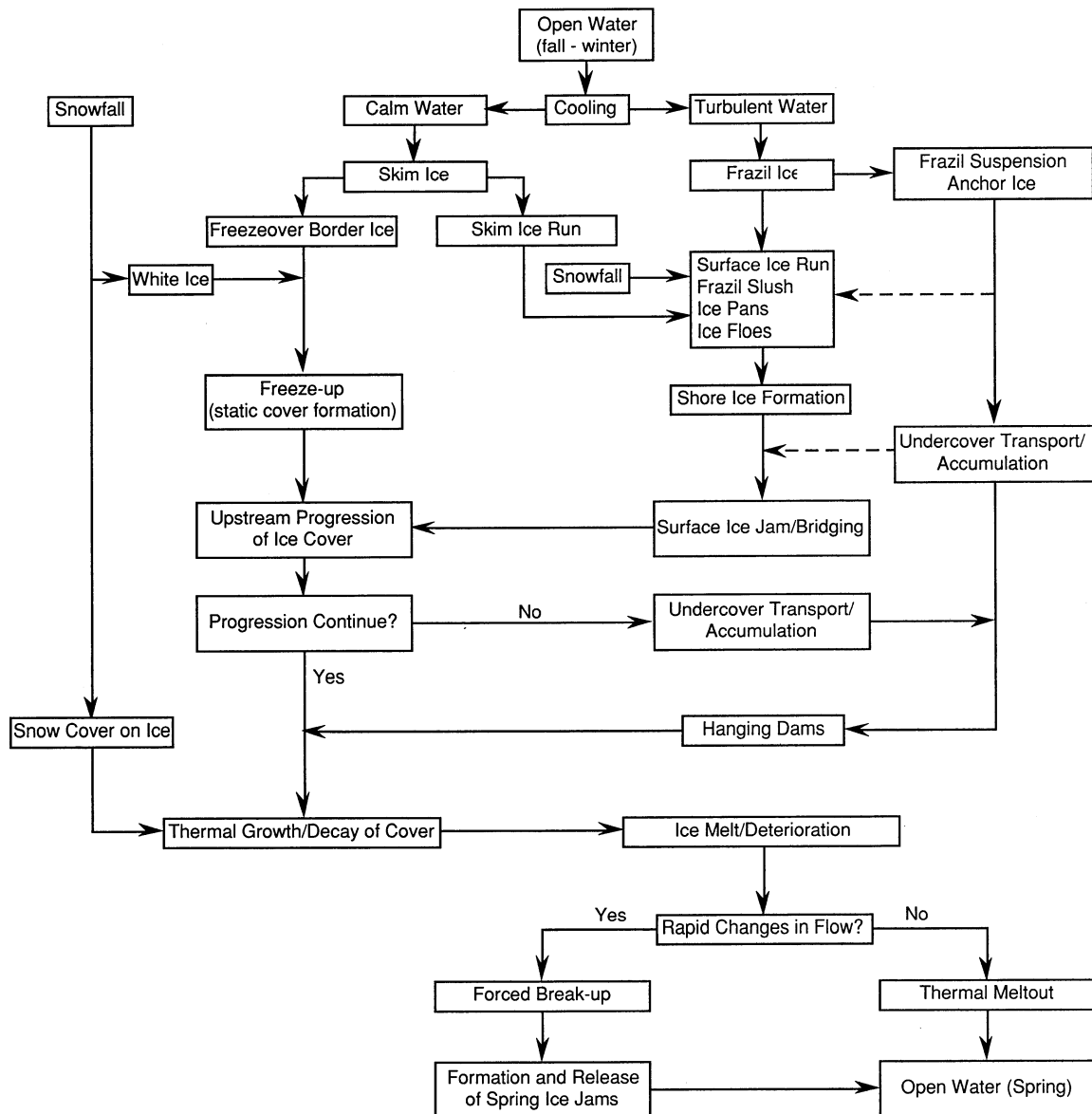


Figure 1. River ice processes.

formation during freeze-up. Similar processes can occur when a large volume of surface ice floes is released during the break-up of the ice cover upstream. In this case the leading edge of the intact downstream ice cover often acts as the obstacle that initiates the ice accumulation process. The dynamic process of the movement and accumulation of ice fragments after the break-up of the ice cover is similar to that of the surface ice run during freeze-up. Both of these processes involve relatively small time scales.

As heat exchange continues over a consolidated ice cover, water-filled voids in the granular ice mass will freeze from the water level downwards. This process is faster than black ice growth, and it can extend into the frazil layer if it exists. The existence of a snow cover can affect the thermal growth and decay of the ice cover. The snow cover provides an insulation layer that can retard the growth of the ice cover thickness. However, when the snow-ice interface submerges below the water level, the snow-ice growth on top of the cover can accelerate the growth of the ice cover thickness. Figure 1 shows a brief summary of these river ice processes.

## Objective of the study

In this study a comprehensive one-dimensional river ice model called RICE was developed. The model includes submodels for river hydraulics, distributions of water temperature and ice discharge, formation of ice covers, formation and erosion of under-cover ice accumulations or hanging dams, thermal growth and decay of ice covers, and stability of ice covers. Existing theories on each ice process were reviewed and weaknesses identified. The model makes maximum use of the existing information, with improvements on analytical and mathematical formulations. Each ice process is modeled in a separate subroutine. With this arrangement the model can easily be modified to accommodate future improvements when new theories become available. The model can be used as a diagnostic tool to determine important factors that control the ice conditions in a river. This will provide guidelines for developing systematic field programs of data collection, designing of ice control projects and planning flow regulations. From the point of view of ice research, it is expected that by applying the model to rivers with existing field data, the shortcomings in the existing theories can be identified for future improvements. Together with weather forecasts, the model should also be able to forecast ice conditions in a river. Because of the limitations in the current knowledge on river ice, the model should first be calibrated with the field data before being used in forecasting.

## UNSTEADY FLOW COMPUTATIONS

### Governing equations

The hydraulics of one-dimensional river flow can be described by Saint Venant equations. These equations represent the conservation of mass and momentum in the river. For a river with a floating ice cover, the equation of conservation of mass is given as

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \quad (1)$$

where  $Q$  = discharge  
 $A$  = net flow cross-sectional area  
 $x$  = distance  
 $t$  = time.

The momentum equation is given as

$$\rho \frac{\partial Q}{\partial t} + \rho \left( \frac{2Q}{A} \frac{\partial Q}{\partial x} - \frac{Q^2}{A^2} \frac{\partial A}{\partial x} \right) + \rho g A \frac{\partial H}{\partial x} + (p_i \tau_i + p_b \tau_b) = 0 \quad (2)$$

where  $H$  = water surface elevation ( $H = z_b + d_w + t_{sub}$ )  
 $g$  = gravitational acceleration  
 $z_b$  = bed elevation  
 $d_w$  = depth of flow  
 $p_b$  = wetted perimeters formed by the channel bed  
 $p_i$  = wetted perimeters formed by the ice cover  
 $\tau_b$  = shear stresses at the channel bottom  
 $\tau_i$  = shear stresses at the ice/water interface  
 $t_{sub}$  = submerged thickness of the ice cover.

### Numerical formulation

#### *Finite-difference formulation of St. Venant equations*

Finite-difference discretization for St. Venant equations requires the river to be divided into a sufficient number of reaches. The discretization should be done in such a manner that the average of the cross sections at the upstream and downstream ends of any reach closely represent the river reach. Junctions where different river branches meet have to be considered as nodes. Schematization of a river involves numbering river reaches and nodes. A method of numbering nodes and reaches for efficient computation is explained later.

The four-point implicit method (e.g. Amein and Chu 1975, Potok 1978, Potok and Quinn 1979) has many advantages over other methods, and it is capable of simulating a wide spectrum of wave types and river characteristics (Fread 1985). In this study the four-point implicit model for river networks developed by Potok and Quinn (1979) is used. The finite-difference formulation of St. Venant equations is presented in this section. In the finite-difference scheme, the river is discretized into a series of reaches. These reaches are connected by cross sections at their upstream and downstream ends. These cross sections are called nodes. The notation used is shown in Figures 2 and 3. The finite-difference grid on the  $x-t$  plane is shown in Figure 4. The finite-difference form of the continuity equation can be written as

$$\left[ \theta (Q_d^p - Q_u^p) + (1 - \theta) (Q_d - Q_u) \right] \frac{1}{\Delta x} + \left[ T_d (H_d^p - H_d) + T (H_u^p - H_u) \right] \frac{1}{2\Delta t} = 0 \quad (3)$$

where  $Q$  = discharge

$H$  = stage

$d, u$  = subscripts representing downstream and upstream ends of the reach

$p$  = superscript indicating the solution at time level  $t + \Delta t$

$T$  = top width

$\theta$  = weighting factor =  $\Delta t_m / \Delta t$

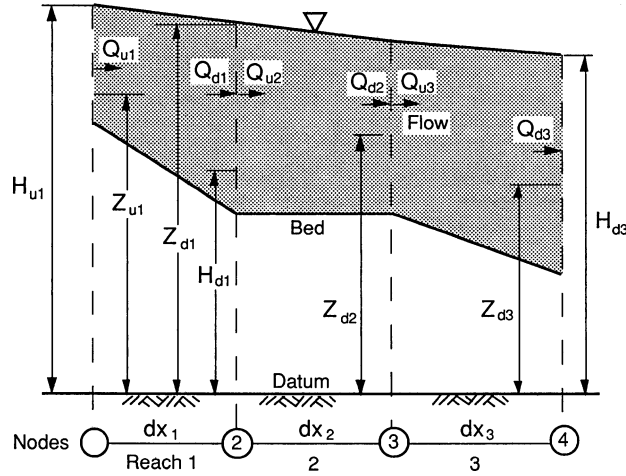


Figure 2. Example of finite-difference discretization.

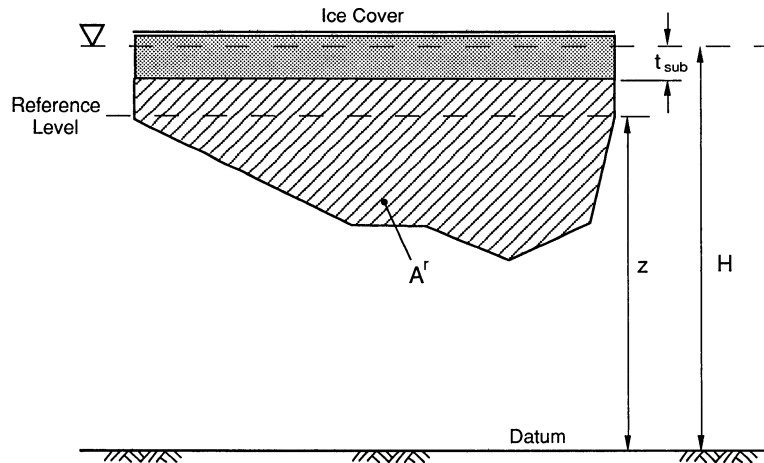


Figure 3. Channel cross section with an ice cover.

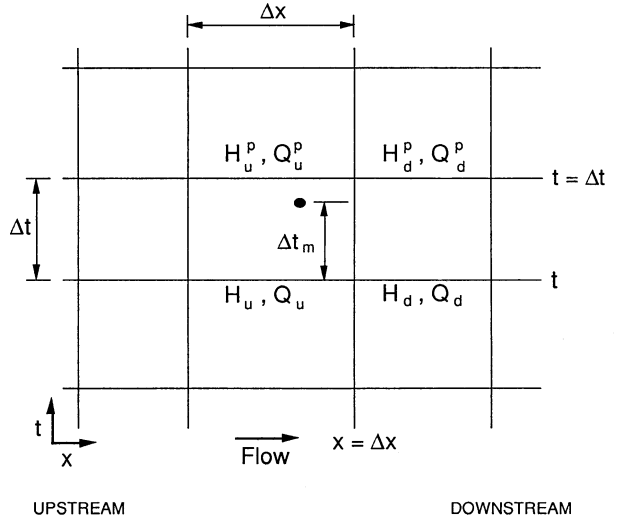


Figure 4. Implicit finite-difference scheme.

$\Delta x$  = length of a reach

$\Delta t$  = time step

$\Delta t_m$  = fraction of  $\Delta t$  as shown in Figure 4 .

To ensure numerical stability and accuracy, the value of  $\theta$  should be greater than 0.5 (Fread 1985). When  $\theta = 1$ , the scheme is fully implicit and unconditionally stable, except that the accuracy is reduced to  $O(\Delta t)$ . A value of 0.75 has been suggested to ensure stability and minimize loss of accuracy. The finite-difference form of the momentum equation can be written as

$$\begin{aligned}
 & \left[ \theta (Q_d^p - Q_u^p) + (1 - \theta) (Q_d - Q_u) \right] \frac{2\bar{Q}^2}{A\Delta x} - \frac{\bar{Q}^2}{A^2} \left\{ \theta [T_d (H_d^p - Z_d) - T_u (H_u^p - Z_u) + \Delta A^r] \right. \\
 & \quad \left. + (1 - \theta) [T_d (H_d - Z_d) - T_u (H_u - Z_u) + \Delta A^r] \right\} \frac{1}{\Delta x} \\
 & \quad + g\bar{A} \left[ \theta (H_d^p - H_u^p) + (1 - \theta) (H_d - H_u) \right] \frac{1}{\Delta x} \\
 & \quad + gn_e^2 \bar{Q} |\bar{Q}| \frac{\bar{P}^{4/3}}{A^{7/3}} + [Q_d^p + Q_u^p - (Q_d + Q_u)] \frac{1}{2\Delta t} = 0
 \end{aligned} \tag{4}$$

$$\text{where } \Delta A^r = A_d^r - A_u^r - t_{\text{sub}} (T_d - T_u) \tag{5}$$

$$\bar{Q} = 0.5 [\theta (Q_d^p + Q_u^p) + (1 - \theta) (Q_d + Q_u)] \tag{6}$$

$$\bar{A} = 0.5 \{ A_d^r + A_u^r + \theta [H_u^p T_u + H_d^p T_d] + (1 - \theta) [H_u T_u + H_d T_d] - (Z_u + t_{\text{sub}}) T_u - (Z_d + t_{\text{sub}}) T_d \} \tag{7}$$

and  $z_u$  and  $z_d$  = reference elevations of upstream and downstream, respectively

$A_u^r$  and  $A_d^r$  = cross-sectional area of the river below reference levels at upstream and downstream, respectively

$g$  = gravitational acceleration

$\bar{P}$  = average wetted perimeter of the channel branch

$n_e$  = equivalent Manning's roughness coefficient

$t_{\text{sub}}$  = submerged depth of the ice cover thickness.

In eq 4,  $n_e$  represents the gross Manning's roughness coefficient of the river reach. It includes the effects of both the flow resistance of the channel bed and the resistance of the ice cover. In the absence of an ice cover,

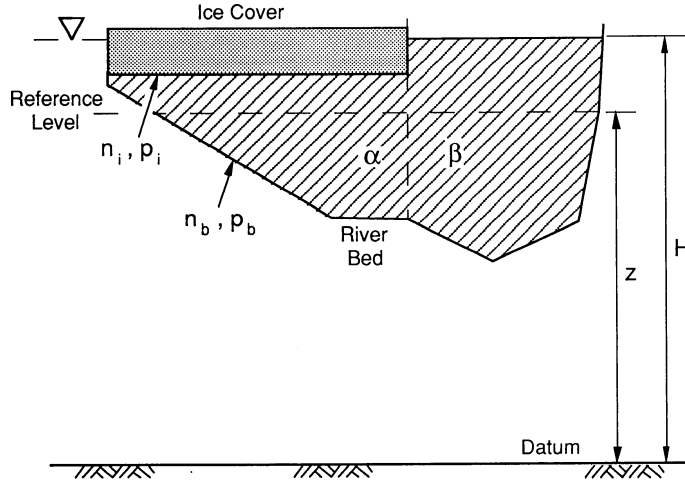
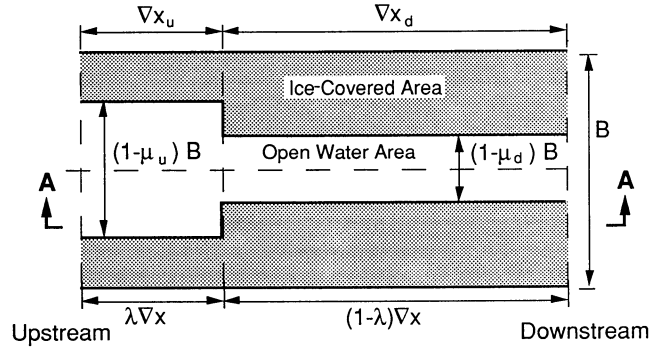
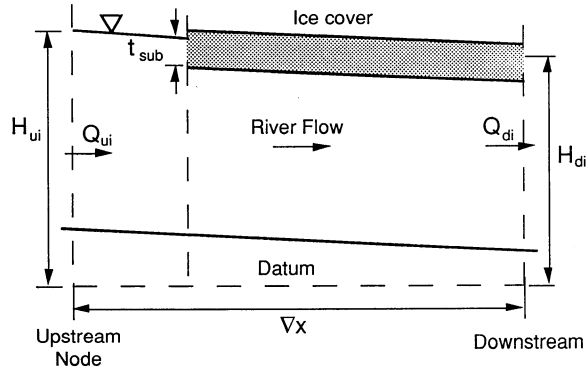


Figure 5. Partially ice-covered river cross section.



a. Top view.



b. Longitudinal section.

Figure 6. Partially ice-covered river reach.

$n_e = n_b$ . The derivation for a general expression of  $n_e$  is presented below. The present model considers the possibility of having border ice covering a fraction of the width and the main ice cover extending over a fraction of the length of a reach. Figure 5 shows a partially ice-covered river cross section. A typical plan area of an ice-covered reach considered in the model is shown in Figure 6, along with its longitudinal section. This river reach has a downstream fraction  $\lambda$  of length covered, except for a fraction  $(1 - \mu_d)$  of the width. This type of partial ice cover can exist due either to the formation of border ice or to the erosion of the ice cover along a deep channel or thalweg in a river reach. The upstream fraction  $(1 - \lambda)$  of the length is considered open, except for a fraction  $(1 - \mu_u)$  of the width.



In the general case of a partially covered cross section as shown in Figure 5, the friction slope and shear resistance at any river cross section can be obtained using

$$S_f = \frac{p_b \tau_b + p_i \tau_i}{\rho g A} = \frac{(p_b \tau_b)_O + (p_b \tau_b)_I + (p_i \tau_i)_I}{\rho g A} \quad (8)$$

where the subscripts  $I$  and  $O$  represent covered and open portions of the cross section, respectively. When a fraction of the length of the river reach is covered with ice of a different width, the average energy slope of the river reach can be determined by considering the head loss  $h_f$  within a reach as the sum of the losses in the upstream and downstream sections of the reach. The total head drop over the length of the reach is given by

$$h_f = \bar{S}_f \Delta x = S_{fu} \Delta x_u + S_{fd} \Delta x_d \quad (9)$$

which reduces to

$$\bar{S}_f = (1-\lambda) S_{fu} + \lambda S_{fd} \quad (10)$$

where  $\bar{S}_f$  = average friction slope over the length of the reach  
 $S_{fu}$  = friction slope of the upstream  $(1-\lambda)$  fraction of the reach  
 $S_{fd}$  = friction slope of the downstream  $\lambda$  fraction of the river  
 $\lambda = \frac{\Delta x_d}{\Delta x}$ .

Shen and Ruggles (1982) showed that

$$p_b \tau_b + p_i \tau_i = \rho g n_b^2 \left[ \left( \frac{Q^2 p_b^{4/3}}{A^{7/3}} \right)_O + (1+F)^{4/3} \left( \frac{Q^2 p_b^{4/3}}{A^{7/3}} \right)_I \right] \quad (11)$$

where the subscripts  $I$  and  $O$  indicate ice-covered and open portions of the cross sections, respectively.  $F$  is defined as

$$F = \left( \frac{n_i}{n_b} \right)^{3/2} \left( \frac{p_i}{p_b} \right)_I \quad (12)$$

and  $n_i$  is the Manning coefficient for ice cover. The time-dependent variation of  $n_i$  can be related to an initial ice cover roughness  $n_{ini}$  and a final ice roughness  $n_{end}$  using the following formula (Nezhikhovskiy 1964, Shen and Yapa 1986):

$$n_i = n_{end} + (n_{ini} - n_{end}) e^{-kt} \quad (13)$$

Values of  $n_{ini}$  and  $n_{end}$  have to be calibrated prior to the application of the model.

In the case of a river with no ice cover, eq 8 and 11 give

$$S_f = \frac{P_b \tau_b}{\rho g A} = \frac{\rho g n_b^2 P_b^{4/3} Q |Q|}{\rho g A^{10/3}} \quad (14)$$

If the ice cover extends over the entire width of the river, eq 11 reduces to

$$S_f = \frac{P_i \tau_i + P_b \tau_b}{\rho g A} = \frac{\rho g n_b^2 P_b^{4/3} Q |Q| (1+F)^{4/3}}{\rho g A^{10/3}} \quad (15)$$

For a river reach that is fully covered over part of its length and fully open over the remaining part,  $S_{fu}$  is obtained using eq 14 and  $S_{fd}$  is obtained using eq 15. The average friction slope over the reach can be obtained using eq 10:

$$\bar{S}_f = Q^2 n_b^2 \left[ (1 - \lambda) \frac{P_u^{4/3}}{A_u^{10/3}} + \lambda (1 + F)^{4/3} \frac{P_d^{4/3}}{A_d^{10/3}} \right]. \quad (16)$$

When the channel cross section is relatively uniform in a reach and the cross-sectional area occupied by the ice cover is small in comparison to the total cross-sectional area, both  $P_u$  and  $P_d$  can be replaced by an average value  $P$  for the reach. Similarly  $A_u$  and  $A_d$  can be replaced by the average cross-sectional area  $\bar{A}$  for the reach. With these approximations, eq 16 reduces to

$$\bar{S}_f = \frac{Q^2 n_b^2 P^{4/3}}{\bar{A}^{10/3}} [(1 - \lambda) + (1 + F)^{4/3} \lambda]. \quad (17)$$

Using eq 17, an expression for  $n_e$  can be obtained to give an equivalent roughness in Manning's relationship (Yapa 1983):

$$n_e^2 = n_b^2 \left[ \frac{\Delta x_o}{\Delta x} + (1 + F)^{4/3} \frac{\Delta x_c}{\Delta x} \right]. \quad (18)$$

The same approach can be made to find the friction slope for a case where the ice extends over part of the width  $(1 - \mu)B$ , assuming the channel to be a wide rectangular such that  $p_\alpha \approx \mu B$ ,  $A_\alpha \approx \mu A$ , etc. Applying Manning's formula to the entire channel section and the ice-covered and open portions, respectively, and assuming the ice cover to be relatively thin in comparison to the flow depth, yields

$$Q = \frac{A R^{2/3} S_f^{1/2}}{n_e} \quad (19)$$

$$Q_\alpha = \frac{\mu A}{n_c} \left( \frac{\mu A}{2\mu B} \right)^{2/3} S_f^{1/2} = \frac{\mu A^{5/3}}{n_c 2^{2/3} B^{2/3}} S_f^{1/2} \quad (20)$$

$$Q_\beta = \frac{(1-\mu)A}{n_b} \left[ \frac{(1-\mu)A}{(1-\mu)B} \right]^{2/3} S_f^{1/2} = \frac{(1-\mu)A^{5/3}}{n_b B^{2/3}} S_f^{1/2}. \quad (21)$$

The continuity condition gives

$$Q = Q_\alpha + Q_\beta. \quad (22)$$

Substituting  $Q_\alpha$ ,  $Q_\beta$ ,  $p_\alpha$ ,  $p_\beta$ ,  $A_\alpha$  and  $A_\beta$  into eq 11 yields the following:

$$S_f = \frac{B^{4/3} n_b^2 Q^2}{A^{7/3}} \frac{(1 + F)^{4/3}}{[\mu + (1 - \mu)(1 + F)^{2/3}]^2}. \quad (23)$$

Equation 23 can be used to obtain  $S_{fu}$  and  $S_{fd}$  of eq 10 by applying  $S_f = S_{fu}$  when  $\mu = \mu_u$  and  $S_f = S_{fd}$  when  $\mu = \mu_d$ . A general expression for the average friction slope  $\bar{S}_f$  can now be expressed as

$$\bar{S}_f = \frac{Q^2 B^{4/3} n_b^2}{A^{10/3}} \left\{ \frac{(1-\lambda)(1+F)^{4/3}}{[\mu_u + (1-\mu_u)(1+F)^{2/3}]^2} + \frac{\lambda(1+F)^{4/3}}{[\mu_d + (1-\mu_d)(1+F)^{2/3}]^2} \right\}. \quad (24)$$

The resistance term  $(p_i \tau_i + p_b \tau_b)$  in eq 2 can now be determined by using eq 8. If the ice cover extends over the entire width in the downstream reach as a result of progression,  $\mu_d = 1$ . If there is no border ice,  $\mu_u = 0$ , and eq 24 reduces to eq 17.

#### Nodal equations

The equation governing the flow continuity at node  $k$  can be expressed as

$$\sum_{i=1}^{\text{in}} Q_{d,i}^p + q_k = \sum_{j=1}^{\text{out}} Q_{u,j}^p \quad (25)$$

where  $Q$  = discharge

$i$  = incoming reaches

$j$  = outgoing reaches

$u$  and  $d$  = subscripts denoting upstream and downstream ends of each reach

$q_k$  = external flow at node  $k$ .

The energy balance for the node  $k$  can be expressed using the following equations:

$$H_{d,1}^p = H_{u,i}^p + \Delta H_i \quad \text{for } i = 2, \text{ in} + \text{out} \quad (26)$$

where  $H$  = water level

$(\text{in} + \text{out})$  = total number of incoming and outgoing reaches for the node

$\Delta H_i$  = head loss between reaches 1 and  $i$  due to control structures at the node.

The variable  $\Delta H_i$  is a function of  $Q_{d,1}$ ,  $H_{d,1}$  and  $H_{u,1}$  etc. There are a total of  $(\text{in} + \text{out} - 1)$  such equations at each internal node.

#### External boundary conditions

Three types of boundary conditions are common in practice. They are the discharge, the water level and the control structure type. In the case of a control structure, a functional relationship between the discharge and the water level is necessary. Only one time history or stage–discharge relationship is required at each boundary. Special treatment may be needed when a downstream control structure is not available in a long river. In such a case the downstream boundary can be treated by satisfying the uniform flow condition at the downstream boundary as presented by Viessman et al. (1972). An alternative method is to use an approximate rating curve developed from the channel characteristics as suggested by Gunaratnam and Perkins (1970). In the case of a boundary node connected to a reservoir having a known inflow and plan area, the continuity condition for the reservoir can be used as the boundary condition (Potok and Quinn 1979).

The general functional form of an upstream and a downstream boundary conditions can be expressed as

$$F_u (H_u^p, Q_u^p) = 0 \quad (27)$$

and

$$F_d (H_d^p, Q_d^p) = 0 \quad (28)$$

where  $F_u$  and  $F_d$  represent known functions. When the head is known at the upstream boundary,

$$F_u(H_u^p, Q_u^p) = H_{u,1}^p - H_{ub} \quad (29)$$

where  $H_{ub}$  is the known head at the upstream boundary. When the discharge is known,

$$F_u(H_u^p, Q_u^p) = Q_{u,1}^p - Q_{ub} \quad (30)$$

where  $Q_{ub}$  is the known discharge at the upstream boundary. Similar expressions can be obtained for the downstream boundary. When the rating curve  $Q = f(H)$  is used, then for the downstream boundary,

$$F_d(H_d^p, Q_d^p) = Q_d^p - f(H_d^p) = 0. \quad (31)$$

#### Initial condition

If the initial conditions of the problem are known, the simulation can start with known initial values given in the input. Otherwise, the steady state solution is commonly used as the initial condition. To obtain a steady state solution, the program can be made to run starting from assumed flow and depth conditions close to those existing in the field. A sufficient number of iterations have to be used to bring the solution to the steady state.

Fread (1985) suggested the use of a backwater computation to determine  $H$  and  $Q$  values. In this study the method suggested by Potok and Quinn (1979) is used. The initial water surface profile and discharge distributions are obtained by first setting a zero discharge and an assumed constant water level for the entire study reach. The boundary conditions are then changed to those corresponding to the first time step. The flow simulation is then run until the river profile becomes steady. The resulting profile is then used as the initial condition.

#### Method of solution

In eq 3 and 4, the unknowns are  $Q_u^p$ ,  $Q_d^p$ ,  $H_u^p$  and  $H_d^p$  for each branch. For a river system with  $NR$  branches,  $NB$  nodes (including boundary nodes) and  $NBND$  boundary conditions,  $4NR$  equations are needed to solve the  $4NR$  unknowns. A breakdown of the number of equations is given in Table 1.

**Table 1. Number of equations available in the system of equations.**

Type	Number of equations
River reach continuity condition	$NR$ (number of reaches)
River reach momentum condition	$NR$ (number of reaches)
Nodal continuity condition	$NB - NBND$ (total no. of nodes – no. of boundary nodes)
Nodal energy condition	sum of (in–out–1) at $NB - NBND$ nodes
Boundary conditions	$NBND$ (no. of boundary nodes)

The finite-difference form of the Saint Venant equations, along with the nodal and boundary conditions, form a system of nonlinear equations of the form

$$\mathbf{F}(\mathbf{x}) = 0 \quad (32)$$

where  $\mathbf{x} = (x_1, x_2, \dots, x_n)^T$  is the vector containing the unknown variables  $Q^p$  and  $H^p$ , and  $f_1, f_2, \dots, f_n$  are the coordinate functions of  $\mathbf{F}$ . Using Newton's method, the iterative procedure to find  $\mathbf{x}$  at the time step  $k$  is given as (Burden and Faires 1985)

$$J[\mathbf{x}^{(k-1)}] [\mathbf{x}^{(k)} - \mathbf{x}^{(k-1)}] = -\mathbf{F}(\mathbf{x}^{(k-1)}) \quad (33)$$

where  $J(\mathbf{x})$  is the Jacobian matrix. For computational efficiency,  $\mathbf{x}$  is solved as a system of linear equations. The  $(i,j)$  element of the Jacobian matrix can be written as

**Table 2. Arrangement of a Jacobian matrix and the right side of eq 30 for the case shown in Figure 2.**

Equation	Reach 1				Reach 2				Reach 3 (NR)				RHS
	H <sub>u</sub>	Q <sub>u</sub>	H <sub>d</sub>	Q <sub>d</sub>	H <sub>u</sub>	Q <sub>u</sub>	H <sub>d</sub>	Q <sub>d</sub>	H <sub>u</sub>	Q <sub>u</sub>	H <sub>d</sub>	Q <sub>d</sub>	
Upstream boundary	A	B	0	0	0	0	0	0	0	0	0	0	C
Continuity: reach 1	X	X	X	X	0	0	0	0	0	0	0	0	X
Momentum: reach 1	X	X	X	X	0	0	0	0	0	0	0	0	X
Continuity: node 2	0	0	0	X	X	0	0	0	0	0	0	0	X
Energy: node 2	0	0	X	0	X	0	0	0	0	0	0	0	X
Continuity: reach 2	0	0	0	0	X	X	X	X	0	0	0	0	X
Momentum: reach 2	0	0	0	0	X	X	X	X	0	0	0	0	X
Continuity: node 3	0	0	0	0	0	0	0	X	0	X	0	0	X
Energy: node 3	0	0	0	0	0	0	X	0	X	0	0	0	X
Continuity: reach 3	0	0	0	0	0	0	0	0	X	X	X	X	X
Momentum: reach 3	0	0	0	0	0	0	0	0	X	X	X	X	X
Downstream boundary	0	0	0	0	0	0	0	0	0	0	D	E	F
(Jacobian matrix, 12*12)													(RHS, 12*1)

Key: X = Variables taken care of by the program.

A,B,C,D,E,F = Variables defined by user in subroutines UPBOUND and DNBOUND to take care of boundary conditions.

$$J_{ij}(\mathbf{x}) = \frac{\partial f_i(\mathbf{x})}{\partial x_j}. \quad (34)$$

At each computational time step, the above iterative procedure is followed to obtain the unknowns  $\mathbf{x}$ , which are the  $H$  and  $Q$  values of the river reaches. The selection of initial values  $\mathbf{x}^0$  is explained later. An alternative method for solving a system of nonlinear equations is available when the analytical expression of  $J(\mathbf{x})$  cannot be obtained easily (Broyden 1965).

By selecting a suitable numbering system for nodes and reaches, the bandwidth of the Jacobian matrix can be reduced to a minimum. This is a very useful technique to reduce computing time and obtain accurate results (Potok 1978). In the method used, nodes have to be numbered along the river from the upstream end of the river to the downstream end of the downstream boundary reach. By doing this, external boundary conditions would not come into the center of the Jacobian matrix. Also, the difference between two neighboring reaches and nodes has to remain at a minimum, which is a condition for minimum bandwidth. Reach numbers should follow node numbers. It is preferable to have the reach number be the same as the node number at the upstream end of the reach. This condition is not possible in the case of river networks. Table 2 shows an example of a numbering system for a river divided into three reaches.

### Calibration of resistance coefficients

A river system consisting of many river reaches can be considered as a discrete system, and the Manning coefficient of each section can be calibrated using influence coefficient methods (Beck and Arnold 1977, Lal and Shen 1990b). Since influence coefficients can be obtained numerically, the calibration methods do not require any manipulation of the governing equations. The numerical model for simulating the unsteady flow can be used without further modification. Observed and simulated values of any number of state variables (water levels) can be used to calibrate a number of parameters equal to the number of state variables.

The calibration uses observations of the state variable  $Y_k$ , for each of  $i = 1, 2, \dots, n$  sensors (water level gauges) at time steps  $k = 1, 2, \dots, m$ . The data are used to calibrate the parameters,  $\theta_i$ ,  $i = 1, 2, \dots, n$  (Manning's roughness coefficients). Three influence coefficient methods, depending on the selection of the objective function, can be used. These methods are the least-square method, the minimax method and the minimum bias method. The objective function used when applying the principle of least squares is the minimization of the error sum of squares  $S$  over all the sensors and all the time steps. The expression for  $S$  is

$$S = \sum_{i=1}^n \sum_{k=1}^m (y_i^k - Y_i^k)^2 \quad (35)$$



where  $y_i^k$  is the simulated value of the  $i^{th}$  state variable at time  $k$ . The objective function of the minimax method is to minimize the sum of maximum errors in the simulation:

$$\min e_1 + e_2 + \dots + e_n \quad (36)$$

where  $e_1, e_2, \dots, e_n$  are the maximum errors at gauges 1, 2,  $\dots, n$ . The objective function of the minimum bias method is to minimize the absolute value of the overall bias of simulated values at all the gauging stations. The overall bias at any gauge  $i$  is given by

$$B_i = \sum_{k=1}^m (y_i^k - Y_i^k). \quad (37)$$

The objective is to minimize  $|B_i|$  for each  $i = 1, 2, \dots, n$ . This objective is achieved by making each element zero. The current study uses the minimum bias method to obtain values of Manning's coefficients, since the method is more efficient and gives accurate results.

## WATER TEMPERATURE AND ICE CONCENTRATION DISTRIBUTIONS

### Governing equations

In a well-mixed river, distributions of water temperature and depth-averaged frazil concentration along the river can be described by the one-dimensional advection–diffusion equation (Fischer et al. 1979). The equation for the cross-section-averaged water temperature  $T_w$  of a river is

$$\frac{\partial}{\partial t} (\rho C_p A T_w) + \frac{\partial}{\partial z} (Q \rho C_p T_w) = \frac{\partial}{\partial x} \left( A E_x \rho C_p \frac{\partial T_w}{\partial x} \right) - B \phi_T \quad (38)$$

where  $A$  = cross-sectional area of the river

$B$  = river width

$Q$  = discharge

$\rho$  = density of water

$C_p$  = specific heat of water

$E_x$  = longitudinal dispersion coefficient

$\phi_T$  = total heat loss rate per unit surface area of the river.

Two boundary conditions and an initial condition are needed to solve this equation.

If the river does not have large temperature gradients in the longitudinal direction, the longitudinal dispersion term

$$\frac{\partial}{\partial x} \left( A E_x \rho C_p \frac{\partial T_w}{\partial x} \right)$$

can be neglected. An analysis of the water temperature data in the upper St. Lawrence River shows that typical values of  $\partial T_w / \partial t$ ,  $\partial T_w / \partial x$  and  $\partial^2 T_w / \partial x^2$  are on the orders of  $10^{-6}$ ,  $10^{-5}$  and  $10^{-8}$ , respectively. Without the dispersion term, the solution procedure simplifies. When flow is unidirectional, only one boundary condition is needed at the upstream end:

$$T_w(0, t) = T_1(t) \quad (39)$$

where  $T_1(t)$  is the time history of the water temperature at the upstream boundary. If the initial condition  $T_w(x, 0)$  is not available, a steady state solution can be used as in the case of the hydraulics computation.

When the water temperature drops below the freezing point, frazil ice will be produced. The one-dimensional equation governing the transport of frazil ice is (Shen and Chiang 1984)

$$\frac{\partial}{\partial t} (\rho_i L_i A C_i) + \frac{\partial}{\partial x} (Q \rho_i L_i C_i) = \frac{\partial}{\partial x} \left( A E_x \rho_i L_i \frac{\partial C_i}{\partial x} \right) - B \phi_T + \Sigma S \quad (40)$$

where  $C_i$  = ice concentration

$L_i$  = latent heat of fusion of water

$\rho_i$  = density of ice

$B$  = width of the river

$\Sigma S$  = additional source or sink terms due to ice cover progression, erosion, deposition and melting.

The total heat flux term  $\phi_T$  may be approximated by the surface heat flux  $\phi^*$ . Since eq 40 is in the same form as eq 38, both the ice concentration and the water temperature can be obtained from the solution of a single equation by considering that  $C_i \rho_i L_i = -\rho C_p T_w$  and letting the water temperature in the heat exchange term  $\phi_T$  in eq 40 be equal to the freezing point of water (Shen and Chiang 1984).

### Heat exchanges

During open water conditions the total heat flux at the free surface is equal to the sum of the net short- and long-wave radiations, the sensible heat exchange and the heat transfers due to evaporation and precipitation. Some of these components, especially the long-wave radiation, are nonlinear functions of air and water temperatures. In this model, linear relationships using constant heat exchange coefficients are used to express the net heat exchange. These relationships can be obtained using multiple linear regression of computed heat exchange and weather parameters (Lal and Shen 1990a).

The net heat exchange rate at the water surface can be approximated by the following linear relationship (Ashton 1986)

$$\phi^* = h_{wa} (T_w - T_a) \quad (41)$$

where  $\phi^*$  = net rate of heat loss from the water surface

$T_a$  = air temperature

$T_w$  = water temperature

$h_{wa}$  = heat exchange coefficient.

Since the short-wave radiation is independent of the water temperature  $T_w$  and the air temperature  $T_a$  and varies with latitude and cloud cover, a modified form of eq 41 may be used (Dingman and Assur 1969):

$$\phi^* = -\phi_R + \alpha' + \beta' (T_w - T_a) \quad (42)$$

where  $\alpha'$  and  $\beta'$  are heat exchange parameters and  $\phi_R$  is the short-wave radiation, which has to be computed separately making use of the latitude and cloud cover. Linear models cannot completely describe the heat exchange process. However, for rivers where extensive weather data are not available, linear models provide a sufficiently accurate alternative for computing the surface heat exchange.

When there is an ice cover in the river, the turbulent heat exchange taking place between water and the underside of the ice cover  $\phi_{wi}$  may be represented by

$$\phi_{wi} = h_{wi} (T_w - T_m) \quad (43)$$

where  $h_{wi}$  is the turbulent heat exchange coefficient between ice and water in  $W m^{-2} ^\circ C^{-1}$ . Considering the ice-covered river as a closed conduit, the following relationship can be used to explain the heat exchange coefficient (Ashton 1973):

$$h_{wi} = C_{wi} \frac{U^{0.8}}{d_w^{0.2}} \quad (44)$$

where  $U$  = average flow velocity ( $\text{m s}^{-1}$ )

$d_w$  = depth of flow (m)

$C_{wi}$  = constant  $\approx 1622 \text{ W s}^{0.8} \text{ m}^{-2.6} \text{ }^\circ\text{C}^{-1}$ .

Laboratory and field investigations indicate that the coefficient  $C_{wi}$  may vary with the resistance of the cover (Haynes and Ashton 1979, Calkins 1984, Marsh and Prowse 1987).

In river reaches where the heat exchange at the river bed and the heat flux from thermal effluents may be significant, these terms have to be considered in the source term. For most rivers the bed heat flux can either be neglected or lumped into the surface heat exchange in the thermal analysis.

### Numerical solution

Many numerical methods have been introduced to solve the transport equations. Most of these methods are based on Eulerian schemes. The method suggested by Leendertse (Cunge et al. 1980) is capable of reducing artificial diffusion but introducing artificial dispersion. It produces oscillatory behavior in the solution. Holly and Preissman (1977) introduced a method aimed at reducing both numerical diffusion and dispersion but at the expense of computational efficiency.

The difficulties in using Eulerian schemes to model convection without causing instabilities, inaccuracies and oscillations led to the use of Lagrangian or Eulerian–Lagrangian schemes (Fischer et al. 1979, Jobson 1987). The Lagrangian scheme uses marked parcels of water moving along the channel at the flow velocity. Lagrangian schemes are stable at any Courant number, and if there is no need for the values of the concentration in the moving parcel to be interpolated to the Eulerian grid, numerical diffusion is virtually nonexistent.

In the present model a Lagrangian–Eulerian scheme is developed to solve the transport equation. The following Lagrangian equation can be obtained from eq 38 or 40 using the continuity equation and neglecting the dispersion term:

$$\frac{DT_w}{Dt} = - \frac{\phi}{\rho C_p d_w} \quad (45)$$

In the computation, marked parcels of water with known water temperature or ice concentration are released from the upstream boundary and nodal points at each time step. The parcels are followed along the river with known flow velocities from the hydraulic computation. The temperature or concentration values of each moving parcel can be determined at any time during the movement. When the parcel velocities are numerically integrated over the time step  $\Delta t$ , the new position of a parcel originally located at the  $i^{\text{th}}$  node with distance  $x_i$  from node 1 can be obtained:

$$S_i = x_i + \sum_{k=i}^{j-1} \Delta x_k + u_j \left( \Delta t - \sum_{k=i}^{j-1} \frac{\Delta x_k}{u_k} \right) \quad (46)$$

where  $\Delta t$  = time increment

$\Delta x_k$  = length of the  $k^{\text{th}}$  reach

$S_i$  = distance to the particle from node 1 at the end of the time step

$x_i$  = distance to node  $i$  from node 1

$j - 1$  = number of the last reach completely passed by the moving parcel before the time step is complete

$u_k$  = average flow velocity in reach  $k$  at time  $t^n$

$i, j, k$  = dummy integer variables.

The movement and positions of parcels can be illustrated by an  $x$ – $t$  plot as shown in Figure 7. In the figure,  $\delta t_i = \Delta x_i / u_i$  within any reach  $i$ . A parcel  $a_1$  at node 1 will go through  $b_1, c_1$  and end up at  $d_1$  at time  $t^{n+1}$ . Similarly parcels starting at  $a_2, a_3, \dots$  move during the time step  $\Delta t$  along the paths shown.

The temperature of the  $i^{\text{th}}$  parcel at time  $t^{n+1}$  can be obtained by integrating eq 45 over time:

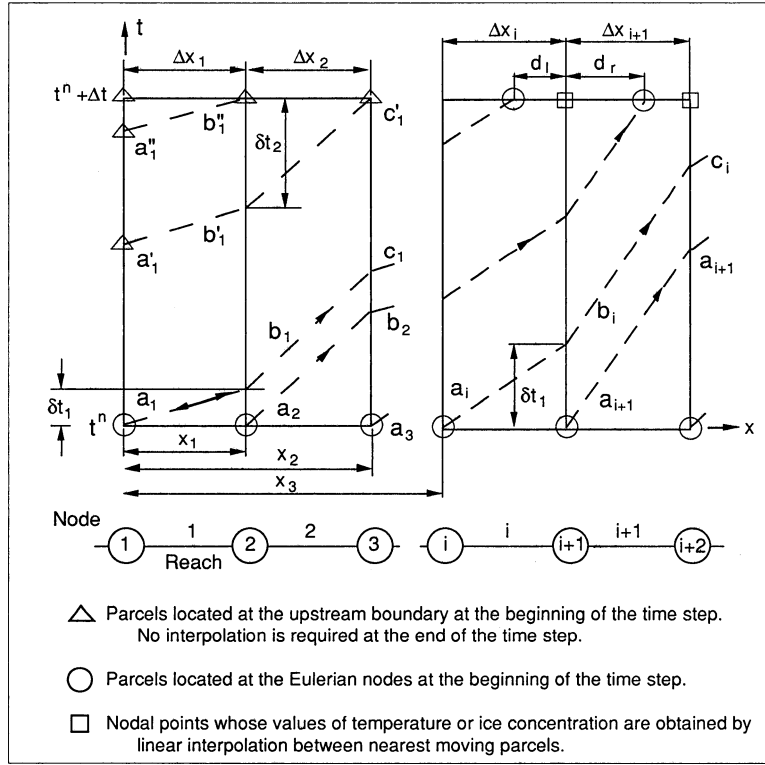


Figure 7. Lagrangian-Eulerian scheme.

$$T_{w,j}^{(n+1)} = T_{w,i}^n - \sum_{k=i}^{j-1} \frac{\phi_k}{\rho C_p d_{wk}} \frac{\Delta x_k}{u_k} - \frac{\phi_j}{\rho C_p d_{wj}} \left( \Delta t - \sum_{k=i}^{j-1} \frac{\Delta x_k}{u_k} \right) \quad (47)$$

where  $T_{w,i}^{n+1} = T_w$  at node  $i$  at time  $t^{n+1}$   
 $T_{w,i}^{(n+1)} = T_w$  of a Lagrangian parcel  $i$  at time  $t^{n+1}$   
 $d_{wk}$  = mean depth of the reach  $k$   
 $\phi_k$  = heat loss rate in reach  $k$ .

For large values of  $\Delta t$ , the above procedure cannot determine the water temperature distribution in the vicinity of the upstream boundary, e.g. at nodes 2 and 3 in Figure 7. The values at these nodes can be determined by backtracking moving parcels to the upstream boundary. For example, parcels that arrive at nodes 2 and 3 at time  $t^{n+1}$  are originated at  $a_1'$  and  $a_1''$ . The positions of  $a'$  and  $a''$  can be determined by drawing  $a_1'b_1'$  and  $a_1''b_1''$  parallel to  $a_1b_1$ , and  $b_1'c_1'$  parallel to  $a_2b_2$ . The values of the water temperature or ice concentration at the boundary node at any time can be determined by linearly interpolating between values at the time levels  $t^n$  and  $t^{n+1}$ . Using this procedure, temperatures at the nodes near the upstream boundary can be determined using the following expression:

$$T_{w,i}^{n+1} = \frac{T_{w,1}^n}{\Delta t} \left( \sum_{k=1}^{i-1} \frac{\Delta x_k}{u_k} \right) + \frac{T_{w,1}^{n+1}}{\Delta t} \left( \Delta t - \sum_{k=1}^{i-1} \frac{\Delta x_k}{u_k} \right) - \sum_{k=1}^{i-1} \frac{\phi_k}{\rho C_p d_{wk}} \frac{\Delta x_k}{u_k}. \quad (48)$$

Equation 47 does not give the values of temperature or concentration directly at the fixed Eulerian nodes. For the convenience of bookkeeping, the information carried by Lagrangian parcels are interpolated to the fixed Eulerian nodal points at the end of each  $\Delta t$  time step. These interpolated nodal values are then used as the starting

For a river reach that is fully covered over part of its length and fully open over the remaining part,  $S_{fu}$  is obtained using eq 14 and  $S_{fd}$  is obtained using eq 15. The average friction slope over the reach can be obtained using eq 10:

$$\bar{S}_f = Q^2 n_b^2 \left[ (1 - \lambda) \frac{P_u^{4/3}}{A_u^{10/3}} + \lambda (1 + F)^{4/3} \frac{P_d^{4/3}}{A_d^{10/3}} \right]. \quad (16)$$

When the channel cross section is relatively uniform in a reach and the cross-sectional area occupied by the ice cover is small in comparison to the total cross-sectional area, both  $P_u$  and  $P_d$  can be replaced by an average value  $P$  for the reach. Similarly  $A_u$  and  $A_d$  can be replaced by the average cross-sectional area  $\bar{A}$  for the reach. With these approximations, eq 16 reduces to

$$\bar{S}_f = \frac{Q^2 n_b^2 P^{4/3}}{\bar{A}^{10/3}} [(1 - \lambda) + (1 + F)^{4/3} \lambda]. \quad (17)$$

Using eq 17, an expression for  $n_e$  can be obtained to give an equivalent roughness in Manning's relationship (Yapa 1983):

$$n_e^2 = n_b^2 \left[ \frac{\Delta x_o}{\Delta x} + (1 + F)^{4/3} \frac{\Delta x_c}{\Delta x} \right]. \quad (18)$$

The same approach can be made to find the friction slope for a case where the ice extends over part of the width  $(1 - \mu)B$ , assuming the channel to be a wide rectangular such that  $p_\alpha \approx \mu B$ ,  $A_\alpha \approx \mu A$ , etc. Applying Manning's formula to the entire channel section and the ice-covered and open portions, respectively, and assuming the ice cover to be relatively thin in comparison to the flow depth, yields

$$Q = \frac{A R^{2/3} S_f^{1/2}}{n_e} \quad (19)$$

$$Q_\alpha = \frac{\mu A}{n_c} \left( \frac{\mu A}{2\mu B} \right)^{2/3} S_f^{1/2} = \frac{\mu A^{5/3}}{n_c 2^{2/3} B^{2/3}} S_f^{1/2} \quad (20)$$

$$Q_\beta = \frac{(1-\mu)A}{n_b} \left[ \frac{(1-\mu)A}{(1-\mu)B} \right]^{2/3} S_f^{1/2} = \frac{(1-\mu)A^{5/3}}{n_b B^{2/3}} S_f^{1/2}. \quad (21)$$

The continuity condition gives

$$Q = Q_\alpha + Q_\beta. \quad (22)$$

Substituting  $Q_\alpha$ ,  $Q_\beta$ ,  $p_\alpha$ ,  $p_\beta$ ,  $A_\alpha$  and  $A_\beta$  into eq 11 yields the following:

$$S_f = \frac{B^{4/3} n_b^2 Q^2}{A^{7/3}} \frac{(1 + F)^{4/3}}{[\mu + (1 - \mu)(1 + F)^{2/3}]^2}. \quad (23)$$

Equation 23 can be used to obtain  $S_{fu}$  and  $S_{fd}$  of eq 10 by applying  $S_f = S_{fu}$  when  $\mu = \mu_u$  and  $S_f = S_{fd}$  when  $\mu = \mu_d$ . A general expression for the average friction slope  $S_f$  can now be expressed as



$$\bar{S}_f = \frac{Q^2 B^{4/3} n_b^2}{A^{10/3}} \left\{ \frac{(1-\lambda)(1+F)^{4/3}}{[\mu_u + (1-\mu_u)(1+F)^{2/3}]^2} + \frac{\lambda(1+F)^{4/3}}{[\mu_d + (1-\mu_d)(1+F)^{2/3}]^2} \right\}. \quad (24)$$

The resistance term ( $p_i \tau_i + p_b \tau_b$ ) in eq 2 can now be determined by using eq 8. If the ice cover extends over the entire width in the downstream reach as a result of progression,  $\mu_d = 1$ . If there is no border ice,  $\mu_u = 0$ , and eq 24 reduces to eq 17.

#### Nodal equations

The equation governing the flow continuity at node  $k$  can be expressed as

$$\sum_{i=1}^{\text{in}} Q_{d,i}^p + q_k = \sum_{j=1}^{\text{out}} Q_{u,j}^p \quad (25)$$

where  $Q$  = discharge

$i$  = incoming reaches

$j$  = outgoing reaches

$u$  and  $d$  = subscripts denoting upstream and downstream ends of each reach

$q_k$  = external flow at node  $k$ .

The energy balance for the node  $k$  can be expressed using the following equations:

$$H_{d,1}^p = H_{u,i}^p + \Delta H_i \quad \text{for } i = 2, \text{ in} + \text{out} \quad (26)$$

where  $H$  = water level

( $\text{in} + \text{out}$ ) = total number of incoming and outgoing reaches for the node

$\Delta H_i$  = head loss between reaches 1 and  $i$  due to control structures at the node.

The variable  $\Delta H_i$  is a function of  $Q_{d,1}$ ,  $H_{d,1}$  and  $H_{u,1}$  etc. There are a total of ( $\text{in} + \text{out} - 1$ ) such equations at each internal node.

#### External boundary conditions

Three types of boundary conditions are common in practice. They are the discharge, the water level and the control structure type. In the case of a control structure, a functional relationship between the discharge and the water level is necessary. Only one time history or stage–discharge relationship is required at each boundary. Special treatment may be needed when a downstream control structure is not available in a long river. In such a case the downstream boundary can be treated by satisfying the uniform flow condition at the downstream boundary as presented by Viessman et al. (1972). An alternative method is to use an approximate rating curve developed from the channel characteristics as suggested by Gunaratnam and Perkins (1970). In the case of a boundary node connected to a reservoir having a known inflow and plan area, the continuity condition for the reservoir can be used as the boundary condition (Potok and Quinn 1979).

The general functional form of an upstream and a downstream boundary conditions can be expressed as

$$F_u (H_u^p, Q_u^p) = 0 \quad (27)$$

and

$$F_d (H_d^p, Q_d^p) = 0 \quad (28)$$

where  $F_u$  and  $F_d$  represent known functions. When the head is known at the upstream boundary,

$$F_u (H_u^p, Q_u^p) = H_{u,1}^p - H_{ub} \quad (29)$$

where  $H_{ub}$  is the known head at the upstream boundary. When the discharge is known,

$$F_u (H_u^p, Q_u^p) = Q_{u,1}^p - Q_{ub} \quad (30)$$

where  $Q_{ub}$  is the known discharge at the upstream boundary. Similar expressions can be obtained for the downstream boundary. When the rating curve  $Q = f(H)$  is used, then for the downstream boundary,

$$F_d (H_d^p, Q_d^p) = Q_d^p - f(H_d^p) = 0. \quad (31)$$

#### Initial condition

If the initial conditions of the problem are known, the simulation can start with known initial values given in the input. Otherwise, the steady state solution is commonly used as the initial condition. To obtain a steady state solution, the program can be made to run starting from assumed flow and depth conditions close to those existing in the field. A sufficient number of iterations have to be used to bring the solution to the steady state.

Fread (1985) suggested the use of a backwater computation to determine  $H$  and  $Q$  values. In this study the method suggested by Potok and Quinn (1979) is used. The initial water surface profile and discharge distributions are obtained by first setting a zero discharge and an assumed constant water level for the entire study reach. The boundary conditions are then changed to those corresponding to the first time step. The flow simulation is then run until the river profile becomes steady. The resulting profile is then used as the initial condition.

#### Method of solution

In eq 3 and 4, the unknowns are  $Q_u^p$ ,  $Q_d^p$ ,  $H_u^p$  and  $H_d^p$  for each branch. For a river system with  $NR$  branches,  $NB$  nodes (including boundary nodes) and  $NBND$  boundary conditions,  $4NR$  equations are needed to solve the  $4NR$  unknowns. A breakdown of the number of equations is given in Table 1.

**Table 1. Number of equations available in the system of equations.**

Type	Number of equations
River reach continuity condition	$NR$ (number of reaches)
River reach momentum condition	$NR$ (number of reaches)
Nodal continuity condition	$NB - NBND$ (total no. of nodes - no. of boundary nodes)
Nodal energy condition	sum of (in-out-1) at $NB - NBND$ nodes
Boundary conditions	$NBND$ (no. of boundary nodes)

The finite-difference form of the Saint Venant equations, along with the nodal and boundary conditions, form a system of nonlinear equations of the form

$$\mathbf{F}(\mathbf{x}) = 0 \quad (32)$$

where  $\mathbf{x} = (x_1, x_2, \dots, x_n)^T$  is the vector containing the unknown variables  $Q^p$  and  $H^p$ , and  $f_1, f_2, \dots, f_n$  are the coordinate functions of  $\mathbf{F}$ . Using Newton's method, the iterative procedure to find  $\mathbf{x}$  at the time step  $k$  is given as (Burden and Faires 1985)

$$J[\mathbf{x}^{(k-1)}] [\mathbf{x}^{(k)} - \mathbf{x}^{(k-1)}] = -\mathbf{F}(\mathbf{x}^{(k-1)}) \quad (33)$$

where  $J(\mathbf{x})$  is the Jacobian matrix. For computational efficiency,  $\mathbf{x}$  is solved as a system of linear equations. The  $(i,j)$  element of the Jacobian matrix can be written as

**Table 2. Arrangement of a Jacobian matrix and the right side of eq 30 for the case shown in Figure 2.**

Equation	Reach 1				Reach 2				Reach 3 (NR)				RHS
	H <sub>u</sub>	Q <sub>u</sub>	H <sub>d</sub>	Q <sub>d</sub>	H <sub>u</sub>	Q <sub>u</sub>	H <sub>d</sub>	Q <sub>d</sub>	H <sub>u</sub>	Q <sub>u</sub>	H <sub>d</sub>	Q <sub>d</sub>	
Upstream boundary	A	B	0	0	0	0	0	0	0	0	0	0	C
Continuity: reach 1	X	X	X	X	0	0	0	0	0	0	0	0	X
Momentum: reach 1	X	X	X	X	0	0	0	0	0	0	0	0	X
Continuity: node 2	0	0	0	X	X	0	0	0	0	0	0	0	X
Energy: node 2	0	0	X	0	X	0	0	0	0	0	0	0	X
Continuity: reach 2	0	0	0	0	X	X	X	X	0	0	0	0	X
Momentum: reach 2	0	0	0	0	X	X	X	X	0	0	0	0	X
Continuity: node 3	0	0	0	0	0	0	0	X	0	X	0	0	X
Energy: node 3	0	0	0	0	0	0	X	0	X	0	0	0	X
Continuity: reach 3	0	0	0	0	0	0	0	0	X	X	X	X	X
Momentum: reach 3	0	0	0	0	0	0	0	0	X	X	X	X	X
Downstream boundary	0	0	0	0	0	0	0	0	0	0	D	E	F
(Jacobian matrix, 12*12)													(RHS, 12*1)

Key: X = Variables taken care of by the program.

A,B,C,D,E,F = Variables defined by user in subroutines UPBOUND and DNBOUND to take care of boundary conditions.

$$J_{i,j}(\mathbf{x}) = \frac{\partial f_i(\mathbf{x})}{\partial x_j}. \quad (34)$$

At each computational time step, the above iterative procedure is followed to obtain the unknowns  $\mathbf{x}$ , which are the  $H$  and  $Q$  values of the river reaches. The selection of initial values  $\mathbf{x}^0$  is explained later. An alternative method for solving a system of nonlinear equations is available when the analytical expression of  $J(\mathbf{x})$  cannot be obtained easily (Broyden 1965).

By selecting a suitable numbering system for nodes and reaches, the bandwidth of the Jacobian matrix can be reduced to a minimum. This is a very useful technique to reduce computing time and obtain accurate results (Potok 1978). In the method used, nodes have to be numbered along the river from the upstream end of the river to the downstream end of the downstream boundary reach. By doing this, external boundary conditions would not come into the center of the Jacobian matrix. Also, the difference between two neighboring reaches and nodes has to remain at a minimum, which is a condition for minimum bandwidth. Reach numbers should follow node numbers. It is preferable to have the reach number be the same as the node number at the upstream end of the reach. This condition is not possible in the case of river networks. Table 2 shows an example of a numbering system for a river divided into three reaches.

### Calibration of resistance coefficients

A river system consisting of many river reaches can be considered as a discrete system, and the Manning coefficient of each section can be calibrated using influence coefficient methods (Beck and Arnold 1977, Lal and Shen 1990b). Since influence coefficients can be obtained numerically, the calibration methods do not require any manipulation of the governing equations. The numerical model for simulating the unsteady flow can be used without further modification. Observed and simulated values of any number of state variables (water levels) can be used to calibrate a number of parameters equal to the number of state variables.

The calibration uses observations of the state variable  $Y_k$ , for each of  $i = 1, 2, \dots, n$  sensors (water level gauges) at time steps  $k = 1, 2, \dots, m$ . The data are used to calibrate the parameters,  $\theta_i, i = 1, 2, \dots, n$  (Manning's roughness coefficients). Three influence coefficient methods, depending on the selection of the objective function, can be used. These methods are the least-square method, the minimax method and the minimum bias method. The objective function used when applying the principle of least squares is the minimization of the error sum of squares  $S$  over all the sensors and all the time steps. The expression for  $S$  is

$$S = \sum_{i=1}^n \sum_{k=1}^m (y_i^k - Y_i^k)^2 \quad (35)$$

where  $y_i^k$  is the simulated value of the  $i^{th}$  state variable at time  $k$ . The objective function of the minimax method is to minimize the sum of maximum errors in the simulation:

$$\min e_1 + e_2 + \dots + e_n \quad (36)$$

where  $e_1, e_2, \dots, e_n$  are the maximum errors at gauges 1, 2,  $\dots, n$ . The objective function of the minimum bias method is to minimize the absolute value of the overall bias of simulated values at all the gauging stations. The overall bias at any gauge  $i$  is given by

$$B_i = \sum_{k=1}^m (y_i^k - Y_i^k). \quad (37)$$

The objective is to minimize  $|B_i|$  for each  $i = 1, 2, \dots, n$ . This objective is achieved by making each element zero. The current study uses the minimum bias method to obtain values of Manning's coefficients, since the method is more efficient and gives accurate results.

## WATER TEMPERATURE AND ICE CONCENTRATION DISTRIBUTIONS

### Governing equations

In a well-mixed river, distributions of water temperature and depth-averaged frazil concentration along the river can be described by the one-dimensional advection-diffusion equation (Fischer et al. 1979). The equation for the cross-section-averaged water temperature  $T_w$  of a river is

$$\frac{\partial}{\partial t} (\rho C_p A T_w) + \frac{\partial}{\partial z} (Q \rho C_p T_w) = \frac{\partial}{\partial x} \left( A E_x \rho C_p \frac{\partial T_w}{\partial x} \right) - B \phi_T \quad (38)$$

where  $A$  = cross-sectional area of the river  
 $B$  = river width  
 $Q$  = discharge  
 $\rho$  = density of water  
 $C_p$  = specific heat of water  
 $E_x$  = longitudinal dispersion coefficient  
 $\phi_T$  = total heat loss rate per unit surface area of the river.

Two boundary conditions and an initial condition are needed to solve this equation.

If the river does not have large temperature gradients in the longitudinal direction, the longitudinal dispersion term

$$\frac{\partial}{\partial x} \left( A E_x \rho C_p \frac{\partial T_w}{\partial x} \right)$$

can be neglected. An analysis of the water temperature data in the upper St. Lawrence River shows that typical values of  $\partial T_w / \partial t$ ,  $\partial T_w / \partial x$  and  $\partial^2 T_w / \partial x^2$  are on the orders of  $10^{-6}$ ,  $10^{-5}$  and  $10^{-8}$ , respectively. Without the dispersion term, the solution procedure simplifies. When flow is unidirectional, only one boundary condition is needed at the upstream end:

$$T_w(0, t) = T_1(t) \quad (39)$$

where  $T_1(t)$  is the time history of the water temperature at the upstream boundary. If the initial condition  $T_w(x, 0)$  is not available, a steady state solution can be used as in the case of the hydraulics computation.

When the water temperature drops below the freezing point, frazil ice will be produced. The one-dimensional equation governing the transport of frazil ice is (Shen and Chiang 1984)

$$\frac{\partial}{\partial t} (\rho_i L_i A C_i) + \frac{\partial}{\partial x} (Q \rho_i L_i C_i) = \frac{\partial}{\partial x} \left( A E_x \rho_i L_i \frac{\partial C_i}{\partial x} \right) - B \phi_T + \Sigma S \quad (40)$$

where  $C_i$  = ice concentration

$L_i$  = latent heat of fusion of water

$\rho_i$  = density of ice

$B$  = width of the river

$\Sigma S$  = additional source or sink terms due to ice cover progression, erosion, deposition and melting.

The total heat flux term  $\phi_T$  may be approximated by the surface heat flux  $\phi^*$ . Since eq 40 is in the same form as eq 38, both the ice concentration and the water temperature can be obtained from the solution of a single equation by considering that  $C_i \rho_i L_i = -\rho C_p T_w$  and letting the water temperature in the heat exchange term  $\phi_T$  in eq 40 be equal to the freezing point of water (Shen and Chiang 1984).

### Heat exchanges

During open water conditions the total heat flux at the free surface is equal to the sum of the net short- and long-wave radiations, the sensible heat exchange and the heat transfers due to evaporation and precipitation. Some of these components, especially the long-wave radiation, are nonlinear functions of air and water temperatures. In this model, linear relationships using constant heat exchange coefficients are used to express the net heat exchange. These relationships can be obtained using multiple linear regression of computed heat exchange and weather parameters (Lal and Shen 1990a).

The net heat exchange rate at the water surface can be approximated by the following linear relationship (Ashton 1986)

$$\phi^* = h_{wa} (T_w - T_a) \quad (41)$$

where  $\phi^*$  = net rate of heat loss from the water surface

$T_a$  = air temperature

$T_w$  = water temperature

$h_{wa}$  = heat exchange coefficient.

Since the short-wave radiation is independent of the water temperature  $T_w$  and the air temperature  $T_a$  and varies with latitude and cloud cover, a modified form of eq 41 may be used (Dingman and Assur 1969):

$$\phi^* = -\phi_R + \alpha' + \beta' (T_w - T_a) \quad (42)$$

where  $\alpha'$  and  $\beta'$  are heat exchange parameters and  $\phi_R$  is the short-wave radiation, which has to be computed separately making use of the latitude and cloud cover. Linear models cannot completely describe the heat exchange process. However, for rivers where extensive weather data are not available, linear models provide a sufficiently accurate alternative for computing the surface heat exchange.

When there is an ice cover in the river, the turbulent heat exchange taking place between water and the underside of the ice cover  $\phi_{wi}$  may be represented by

$$\phi_{wi} = h_{wi} (T_w - T_m) \quad (43)$$

where  $h_{wi}$  is the turbulent heat exchange coefficient between ice and water in  $W m^{-2} ^\circ C^{-1}$ . Considering the ice-covered river as a closed conduit, the following relationship can be used to explain the heat exchange coefficient (Ashton 1973):

$$h_{wi} = C_{wi} \frac{U^{0.8}}{d_w^{0.2}} \quad (44)$$



where  $U$  = average flow velocity ( $\text{m s}^{-1}$ )  
 $d_w$  = depth of flow (m)  
 $C_{wi}$  = constant  $\approx 1622 \text{ W s}^{0.8} \text{ m}^{-2.6} \text{ }^\circ\text{C}^{-1}$ .

Laboratory and field investigations indicate that the coefficient  $C_{wi}$  may vary with the resistance of the cover (Haynes and Ashton 1979, Calkins 1984, Marsh and Prowse 1987).

In river reaches where the heat exchange at the river bed and the heat flux from thermal effluents may be significant, these terms have to be considered in the source term. For most rivers the bed heat flux can either be neglected or lumped into the surface heat exchange in the thermal analysis.

### Numerical solution

Many numerical methods have been introduced to solve the transport equations. Most of these methods are based on Eulerian schemes. The method suggested by Leendertse (Cunge et al. 1980) is capable of reducing artificial diffusion but introducing artificial dispersion. It produces oscillatory behavior in the solution. Holly and Preissman (1977) introduced a method aimed at reducing both numerical diffusion and dispersion but at the expense of computational efficiency.

The difficulties in using Eulerian schemes to model convection without causing instabilities, inaccuracies and oscillations led to the use of Lagrangian or Eulerian–Lagrangian schemes (Fischer et al. 1979, Jobson 1987). The Lagrangian scheme uses marked parcels of water moving along the channel at the flow velocity. Lagrangian schemes are stable at any Courant number, and if there is no need for the values of the concentration in the moving parcel to be interpolated to the Eulerian grid, numerical diffusion is virtually nonexistent.

In the present model a Lagrangian–Eulerian scheme is developed to solve the transport equation. The following Lagrangian equation can be obtained from eq 38 or 40 using the continuity equation and neglecting the dispersion term:

$$\frac{DT_w}{Dt} = - \frac{\phi}{\rho C_p d_w}. \quad (45)$$

In the computation, marked parcels of water with known water temperature or ice concentration are released from the upstream boundary and nodal points at each time step. The parcels are followed along the river with known flow velocities from the hydraulic computation. The temperature or concentration values of each moving parcel can be determined at any time during the movement. When the parcel velocities are numerically integrated over the time step  $\Delta t$ , the new position of a parcel originally located at the  $i^{\text{th}}$  node with distance  $x_i$  from node 1 can be obtained:

$$S_i = x_i + \sum_{k=i}^{j-1} \Delta x_k + u_j \left( \Delta t - \sum_{k=i}^{j-1} \frac{\Delta x_k}{u_k} \right) \quad (46)$$

where  $\Delta t$  = time increment

$\Delta x_k$  = length of the  $k^{\text{th}}$  reach

$S_i$  = distance to the particle from node 1 at the end of the time step

$x_i$  = distance to node  $i$  from node 1

$j - 1$  = number of the last reach completely passed by the moving parcel before the time step is complete

$u_k$  = average flow velocity in reach  $k$  at time  $t^n$

$i, j, k$  = dummy integer variables.

The movement and positions of parcels can be illustrated by an  $x-t$  plot as shown in Figure 7. In the figure,  $\delta t_i = \Delta x_i / u_i$  within any reach  $i$ . A parcel  $a_1$  at node 1 will go through  $b_1, c_1$  and end up at  $d_1$  at time  $t^{n+1}$ . Similarly parcels starting at  $a_2, a_3, \dots$  move during the time step  $\Delta t$  along the paths shown.

The temperature of the  $i^{\text{th}}$  parcel at time  $t^{n+1}$  can be obtained by integrating eq 45 over time:

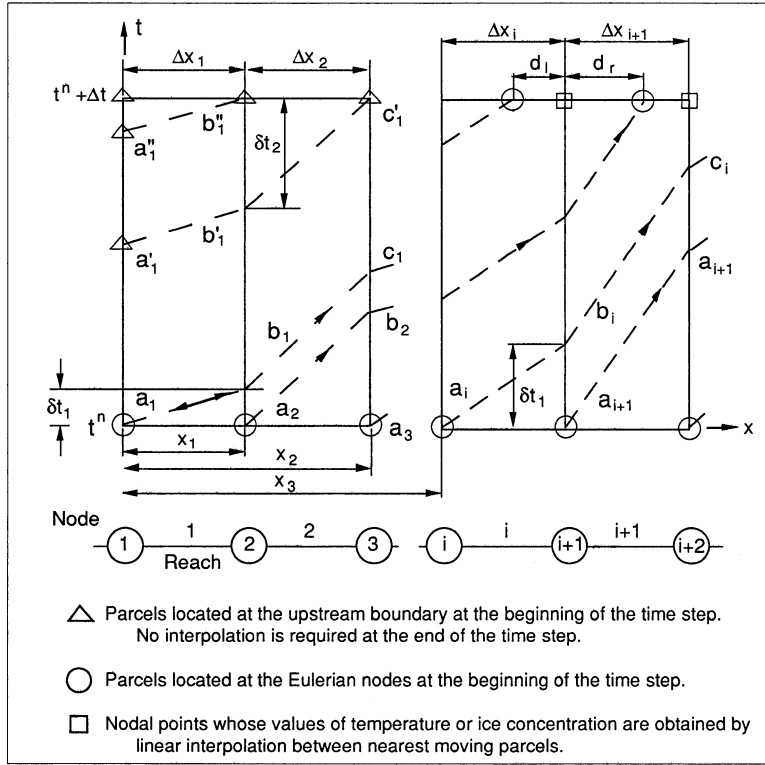


Figure 7. Lagrangian-Eulerian scheme.

$$T_{w,j}^{(n+1)} = T_{w,i}^n - \sum_{k=i}^{j-1} \frac{\phi_k}{\rho C_p d_{wk}} \frac{\Delta x_k}{u_k} - \frac{\phi_j}{\rho C_p d_{wj}} \left( \Delta t - \sum_{k=i}^{j-1} \frac{\Delta x_k}{u_k} \right) \quad (47)$$

where  $T_{w,i}^{n+1} = T_w$  at node  $i$  at time  $t^{n+1}$   
 $T_{w,i}^{(n+1)} = T_w$  of a Lagrangian parcel  $i$  at time  $t^{n+1}$   
 $d_{wk}$  = mean depth of the reach  $k$   
 $\phi_k$  = heat loss rate in reach  $k$ .

For large values of  $\Delta t$ , the above procedure cannot determine the water temperature distribution in the vicinity of the upstream boundary, e.g. at nodes 2 and 3 in Figure 7. The values at these nodes can be determined by backtracking moving parcels to the upstream boundary. For example, parcels that arrive at nodes 2 and 3 at time  $t^{n+1}$  are originated at  $a'_1$  and  $a'_2$ . The positions of  $a'$  and  $a''$  can be determined by drawing  $a'_1 b'_1$  and  $a'_2 b'_2$  parallel to  $a_1 b_1$ , and  $b'_1 c'_1$  parallel to  $a_2 b_2$ . The values of the water temperature or ice concentration at the boundary node at any time can be determined by linearly interpolating between values at the time levels  $t^n$  and  $t^{n+1}$ . Using this procedure, temperatures at the nodes near the upstream boundary can be determined using the following expression:

$$T_{w,i}^{n+1} = \frac{T_{w,i}^n}{\Delta t} \left( \sum_{k=1}^{i-1} \frac{\Delta x_k}{u_k} \right) + \frac{T_{w,i}^{n+1}}{\Delta t} \left( \Delta t - \sum_{k=1}^{i-1} \frac{\Delta x_k}{u_k} \right) - \sum_{k=1}^{i-1} \frac{\phi_k}{\rho C_p d_{wk}} \frac{\Delta x_k}{u_k}. \quad (48)$$

Equation 47 does not give the values of temperature or concentration directly at the fixed Eulerian nodes. For the convenience of bookkeeping, the information carried by Lagrangian parcels are interpolated to the fixed Eulerian nodal points at the end of each  $\Delta t$  time step. These interpolated nodal values are then used as the starting

form. The value of  $T_{cr}$  is site dependent. Matousek obtained a value of  $-1.1^\circ\text{C}$  for River Ohre. In the current model,  $T_{cr} = -0.5^\circ\text{C}$  was found to describe the skim ice formation in the upper St. Lawrence River. This discrepancy may be partly caused by the use of one-dimensional velocity in eq 79.

The above formulation reflects the effect of turbulent intensity on the river ice formation process. The stability of frazil ice crystals formed in the supercooled surface layer is governed by the relative magnitudes of the vertical component of the turbulent fluctuation  $v_z'$  and the buoyant velocity of ice crystals  $v_b$  (Matousek 1984b). When  $v_z' > v_b$ , ice crystals formed in the surface layer are entrained into the depth of the flow. If  $T_w > 0^\circ\text{C}$ , ice crystals melt into the flowing water. If  $T_w < 0^\circ\text{C}$ , ice crystals grow and will be transported as frazil suspension. The equations developed by Ramseier (1970) and Zacharov et al. (1972) are used to obtain an expression for  $v_b$  (Matousek 1984b):

$$v_b = -0.025T_{w,s} + 0.005. \quad (80)$$

Matousek (1984b) used the equation developed by Makaveyev to obtain  $v_z'$  (Karaushev 1969):

$$v_z' = \frac{\sqrt{g}}{5\sqrt{(0.7C + 6)C}} u \quad (81)$$

where  $u$  = depth-averaged velocity of the water flow ( $\text{m s}^{-1}$ )  
 $v_z'$  = vertical fluctuating component of water velocity ( $\text{m s}^{-1}$ )  
 $C$  = Chezy's coefficient ( $\text{m}^{0.5} \text{s}^{-1}$ )  
 $g$  = gravitational acceleration ( $\text{m s}^{-2}$ ).

Equation 81 is valid for  $10 < C < 60$ . Figure 12 illustrates freeze-up and ice run types obtained for a given set of  $T_w$ ,  $b$  and  $W$  values by Matousek. For lack of a better analytical formulation, Matousek's empirical formulation is used in the present model to determine skim and static shore ice formation. Equation 81 does not include the effect of wind on the turbulent fluctuation velocity. It gives a very low value for the turbulence fluctuation velocity on the water surface. Following is a simple derivation for turbulent velocity, similar to the one presented in Fischer et al. (1979) for vertical mixing under the influence of wind. In this method the resultant turbulent velocity fluctuation due to both wind and bed shears is obtained by assuming that the input powers of both effects can be added linearly.

To find the turbulent fluctuation velocity due to wind, the rate of work done by wind on the water surface

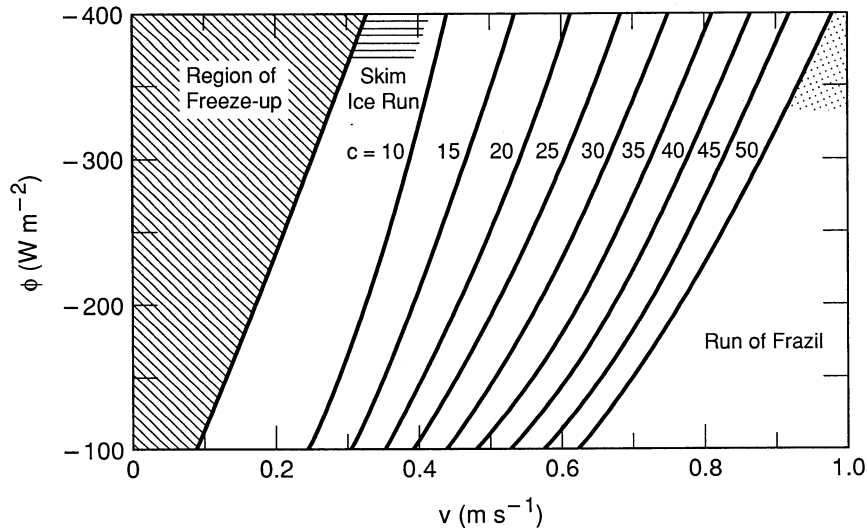


Figure 12. Relationship between ice run types, freeze-up mode and  $\phi$ ,  $u$  and  $C$  at  $T_w = 0$ ,  $b = 27$  and  $W = 0.5 \text{ m s}^{-1}$  (Matousek 1984).

To find the turbulent fluctuation velocity due to wind, the rate of work done by wind on the water surface  $P$  is

$$P = \tau_w u_w \quad (82)$$

where  $u_w$  is the velocity of water near the surface and  $\tau_w$  is the shear stress of wind. Then,

$$\tau_w = C_D \rho_a W^2 \quad (83)$$

where  $W$  = wind velocity ( $\text{m s}^{-1}$ ) at 10 m above the water surface

$\rho_a$  = density of air ( $1.22 \times 10^3 \text{ kg m}^{-3}$ )

$C_D$  = drag coefficient.

$C_D = 1.3 \times 10^{-3}$  is used with wind velocities measured 10 m above the water surface. The shear velocity due to wind  $w_*$  can be obtained by using

$$\tau_w = \rho w_*^2 \quad (84)$$

where  $\rho$  is the density of water. After combining with eq 83, it gives

$$w_* = \left| C_D \frac{\rho_a}{\rho} W^2 \right|^{1/2}. \quad (85)$$

Assuming  $u_w$  is of the same order of magnitude as  $w_*$ , and using  $\tau_w$  of eq 84 in eq 82,

$$P \approx \rho w_*^3 \quad (86)$$

The second source of turbulence at the river surface is due to the shear at the river bed. The turbulent fluctuation velocity  $v'$  at the channel surface due to bed shear has been related to the shear velocity  $u_*$  by Rodi (1980) using plots that give the ratio  $C_*$ . This relationship can be expressed as

$$\frac{v'^2}{u_*^2} = C_*. \quad (87)$$

Rodi's experimental results show that the ratio  $C_*$  lies between 0.2 and 0.3. An expression for  $u_*$  can be obtained using the following equations:

$$u = \frac{1}{n_b} R^{2/3} S_f^{1/2} \quad (88)$$

$$\tau_o = \rho g R S_f \quad (89)$$

$$\tau_o = \rho u_*^2 \quad (90)$$

where  $u$  = average flow velocity

$n_b$  = Manning's roughness coefficient

$R$  = hydraulic radius

$S_f$  = energy slope.

Equations 88–90 give

$$u_* = g^{1/2} \frac{u n_b}{R^{1/6}}. \quad (91)$$

Assuming that the input powers of bed shear and wind shear can be added, a measure of the turbulent fluctuating velocity  $q$  due to both effects can be obtained using

$$q^3 = (v'^3 + C_{w*} w_*^3) \quad (92)$$

where  $C_{w*}$  is a constant explaining the efficiency of wind energy utilization.  $C_{w*} = 1$  was selected in the current study after considering different experimental observations (Fischer et al. 1979). Equations 87 and 91 are then used to obtain  $v'$ , when eq 85 is used to obtain  $w_*$ . Equation 92 can now be expressed as

$$q^3 = \left( C_*^{1/2} g^{1/2} \frac{un}{R^{1/6}} \right)^3 + C_{w*} \left( C_D \frac{\rho_a}{\rho} \right)^{3/2} W^3 \quad (93)$$

where  $q$  is an indicator of the magnitude of the turbulent fluctuation velocity. In the current study,  $q$  obtained using eq 93 is used to replace the turbulent fluctuation velocity  $v_z$  of eq 81. When the condition in a given reach favors skim ice formation, the rate of growth across the width is assumed to be infinite. The maximum width up to which skim ice can form is given as input data because it depends on the distribution of flow across the width. A variable  $\mu$  is used to represent the fraction of width that can be covered by skim ice. In this section, plausible empirical formulations are used for simulating static ice formations. Further studies are needed to improve the formulation.

### Dynamic border ice formation

In addition to the static thermal mode, dynamic border ice can form due to the accumulation of surface ice along the shore or edges of existing border ice. This type of lateral growth is limited by the stability of surface ice particles in contact with the existing edge of the border ice. The rate of growth of the width of the border ice will also be governed by the surface concentration of the frazil ice run. The stability of the ice particles that are in contact with the edge of existing border ice is governed by the drag force acting on the particle, the component of the gravity force along the water surface, and forces from neighboring ice particles. These forces are resisted by the friction or strength at the contact point. A theoretical model that is capable of describing the above phenomena is yet to be developed. An empirical model has been proposed by Michel et al. (1980), and this study uses that model.

Based on the border ice development in the St. Anne River, Canada, Michel et al. (1980) obtained the following empirical relationship for the rate of lateral growth of border ice:

$$R = 14.1 V_*^{-0.93} N^{1.08} \quad (94)$$

where  $R = \frac{\rho L_i \Delta W}{\Delta \phi}$

$$V_* = \frac{u}{V_c}$$

$u$  = depth-averaged flow velocity in the open water adjacent to edge of the existing border ice (m/s<sup>-1</sup>)

$V_c$  = maximum velocity at which a surface ice particle can adhere to the border ice

$\rho$  = density of water

$L_i$  = latent heat of fusion of ice

$\Delta W$  = growth of border ice per unit time

$\Delta \phi$  = heat exchange per unit area per unit time

$$N = \frac{Q_i^s}{ut_i B} = \text{aerial concentration of the surface ice}$$

$Q_i^s$  = surface ice transport rate

$t_i$  = ice floe thickness

$B$  = width of the reach.

According to Michel et al. (1980), only the static mode exists when  $N < 0.1$ , and eq 48 should be used with  $N = 0.1$ .

Michel et al. (1980) indicated that eq 94 is valid for  $0.167 < V_* < 1.0$ . When  $V_* < 0.167$ , static ice or skim ice formation occurs; when  $V_* < 1.0$ , only thermal growth occurs because frazil cannot adhere to the border ice. The rate of thermal growth is negligible. In the present study, since the static ice formation is modeled using the formulation given earlier, the lower limit is not used. For the St. Anne River, Michel et al. (1980) found that the critical velocity  $V_c$  is  $1.2 \text{ m s}^{-1}$ . According to the preceding discussion, however, this value is governed by gravity, drag and friction forces acting on the surface particle, and it will vary accordingly. For the upper St. Lawrence River, the  $V_c$  value is about  $0.4 \text{ m s}^{-1}$  for typical flow conditions (Shen and Van DeValk 1984).

### Dynamic ice cover formation

When the ice run occurs, an ice cover may be formed due to the accumulation of surface ice on the river surface. If conditions are favorable, this type of ice cover can extend upstream with the accumulation of surface ice. In the present study, existing quasi-static ice jam theories (Pariset and Hausser 1961, Uzunur and Kennedy 1976) are used by taking into consideration the interaction between flow and ice conditions to determine the rate of ice cover progression. The present model is capable of simulating ice cover formation by particle juxtaposition, hydraulic thickening (commonly known as a narrow river jam) and mechanical thickening (commonly known as a wide river jam).

#### Particle juxtaposition

In regions with a relatively low flow velocity, an ice cover can form by the juxtaposition of ice floes. For an ice cover to progress in this mode, a stability condition for incoming ice floes must be satisfied. The stability of an ice floe when the leading edge thickness of the ice cover equals that of the ice floe can be determined by an equilibrium analysis of an arriving ice floe.

Pariset and Hausser (1961) and Pariset et al. (1966) suggested that the critical stability criterion can be expressed in terms of the Froude number of the flow:

$$F_{rc} = \frac{V_c}{\sqrt{gH}} = F \left( \frac{t_i}{l_i} \right) \left( 1 - \frac{t_i}{H} \right) \sqrt{2g \left( 1 - \frac{\rho_i}{\rho} \right) (1 - e) \frac{t_i}{H}} \quad (95)$$

where  $F_{rc}$  = Froude number of the river

$V_c$  = critical velocity upstream of leading edge for overturning and submergence

$t_i$  = thickness of the ice floe

$H$  = upstream flow depth

$h = H - t_i$

$g$  = acceleration of gravity

$F(t_i/l_i)$  = form factor that varies between 0.66 and 1.30

$l_i$  = length of the ice floe.

Ashton (1974) pointed out that it is more appropriate to express the stability criterion in terms of the particle Froude number. For ice floes with  $0.1 < t_i/l_i < 0.5$ , the following analytical expression derived by Ashton (1974) compared well with laboratory data:

$$F_{rc}^p \equiv \frac{V_c}{\sqrt{[gt_i(1 - \frac{\rho_i}{\rho})]}} = \frac{2 \left( 1 - \frac{t_i}{H} \right)}{\sqrt{[5 - 3 \left( 1 - \frac{t_i}{H} \right)^2]}} \quad (96)$$

where  $F_{rc}^p$  is the critical value of the particle Froude number. The effect of porosity in the ice particle is not considered in this equation. Equations 95 and 96 are identical if

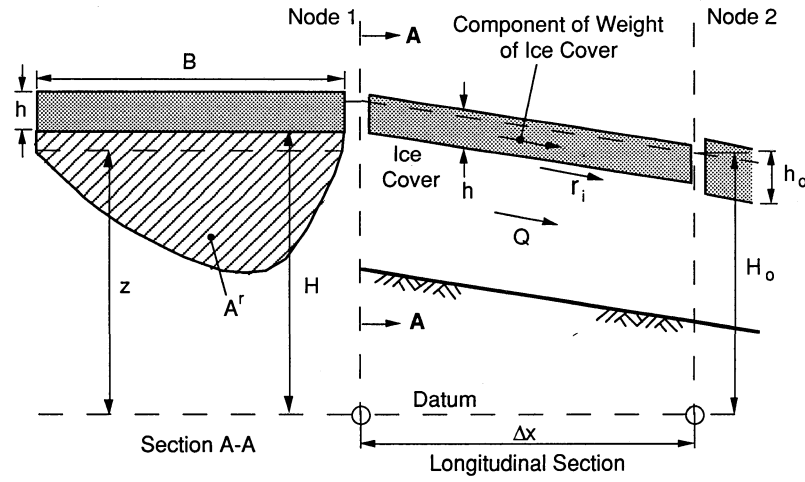


Figure 13. Definition sketch of an ice-covered river section.

$$F\left(\frac{t_i}{l_i}\right) = \frac{\sqrt{2}}{\left[5 - 3\left(\frac{t_i}{H}\right)\right]^{1/2}}.$$

The term  $(1 - e)$  is introduced into eq 95 when the effect of porosity is considered.

Under the criteria given by eq 95 or 96, an ice cover of one floe thickness will form by juxtaposition. At higher velocities, ice floes will accumulate into a cover of more than one floe thick. Equations 95 and 96 require ice floe dimensions to determine the critical velocity. Since no reliable analytical method is available to determine these dimensions, field observations or calibrated  $V_c$  or  $F_{rc}$  values are used in the present model. Most laboratory data lie between  $F_{rc} = 0.08$  and  $0.13$ , while field data lie between  $F_{rc} = 0.06$  and  $0.08$  (Kivisild 1959, Ashton 1986). For an ice floe 6.0 thick and 10.0 ft in diameter and when  $F(t_i/l_i) = 1.2$  with  $e = 0.5$ ,  $V_c$  is about 1.4 ft  $s^{-1}$ . For a flow depth of 30.0 ft, this is equivalent to  $F_{rc} = 0.055$ .

During the progression of an ice cover in a river reach, a simultaneous change in hydraulic conditions takes place. To improve the accuracy of the numerical computation, this change must be considered so that a reasonably large time step can be used in the simulation. The present model assumes that the local hydraulic conditions can be approximated by a backwater profile over the discretized river reach or reaches where progression takes place during the current time step.

Consider the case of ice cover progression by juxtaposition in the river reach shown in Figure 13. The ice cover is assumed to have reached node 2 at time  $t^n$ , so that conditions at node 2 are known. Energy balance between sections 1 and 2 gives

$$H + \frac{Q^2}{2gA^2} = H_o + \frac{Q^2}{2gA_o^2} + \bar{S}_f \Delta x \quad (97)$$

where  $\bar{S}_f$  is the average friction slope over the reach. The subscript  $o$  denotes a variable at the downstream section whose values are known. In a typical cross section of the river, the water level  $H$  and the net flow cross area  $A$  are related by

$$A = A^r + B\left(H - z - \frac{\rho_i}{\rho} h_o\right) \quad (98)$$

where  $h_o$  is the thickness of the ice cover, which is equal to the thickness of the ice floes  $t_i$ . The friction slope  $\bar{S}_f$  is obtained using eq 17. Substituting eq 98 into eq 97, the following equation can be obtained:

$$F_1(A) = \frac{A - A^r}{B} + z + t_1 \frac{\rho_i}{\rho} + \frac{Q^2}{2gA^2} - H_o - \frac{Q^2}{2gA_o^2} - 2^{1/3} Q^2 n_c^2 \Delta x \left\{ \frac{B^{4/3}}{A^{10/3}} \left[ 1 + \left( \frac{A}{B^2} \right)^{4/3} \right] + \frac{B_o^{4/3}}{A_o^{10/3}} \left[ 1 + \left( \frac{A_o}{B_o^2} \right)^{4/3} \right] \right\} = 0. \quad (99)$$

Since the thickness  $t$  is known, the flow area  $A$  can be solved by the Newton–Raphson method (Burden and Faires 1985). In eq 99 it is assumed that the entire length of  $\Delta x$  of the reach will be covered.

When the solution  $A$  is known,  $H$  is determined using eq 98, and the water surface profile is updated locally to take the effect of ice cover into account. If progression continues to the next reach upstream, eq 99 has to be applied to that reach using the  $A$  and  $t$  values just determined. The process continues until either the surface ice supply for the time step is used up or the Froude number exceeds  $F_{rc}$ . It should be noted that  $A$  and  $H$  values computed according to the above procedure are only the approximate values to be used to determine the ice conditions during the current time step. These values will be replaced by the solutions of the St. Venant equations at the time level  $t^{n+1}$  obtained from the hydraulic routine.

#### Hydraulic thickening

When the Froude number is greater than  $F_{rc}$ , the juxtaposition mode cannot exist. The ice cover will then form in the hydraulic thickening mode. The accumulation process responsible for this mode of ice cover formation is commonly known as narrow river jam formation. The equation for the narrow jam thickness was derived by Pariset and Hausser (1961) using a simple non-submergence or “no spill” condition. With a modification proposed by Michel (1971), the equation for narrow jam thickness is

$$\frac{V}{\sqrt{gH}} = \left[ 2 \frac{h_o}{H} (1 - e_c) \left( 1 - \frac{\rho_i}{\rho} \right) \right]^{1/2} \left( 1 - \frac{h_o}{H} \right) \quad (100)$$

where

$$e_c = e_p + (1 - e_p)e \quad (101)$$

and  $V$  = velocity upstream of the leading edge

$h_o$  = equilibrium initial thickness of the ice cover

$H$  = upstream flow depth

$e_p$  = porosity in the accumulation representing the space between ice floes  $\approx 0.4$

$e$  = porosity of an ice floe  $\approx 0.2$

$e_c$  = overall porosity  $\approx 0.5$ .

Equation 100 gives a maximum critical Froude number for progression  $F_{rc}^*$  at  $(h_o/H) = (1/3)$  (Pariset and Hausser 1961):

$$F_{rc}^* = \frac{V_c}{\sqrt{gH}} = 0.158 \sqrt{(1 - e_c)}. \quad (102)$$

Field observations (Kivisild 1959) indicated that  $F_{rc}^*$  varies between 0.05 and 0.1. Recent studies indicated that this value is approximately equal to 0.9 in the St. Lawrence River (Shen et al. 1984) and the Yellow River (Sun and Shen 1988). Equation 100 is used to compute the initial ice cover thickness. In the hydraulic thickening case, the interaction between the ice cover formation and the hydraulics is considered using a procedure similar to that used earlier. This procedure is as follows.

For the convenience of formulation, eq 100 may be written as



$$\frac{V_u^2}{2g} = (1 - e_c) \left(1 - \frac{\rho_i}{\rho}\right) h_o \quad (103)$$

or

$$F_2(A, h_o) = Q^2 - 2gA^2 \left(1 - \frac{\rho_i}{\rho}\right) h_o = 0 \quad (104)$$

where  $A$  = net cross-sectional area of flow under the cover

$h_o$  = thickness of the initial ice cover

$V_u$  = velocity under the ice cover

$Q = V_u A$  = river discharge.

In addition to eq 104, the energy equation (eq 99) for the flow is needed to solve for the unknowns  $A$  and  $h_o$ .

Eliminating  $h_o$  from eq 104 and 99, with  $t_i$  replaced by  $h_o$ , a nonlinear equation in the form of  $F_3 = 0$  can be obtained:

$$F_3(A) = \frac{A-A^r}{B} + z + \frac{Q^2}{2gA^2 \left(1 - \frac{\rho_i}{\rho}\right)} \frac{\rho_i}{\rho} + \frac{Q^2}{2gA^2} - H_o - \frac{Q^2}{2gA_o^2} - 2^{1/3} Q^2 n_c^2 \Delta x \left[ \frac{B^{4/3}}{A^{10/3}} \left(1 + \frac{A}{B^2}\right) + \frac{B_o^{4/3}}{A_o^{10/3}} \left(1 + \frac{A_o}{B_o^2}\right) \right] = 0. \quad (105)$$

This equation can be solved using the Newton–Raphson algorithm.

Once  $A$  is obtained from eq 105, the value of  $h_o$  can then be obtained from eq 104. Progression will continue to the next river reach upstream from cross section 2 if more surface ice supply is available. This solution procedure should continue until the ice supply is exhausted.

A study of the behavior of eq 99 and 104 indicates that there can be two roots of  $A$  and  $h_o$  in a given river reach. This is similar to the solution of Pariset and Hausser (1961), which neglected the interaction between the ice cover formation and the hydraulic condition. In addition, there exists a Froude number beyond which no root for  $A$  or  $h_o$  exists. This Froude number corresponds to the critical Froude number for progression given by eq 102.

To obtain the correct root of  $A$ , an appropriate initial value of  $A$  has to be chosen in the Newton–Raphson procedure. As pointed out by Pariset and Hausser (1961), the smaller positive root of  $h_o$ , which occurs at a Froude number less than  $F_{rc}^*$ , is the physically correct solution. Since smaller  $h_o$  values correspond to larger  $A$  values, a larger initial trial value of  $A$  should be used in the Newton–Raphson procedure.

#### *Mechanical thickening*

In a wide or steep river the increase in streamwise forces acting on the ice cover may exceed the increase in bank resistance. In this case the internal resistance of the ice accumulation may not be able to resist the increasing stress as the cover extends upstream. If the stress in the ice accumulation exceeds its internal strength, the cover will collapse and thicken until an equilibrium thickness is reached (Pariset and Hausser 1961). This process of mechanical thickening is commonly known as shoving, and an accumulation of this kind is often called a wide river jam. When shoving occurs, a relatively long reach of ice cover will collapse, and the leading edge will move a long distance downstream before new progression occurs.

The formula for equilibrium wide jam thickness is given by Pariset and Hausser (1961) as

$$\frac{BV_u^2}{\mu C^2 H^2} \left(1 + \frac{\rho_i h_o}{\rho R_H}\right) = \frac{2\tau_c h_o}{g\rho\mu H^2} + \frac{\rho_i}{\rho} \left(1 - \frac{\rho_i}{\rho}\right) \frac{h_o}{H^2} \quad (106)$$

where  $B$  = width of the river

$V_u$  = velocity under the cover

$\mu$  = ice-over-ice friction coefficient, approximately equal to 1.28

$H$  = depth of water

$C$  = Chezy coefficient

$R_H$  = hydraulic radius of the water passage under the cover

$h_o$  = thickness of the ice cover

$\rho, \rho_i$  = densities of water and ice

$\tau_c$  = cohesion term in the bank shear.

According to Pariset and Hausser,  $\tau_c h_o$  has an approximate range of 75–91 lbf ft<sup>-1</sup> during freeze-up. The current model uses  $\tau_c = 0.98$  kPa (100 kgf m<sup>-2</sup> or 20.48 lbf ft<sup>-2</sup>). During the break-up period  $\tau_c$  is usually negligible.

Equation 106 has two roots for  $h_o$ . The smaller root is considered to be the physically correct solution for thickness. There exists a maximum discharge in the river beyond which a solution does not exist and a stable ice cover cannot exist (Pariset and Hausser 1961). For  $\tau_c = 0$ , when the interaction with flow is neglected, this condition is given by

$$\frac{Q^2}{BC^2 H^4} \leq 2.8 \times 10^{-3}. \quad (107)$$

Based on the analysis of Uzuner and Kennedy (1976) and Pariset and Hausser (1961), the following modified form of eq 107 can be obtained for steady-state flow (Shen and Yapa 1984):

$$\left[ f_1 + \frac{\rho_i}{\rho} (f_b + f_i) \frac{h_o}{d_w} \right] \frac{V_u^2 B}{8g} = \frac{2\tau_c h_o}{\rho g} + \mu(1 - e_c) \left( 1 - \frac{\rho_i}{\rho} \right) \frac{\rho_i h_o^2}{\rho} \quad (108)$$

where  $f_1$  and  $f_2$  = Darcy–Weisbach friction factors related to the ice covers and the channel bed, respectively

$d_w$  = depth of the flow under the ice cover

$V_u$  = flow velocity under the cover

$e$  = porosity of the ice accumulation.

Equation 106 and eq 108 were obtained by treating the ice cover as an accumulation of granular material.

Both eq 106 and 108 were derived by assuming that the flow condition when the ice cover reaches the equilibrium thickness is known. In practice, however, only the flow condition before shoving is known. A solution procedure taking into consideration the interaction between the flow condition and the cover formation is needed. The following discussion presents a method that considers the interaction of wide jam formation with the hydraulics within a reach of the river. The method involves solving the coupled backwater and jam equations simultaneously for each reach where ice cover progression takes place, starting from the position of the existing leading edge. This method is based on the Newton–Raphson procedure similar to that presented for hydraulic thickening.

The following force balance equation can be written for the case of an equilibrium ice jam within a discretized river reach:

$$2(\tau_c h_o + \mu f) \Delta x = (\tau_i + \tau_g + \tau_a) B \Delta x \quad (109)$$

where  $f$  = longitudinal force of ice

$\tau_i$  = shearing stress of flowing water on the underside of the cover

$\tau_g$  = component of the weight along the cover

$\tau_a = C_a \rho_a |V_a| V_a \cos \theta_a$  = wind stress along the cover

$\rho_a$  = density of air

$V_a$  = wind speed

$\theta_a$  = angle between wind direction and downstream direction of the river

$C_a$  = resistance coefficient depending on surface roughness

$\mu_1$  = bank friction coefficient

$\tau_c$  = cohesive component of the bank shear.

Assuming complete mobilization of the granular mass, the balance between the passive resistance of the ice cover and the net longitudinal force  $f$  gives

$$f = \rho_i K_2 \left(1 - \frac{\rho_i}{\rho}\right) \frac{g h_o^2}{2} \quad (110)$$

where  $K_2 = \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) (1 - e_c)$

$e_c$  = porosity of the cover

$\tan \phi$  = internal friction coefficient of the granular ice accumulation.

Before using eq 109,  $\tau_i$  and  $\tau_g$  have to be determined using known variables. An expression for  $\tau_i$  is determined by first assuming that Manning's equation can be applied for the average flow condition prevailing in the river:

$$V = \frac{1}{n_i} R_i^{2/3} S_f^{1/2} = \frac{1}{n_c} R^{2/3} S_f^{1/2} \quad (111)$$

where  $R_i$  = hydraulic radius for the portion of the flow affected by the resistance on the underside of the ice cover

$R$  = hydraulic radius of the entire flow cross section

$V$  = average flow velocity

$S_f$  = friction slope of the flow

$n_c$  = composite Manning's coefficient.

The following expression is obtained from eq 111 for the shear stress due to flow:

$$\tau_i = \rho g R_i S_f = \rho g \left( \frac{n_i}{n_c} \right)^{3/2} R S_f. \quad (112)$$

The weight component of the ice cover in the direction of river flow  $\tau_g$  is obtained by taking the weight of ice and water in the voids into account:

$$\tau_g = \rho_i g h_o S_f. \quad (113)$$

An expression for  $\bar{S}_f$  is obtained using Manning's equation and approximating  $R$  as  $A/2B$ :

$$S_f = 2^{4/3} Q^2 n_c^2 \left[ \frac{B^{4/3}}{A^{10/3}} \right]. \quad (114)$$

Substituting the expressions for  $\tau_g$ ,  $\tau_i$  and  $S_f$  in eq 109 and using a new variable  $\mu$  for  $\mu_1 K_2$ ,

$$F_4(A, h_o) = \mu \frac{\rho_i}{\rho} \left(1 - \frac{\rho_i}{\rho}\right) h_o^2 + \frac{2\tau_c h_o}{\rho g} - 2^{1/3} \frac{Q^2 n_c^2 B^{4/3}}{A^{7/3}} \left[ \left( \frac{n_i}{n_c} \right)^{3/2} + 2 \frac{\rho_i}{\rho} \frac{h_o B}{A} + B C_a \frac{p_a}{p} V_a |V_a| \cos \theta_a \right] = 0. \quad (115)$$

Equation 99 is the second equation required in the procedure. Solutions of  $A$  and  $h_o$  are obtained by solving the two nonlinear equations  $F_1(A, h_o) = 0$  and  $F_4(A, h_o) = 0$  simultaneously.

The solution of eq 99 and eq 115 can give two sets of roots for  $A$  and  $h_o$ . The two roots of  $h_o$  correspond to those obtained in the derivation by Pariset and Hausser (1961). In both cases the smaller root of  $h_o$  gives the correct solution.

Equation 107 for wide jams derived by Pariset and Hausser (1961) gives the condition for the existence of roots in the wide jam equation. A similar condition exists for the present analysis. When the flow rate is larger than a certain critical value, no solution exist for eq 99 and 115. This indicates that there is a critical Froude number beyond which progression will not occur. This is similar to eq 4.32 obtained by Pariset and Hausser (1961).

The interaction between ice cover progression and river flow is more significant in shallow rivers than in deep rivers, due to the large relative thickness  $h_o A/B$  in a shallow river. The present procedure has an additional length parameter  $\Delta x$  in the formulation. Values of thickness and water depth obtained by this procedure are more accurate for smaller values of  $\Delta x$ .

#### *Effect of freezing and cover stability*

Michel (1986) pointed out that it takes only a little freezing to form a solid crust near the top surface of the ice cover. This can prevent shoving. Shoving can take place at any time after formation when the internal strength is not capable of withstanding the external forces. Conditions of failure of an ice cover due to shoving can be formulated by considering the force balance of a section of ice cover along the longitudinal direction.

Assuming steady uniform flow and uniform cover thickness, a force balance can be expressed using eq 109. For shoving to occur, the condition to be satisfied is given by

$$2(\tau_c h_o + \mu_1 f) \Delta x < (\tau_i + \tau_g + \tau_a) B \Delta x. \quad (116)$$

Considering that the ice cover consists of a granular accumulation with a solid ice crust near the top surface and a frazil deposit on its underside, the maximum longitudinal force  $f$  that can be exerted by the ice cover per unit width can be expressed in terms of maximum stresses in the ice cover layers:

$$f = f_i + f_n + f_f \quad (117)$$

where the subscripts  $i$ ,  $n$  and  $f$  represent contributions from the solid crust, the granular layer and the frazil ice layer, respectively. Then

$$f_i = \sigma t_i \quad (118)$$

$$f_n = \rho K_2 \left( 1 - \frac{\rho_i}{\rho} \right) g \left[ \frac{1}{2} \frac{(1-e_n)}{(1-e_c)} t_n^2 + \left( \frac{1-e_f}{1-e_c} \right) t_f t_n \right] \quad (119)$$

$$f_f = \rho K_2' \left( 1 - \frac{\rho_i}{\rho} \right) g \frac{1}{2} \frac{(1-e_f)}{(1-e_c)} t_f^2 \quad (120)$$

where  $t_i$ ,  $t_f$  and  $t_n$  = thicknesses of the solid ice layer, frazil ice layer, and the granular layer, respectively  
 $e_f$  and  $e_n$  = porosities of frazil ice and initial cover, respectively  
 $\sigma$  = strength of the solid ice crust.

According to Michel (1986),  $\sigma \approx 0.8$  MPa. The weight component of the ice cover in the direction of the flow is

$$\tau_g = [\rho_i g t_i + \rho_i g t_n (1 - e_n) + \rho g t_n e_n + \rho_i g t_f (1 - e_f) + \rho g t_f e_f] S_f \quad (121)$$

$$= [\rho_i (t_i + t_n + t_f) + (\rho - \rho_i) (t_f e_f + t_n e_n)] g S_f. \quad (122)$$

Using eq 112 to express  $\tau_i$  and assuming  $\tau_a = 0$ , the right side of eq 116 can be expressed as

$$(\tau_i + \tau_g) B = \rho g S_f B \left\{ \left( \frac{n_i}{n_c} \right)^{3/2} R + \frac{\rho_i}{\rho} [t_i + (1 - e_n) t_n + (1 - e_f) t_f] + e_n t_n + e_f t_f \right\} \quad (123)$$

and the left side can be expressed assuming  $K_2' = K_2$ :

$$2(\tau_c h_o + \mu_1 f) = 2\tau_c(t_i + t_n + t_f) + 2\mu_1 \left\{ \sigma t_i + \frac{\rho g K_2}{(1-e_c)} \left( 1 - \frac{\rho_i}{\rho} \right) \left[ \frac{1}{2} (1 - e_n) t_n^2 + (1 - e_f) t_n t_f + \frac{1}{2} (1 - e_f) t_f^2 \right] \right\}. \quad (124)$$

$K_2$  is defined as in eq 110. Further study is needed for estimating the value of  $K_2$ . The failure of an ice cover in a river reach can be checked using eq 116 because all ice and hydraulic conditions in eq 123 and 124 are known. Equation 124 can also be written as

$$(2\tau_c t + \mu_1 f) = 2\tau_c(t_i + t_n + t_f) + 2\sigma\mu_1 t_i + \frac{2\mu\rho g}{(1-e_c)} \left[ \frac{1}{2} (1 - e_n) t_n^2 + (1 - e_f) t_n t_f + \frac{1}{2} (1 - e_f) t_f^2 \right] \quad (125)$$

in which  $\mu = K_2\mu_1$ . The values of the physical constants  $\mu$ ,  $K_2$ , and  $\tau_c$  used in the wide jam case can be used in this case too. The strength of the ice cover against shoving mostly depends on the strength of the crust. The strength of the crust depends on the air temperature and the amount of solar radiation absorbed. The conditions of shoving can be more accurately predicted if the strength of crust is modeled accurately.

## ICE TRANSPORT AND COVER PROGRESSION

### Two-layer model for frazil suspension and surface ice transport

Ice cover progression starts with the formation of frazil ice in a river. In supercooled turbulent water, frazil ice crystals are produced over the entire flow depth. Suspended frazil particles will grow in size, may cluster together, and move up to the water surface to form surface ice runs. This upward movement is governed by the buoyant velocity and the turbulent mixing. Surface ice flocs can collect into ice pans and floes when traveling along the river. Water in the interstices can freeze to decrease the porosity of the surface ice pieces. The thickness of the surface ice pieces can increase due to the accumulation of frazil ice particles on the underside of these ice pieces and the downward freezing caused by surface heat losses.

Besides buoyant velocity and turbulent mixing, the amount of ice in the surface layer depends on the travel time and the rate of surface heat exchange. When the travel time increases, the fraction of ice discharge on the surface increases. A complete formulation for the process of frazil transport and the evolution of surface ice discharge does not exist, although the vertical transport of frazil ice suspension in a river may be described by the following advection–diffusion equation (Shen and Harden 1978):

$$\frac{\partial c}{\partial t} + u \frac{\partial c}{\partial x} + V_b \frac{\partial c}{\partial y} = \frac{\partial}{\partial x} \left( \epsilon_x \frac{\partial c}{\partial x} \right) + \frac{\partial}{\partial y} \left( \epsilon_y \frac{\partial c}{\partial y} \right) - \frac{\phi_v}{\rho_i L_i d} \quad (126)$$

where  $c$  = volumetric concentration of the frazil suspension

$u$  = longitudinal velocity

$V_b$  = buoyant velocity of frazil ice

$\epsilon_x$  and  $\epsilon_y$  = horizontal and vertical mixing coefficients

$d$  = depth of flow

$\rho_i$  = density of ice

$L_i$  = latent heat of fusion of ice

$\phi_v$  = net rate of heat gain per unit volume due to absorption of short-wave radiation.

The variable  $\phi_v$  is a function of the distance from the water surface and the characteristics of the surface ice layer. In this study a simplified two-layer approach is used.

In the following discussion the ice discharge in the river is considered to consist of a surface layer and a suspended layer, as shown in Figure 14. The thickness of the surface layer, although it varies with time and distance, is considered to be negligible compared to the flow depth. The suspended layer is assumed to extend approxi-

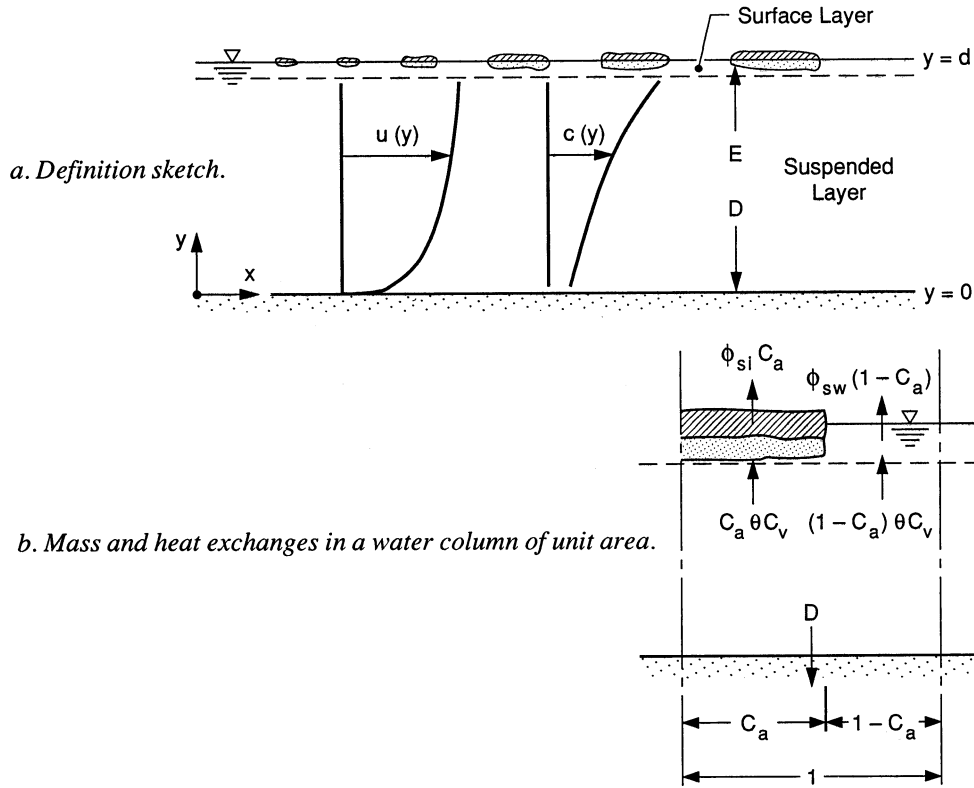


Figure 14. Two-layer system for ice transport.

mately over the entire depth. In the present model the influence of moving ice on the flow, as well as the dynamic interactions between ice particles, is neglected. Volumetric rates of ice transport in these two layers are given by the following expressions:

$$Q_s^i = [h_i + (1 - e_f)h_f]C_a B u \quad (127)$$

$$Q_d^i = C_v A u \quad (128)$$

where  $Q_s^i$  and  $Q_d^i$  = volumetric rates of ice discharges in the surface and in the suspended layers, respectively

$h_i$  = solid ice thickness in the floating ice pans

$e_f$  = porosity of the frazil ice in the surface layer

$h_f$  = thickness of frazil ice on the underside of floating ice pans

$C_a$  = area concentration, or the fraction of water surface area covered by floating ice particles

$C_v$  = average volumetric concentration of frazil ice in the suspended layer

$u$  = cross-sectional average flow velocity in the river

$B$  = top width of the channel

$A$  = cross-sectional area of the channel.

The velocity of ice floes in the surface layer is assumed to be the same as the mean flow velocity  $u$ .

For the surface layer, the equation of mass conservation can be written as the following:

$$\frac{\partial}{\partial t} \{ [h_i + (1 - e_f)h_f] C_a B \} + \frac{\partial}{\partial x} \{ [h_i + (1 - e_f)h_f] C_a B u \} = \frac{B C_a \phi_{si}}{\rho_i L} + E. \quad (129)$$

Similarly, for the suspended layer, the equation of mass conservation is:

$$\frac{\partial}{\partial t} (C_v A) + \frac{\partial}{\partial x} (C_v A u) = \frac{B (1 - C_a) \phi_{sw}}{\rho_i L} - D - E \quad (130)$$

where  $\rho_i$  = density of ice

$L$  = latent heat of fusion of ice

$\phi_{si}$  = net rate of heat loss per unit area over the portion of the water surface covered by ice

$\phi_{sw}$  = net rate of heat loss per unit area over the open water portion of the water surface

$D$  and  $E$  = net exchanges of ice flux at the bed and the interface between the surface and the suspended layers, respectively.

Longitudinal dispersion of both volumetric and surface concentrations is neglected, assuming concentration gradients are small.

The rate of mass exchange at the interface between the two layers can be expressed as

$$\begin{aligned} \frac{E}{B} &= \left( -\epsilon_y \frac{\partial C_v}{\partial y} + V_b C_v \right)_{\text{at } y=d} \\ &= (\alpha V_b c)_{\text{at } y=d} - \gamma [h_i + (1 - e_f) h_f] C_a \end{aligned} \quad (131)$$

where  $\alpha$  is a coefficient representing the probability that frazil reaching the surface layer will remain there and  $\gamma$  is a coefficient representing the speed at which the surface ice is re-entrained into the suspension. Since  $\epsilon_y$  is small near the water surface, it is expected that  $\alpha$  has a value close to 1.0 and  $\gamma$  should be close to zero, except in rapid reaches. If we further assume that the rate of the exchange between the surface and the suspended layers  $E/B$  can be represented by  $\theta V_b C_v$ , and neglect the exchange at the bed  $D$ , eq 129 and 130 become

$$A \frac{DC_v}{Dt} = \frac{B\phi_{sw}(1 - C_a)}{\rho_i L} - \theta V_b C_v B \quad (132)$$

and

$$\begin{aligned} C_a B \frac{D}{Dt} [h_i + (1 - e_f) h_f] + [h_i + (1 - e_f) h_f] \frac{D}{Dt} (C_a B) \\ = \frac{BC_a \phi_{si}}{\rho_i L} + \theta V_b C_v B - [h_i + (1 - e_f) h_f] C_a B \frac{\partial u}{\partial x} \end{aligned} \quad (133)$$

By assuming that the heat loss over floating ice pans is responsible for the growth of solid ice thickness, the rate of change of solid ice thickness can be obtained using a quasi-steady-state finite-difference calculation:

$$\frac{Dh_i}{Dt} = \frac{\phi_{si}}{e_f \rho_i L} \quad (134)$$

Expressing surface heat exchange in the form of  $\phi_{si} = \alpha + \beta(T_s - T_a)$ , in which  $T_s$  is the temperature at the ice surface, the rate of thermal growth of thickness can be determined.

For  $h_f = 0$ , and no frazil supply to the underside of ice pans,

$$\frac{Dh_i}{Dt} = \frac{1}{\rho_i L} = \frac{\alpha + \beta(T_m - T_a)}{\left(1 + \frac{\beta h_i}{K_i}\right)} \quad (135)$$

For  $h_f > 0$ ,

$$\frac{Dh_i}{Dt} = \frac{1}{e_f \rho_i L} = \frac{\alpha + \beta(T_m - T_a)}{\left(1 + \frac{\beta h_i}{K_i}\right)} \quad (136)$$

where  $T_m$  = melting point of ice  
 $T_a$  = air temperature  
 $K_i$  = thermal conductivity of ice  
 $\alpha$  and  $\beta$  = known constants in the heat exchange model.

Equation 136 is valid only when the solid ice does not grow beyond the bottom of the frazil accumulation at the end of the time step.

The rate of change of frazil ice thickness depends on the rate of frazil ice deposition on the underside of ice pans and the rate of downward growth of solid ice into the frazil ice deposit:

$$\frac{Dh_f}{Dt} = \frac{\theta V_b C_v}{(1 - e_f)} - \frac{Dh_i}{Dt}. \quad (137)$$

Substituting eq 136 into eq 137 yields

$$\frac{Dh_f}{Dt} = \frac{\theta V_b C_v}{(1 - e_f)} - \frac{1}{e_f \rho_i L} \frac{\alpha + \beta (T_m - T_a)}{\left(1 + \frac{\beta h_i}{K_i}\right)}. \quad (138)$$

As with eq 136, eq 138 is valid only when the solid ice does not grow beyond the bottom of the frazil ice accumulation at the end of the time step. Since during the time step  $\Delta t$  the thickness of frazil deposited on the underside is

$$\frac{\theta V_b C_v}{(1 - e_f)} \Delta t,$$

the time required for the solid ice grow to the bottom of the frazil ice accumulation is

$$\Delta t' = \frac{\theta V_b C_v \Delta t}{(1 - e_f)} \left[ \frac{1 + \frac{\beta h_i}{K_i}}{\alpha + \beta (T_m - T_a)} \right]. \quad (139)$$

If  $\Delta t' < \Delta t$ , i.e.

$$\frac{\theta V_b C_v}{(1 - e_f)} < \frac{\alpha + \beta (T_m - T_a)}{e_f \rho_i L \left(1 + \frac{\beta h_i}{K_i}\right)},$$

then additional solid ice growth will occur during a period  $\Delta t - \Delta t'$  according to the rate described in eq 135. Then the overall rate of growth of solid ice can be expressed as

$$\frac{Dh_i}{Dt} = \theta V_b C_v + \frac{1}{\rho_i L} \frac{\alpha + \beta (T_m - T_a)}{\left(1 + \frac{\beta h_i}{K_i}\right)}. \quad (140)$$

Equation 138 is valid only if  $\Delta t' > \Delta t$ .

Using eq 135 and 137 (or their equivalent), eq 133 can be further reduced to

$$[h_i + (1 - e_f)h_f] \frac{D}{Dt} (C_a B) = (1 - C_a) \theta V_b C_v B - [h_i + (1 - e_f)h_f] C_a B \frac{\partial u}{\partial x}. \quad (141)$$



This equation describes the increase of ice area concentration in the surface layer due to lateral accretion and flow convergence. Solutions of variables  $h_i$ ,  $h_f$ ,  $C_v$  and  $C_a$  can be obtained using equations discussed above. The model has one calibration constant,  $\theta$ . Since there is no analytical means to determine  $V_b$  accurately, and  $V_b$  always appears with  $\theta$ , calibration can be done by considering  $\theta V_b$  as a single parameter. In river channels, both the ice pieces in the surface layer and the frazil ice in the suspended layer are mixtures of particles of different sizes. These size distributions are not considered in the present model. Values of  $h_f$ ,  $h_i$  and  $\theta V_b$  can be considered as weighted average values for the mixture.

#### *Solutions for two-layer model equations*

An analytical solution for the governing equations for the two-layer model cannot be obtained. A simplified solution for the surface layer, neglecting the existence of the suspended layer, was obtained by Hausser et al. (1984) to examine the effect of floating ice on river ice production. Their solution used the following assumptions: a) all the surface heat exchange over the open water area contributes to the lateral growth of floating ice, and b) the thickness of floating ice is computed by assuming that the surface temperature of ice floes is equal to the air temperature.

The presence of surface ice floes on the water surface reduces the surface heat exchange and the total ice production rate in the river. The present model uses a quasi-steady-state finite-difference solution to estimate the effect of a surface ice layer on the reduction of heat exchange and the fraction of ice discharge in the surface layer. The parameters  $\bar{\alpha}_c$  and  $\bar{\alpha}_a$ , which account for the effect of a surface layer, are computed separately from the main river ice model at each time step for each reach. The overbar represents the average value for a river reach. Definitions of these parameters are given by eq 142 and 143:

$$\alpha_c = \frac{Q_s^i}{Q^i} = \frac{Q_s^i}{Q_s^i + Q_d^i} \quad (142)$$

and

$$\alpha_a = \frac{Q^i}{Q_o^i} \quad (143)$$

$Q_s^i$  and  $Q_d^i$  are as defined in eq 127 and 128, and  $Q_o$  is the rate of ice discharge if the effect of surface ice on ice production is neglected. The value of  $Q_o$  can be calculated from  $Q_o^i = QC_o$  and

$$\frac{DC_o}{Dt} = \frac{h_{wa}(T_m - T_a)}{\rho_i L} \quad (144)$$

where  $C_o$  is the volumetric concentration of ice calculated by neglecting the effect of surface ice.

In the present study the variation of ice conditions is solved using the Euler method (Burden and Faires 1985). By assuming a quasi-steady condition,  $DF/Dt$  in eq 132, 134, 137 and 141 can be replaced by  $u(\partial F/\partial x)$ . The terms on the right side are computed using values of the previous time step. A sample simulation for a steady-state case is presented below for a channel with uniform flow. The parameters used are

Air temperature	$T_a$	$-20.0^\circ\text{C}$
Buoyancy velocity	$V_b$	$0.001 \text{ m s}^{-1}$
Width	$B$	$100.0 \text{ m}$
Frazil porosity	$e_f$	$0.5$
Cross sectional area	$A$	$500 \text{ m}^2$
Step length	$\Delta x$	$100.0 \text{ m}$
Initial solid ice thickness	$(h_i)_{\text{init}}$	$0.005 \text{ m}$
Average flow velocity	$\bar{u}$	$0.5 \text{ m s}^{-1}$

Initial values of  $h_f$ ,  $C_a$  and  $C_v$  are assumed as zero at  $t = 0$ . The results are shown in Figures 15–18. The Euler method was used instead of a second-order Runge Kutta method (Burden and Faires 1985) since  $Dh_i/Dt$  is discontinuous between  $h_f = 0$  and  $h_f > 0$ .

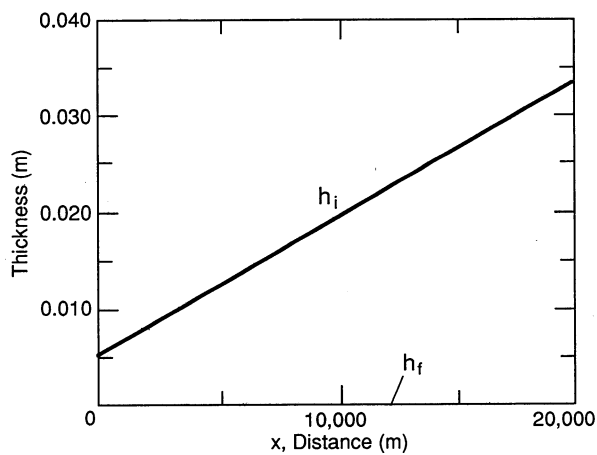


Figure 15. Variation of solid and frazil thickness with distance.

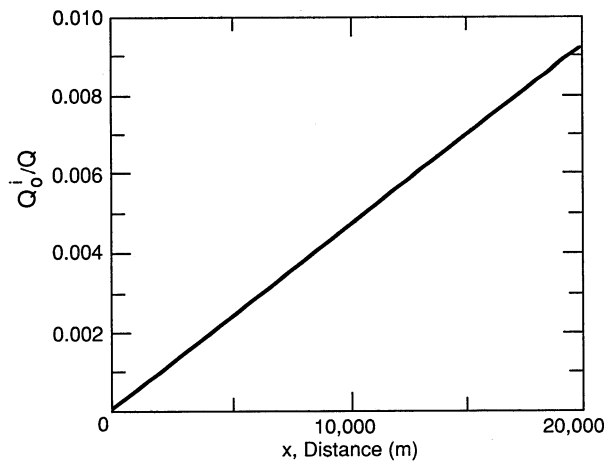


Figure 16. Ice concentration, neglecting the effect of surface ice.

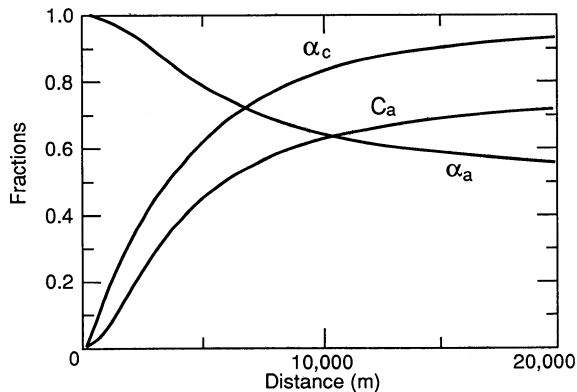


Figure 17. Variations of  $\alpha_c$ ,  $\alpha_a$  and  $C_a$  with distance.

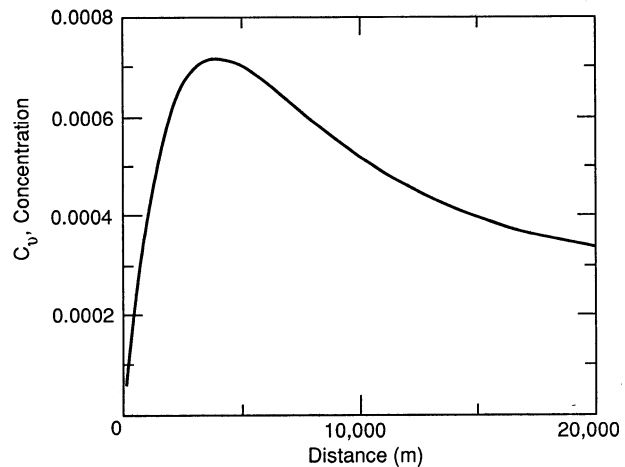


Figure 18. Variation of  $C_v$  with distance.

Figure 15 shows that the frazil ice thickness remains at zero due to the thermal growth of the solid ice. Figure 16 shows that when the effect of surface ice is neglected, the rate of ice production remains constant if the air temperature remains constant. Figure 17 shows that the rate of increase in surface area concentration  $C_a$  decreases with the travel distance due to reductions in free surface area and the suspended concentration  $C_v$ , as shown in Figure 18. The surface fraction of the ice discharge  $\alpha_c$  increases with the increase in  $C_a$ . The ratio  $\alpha_a$  between the total ice discharge and the ice discharge calculated by neglecting the surface ice effect decreases with increasing  $C_a$ .

Parameters  $\alpha_a$  and  $\alpha_c$  are used to modify the quantities of ice discharge computed in the main program, neglecting the effect of surface ice. In applying the parameter  $\alpha_a$ , the main program is first used to compute the ice production assuming no surface ice effect. The result is then multiplied by  $\alpha_a$  to obtain the correct ice discharge. The amount of ice in the surface layer that contributes to the cover progression is obtained by multiplying the ice discharge rate by a factor  $\alpha_c$ .

Values of  $\alpha_c$  and  $\alpha_a$  are computed by applying the two-layer governing equations separately for each reach as described earlier. Computations are carried out reach by reach along the river starting from the upstream end of the most upstream reach having 0°C water temperature or zero isotherm. These computations are continued up to the nearest leading edge and commenced again at the zero isotherm in the next open water area. Values of variables at the downstream end of a reach are used as the upstream boundary conditions for the next reach

downstream. The longitudinal element length in this computation is obtained by dividing a reach into 20–40 equal sections to ensure the stability of the numerical scheme. Variables  $u$ ,  $A$  and  $B$  are assumed to be constant in a reach. Reach-averaged values of  $\alpha_c$  and  $\alpha_a$  are calculated by

$$\overline{\alpha_c} = \frac{\alpha_c^u + \alpha_c^d}{2} \quad (145)$$

and

$$\overline{\alpha_a} = \frac{Q_d^i + Q_u^i}{Q_{o,d}^i + Q_{o,u}^i} \quad (146)$$

where  $u$  and  $d$  represent values at the upstream and downstream ends of the reach. Reach-averaged values of thicknesses  $h_i$  and  $h_f$  are computed in a similar manner. These values are used as the minimum ice thickness in the ice cover formation computation.

#### *Two-layer model for skim ice run*

Skim ice runs can occur in a river reach when the underlying water temperature is above freezing. In this case the lower layer does not have frazil ice suspension, and the surface ice concentration is assumed to be approximately 100%. Assuming that the entire water surface area is covered by skim ice that is free to drift with the flow, the rate of transport of skim ice is

$$Q_s^i = B h_i \bar{u} \quad (147)$$

where  $h_i$  is the skim ice thickness. The cross-section-averaged “water temperature” is

$$T_w = -\frac{B h_i}{A} \frac{\rho_i L}{\rho C_p} + T_d \quad (148)$$

where  $T_d$  is the average water temperature over the flow cross section underneath the surface layer.

The rate of growth of skim ice thickness can be calculated from

$$\frac{Dh_i}{Dt} = \frac{1}{\rho_i L} \left[ \frac{\alpha + \beta(T_m - T_a)}{1 + \frac{\beta h_i}{K_i}} - h_{wi}(T_d - T_m) \right] \quad (149)$$

The convergence and divergence effects caused by the changing river width are neglected. The rate of change of water temperature in the lower layer can be expressed as

$$\frac{DT_d}{Dt} = \frac{h_{wi}(T_d - T_m)}{\rho C_p A} \quad (150)$$

Equations 149 and 150 can be solved for  $h_i$  and  $T_d$  using the Lagrangian method. In the current model, quasi-steady-state solutions of the equations are obtained using the Euler method discussed earlier. Reach-averaged values of  $h_i$  are used in the model for ice cover progression.

#### **Progression of the initial ice cover**

The initiation of an ice cover occurs as a result of the obstruction of a surface ice run by natural or artificial obstacles. Natural obstacles are usually surface ice bridges formed by the congestion of surface ice runs. Static ice covers formed across the river in a slow-flow region can also act as a natural obstruction to ice runs. The most common artificial obstructions to ice runs are ice control structures, ice booms, weirs, bridges and dams.

The phenomenon of ice bridging is not well understood, even though ice bridges usually form at the same location of a river each winter. The formation of an ice bridge at a river section is related to the ice transport capacity of the section and the rate of ice discharge from upstream. The maximum rate of ice discharge that can pass through a river section not forming an ice bridge is dependent on the flow discharge, the channel top width between banks or shore ice, the surface slope, and the size and concentration of the surface ice layer, among other things (Shen et al. 1988).

When the hydraulic conditions permit, the leading edge of an ice cover will progress upstream due to the accumulation of surface ice against the leading edge. This type of ice cover progression depends on the rate of ice supply and the thickness of the ice cover formed. The conservation of surface ice mass at the leading edge gives

$$(Q_s^i - Q_u) \frac{(V_s + V_{cp})}{V_s} = Bh_o(1 - e)(1 - e_p)V_{cp}. \quad (151)$$

The rate of progression of the leading edge  $V_{cp}$  can be obtained from

$$V_{cp} = \frac{(Q_s^i - Q_u)}{Bh_o(1 - e)(1 - e_p) - \frac{(Q_s^i - Q_u)}{V_s}} \quad (152)$$

where  $V_{cp}$  = rate of progression of the leading edge

$Q_u$  = volumetric rate of ice entrainment under the cover at the leading edge

$B$  = width of the ice cover

$e$  = porosity of individual ice floes

$e_p$  = porosity in the accumulation representing voids between floes

$h_o$  = initial cover thickness calculated using the method discussed earlier

$V_s$  = average velocity of the incoming surface ice particles.

Equation 152 is valid when

$$Bh_o(1 - e_p)(1 - e) > \frac{(Q_s^i - Q_u)}{V_s};$$

otherwise the progression will take place at an infinite speed with a larger thickness  $h'_o$  given by

$$h'_o = \frac{Q_s^i - Q_u}{B(1 - e_p)(1 - e)V_s}. \quad (153)$$

Further discussion of this case is given in the next section. When

$$Bh_o(1 - e_p)(1 - e) >> \frac{(Q_s^i - Q_u)}{V_s}$$

i.e.,  $V_{cp} \ll V_s$ , eq 151 reduces to

$$V_{cp} = \frac{(Q_s^i - Q_u)}{(1 - e)(1 - e_p)h_oB}. \quad (154)$$

#### Computational procedure

Due to the advection of ice in the river, ice upstream from the leading edge at the beginning of a time step can reach the leading edge before the end of that time step. This leads to a continuous change in the rate of ice

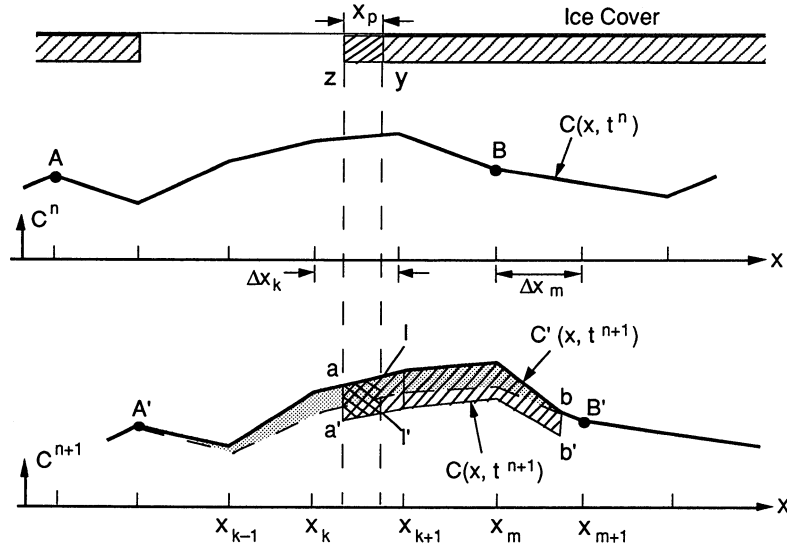


Figure 19. Definition sketch for the computation of ice cover progression.

supply at the leading edge during a time step. This effect is included in the model using a Eulerian–Lagrangian numerical procedure, similar to the water temperature simulation discussed earlier. This procedure may be considered to be a generalized upwind scheme commonly used in simulating transport processes.

The numerical procedure used in modeling ice cover progression is formulated by considering the following phenomena:

- The ice discharge that passes the leading edge during the time interval  $\Delta t$  will contribute to the leading edge progression during that time interval;
- The ice discharge includes contributions from the production due to surface heat exchanges and the ice volume that will be added to the ice discharge if there is a collapse of the existing ice cover;
- Only ice in the surface layer, which is assumed to be equal to an  $\alpha_c$  fraction of the ice discharge, will contribute to the volume of ice supply for cover progression;
- The leading edge position will migrate upstream during time  $\Delta t$ ; and
- If the leading edge progression stops before the end of the time step due to the effect of the hydraulic condition, the ice discharge passing through the leading edge after the cessation of progression will not contribute to the cover progression.

Consider the progression of the leading edge of an ice cover as shown in Figure 19. The leading edge progresses from Y to Z during the time interval between  $t^n$  and  $t^{n+1}$ . The Lagrangian scheme will first compute the ice concentration profile  $C'(x, t^{n+1})$  assuming no ice cover progression. This concentration profile includes the ice production and the contribution from the collapsed surface ice cover. During this time step, ice particles located at A and B will move to A' and B'. The amount of ice passing the leading edge Y that can contribute to ice cover progression is given by the shaded area ll'b'b. The volume of the ice is given as

$$V_c = \bar{\alpha}_k r_k \bar{C}_k \Delta x_k A_k + \sum_{i=k+1}^{m-1} \bar{\alpha}_i \bar{C}_i \bar{A}_i \Delta x_i + \Delta t' \bar{u}_m \bar{C}_m \bar{A}_m \quad (155)$$

$$\text{where } \Delta t' = \Delta t - \left( \frac{r_k \Delta x_k}{u_k} + \sum_{i=k+1}^{m-1} \frac{\Delta x_i}{u_i} \right) \quad (156)$$

$r_k$  = fraction of the ice-covered length in the  $k^{th}$  reach

$\bar{\alpha}_c$  = fraction of the total ice discharge contributing to cover formation

$\bar{u}_i$  = average flow velocity in  $i^{th}$  reach

$\bar{C}_i$  = average of concentration  $C'(x, t^{n+1})$  of ice in  $i^{th}$  reach

$\bar{A}_i$  = average cross-sectional area of the  $i^{th}$  reach

$\Delta x_i$  = length of the  $i^{th}$  reach

$k$  = index number of the reach that contains the leading edge  $Y$  at time  $t^n$

$m$  = reach number where a parcel starting at the leading edge  $Y$  at time  $t^n$  is located at the end of time step  $\Delta t$ .

If progression ceases before the end of the time interval, then the portion of the ice volume that passed the leading edge after the cessation of progression should not be included in eq 155. This part of the concentration profile  $C'(x, t^{n+1})$  is restored to its original shape.

When the volume of the ice supply  $V_c$  is known, the length of progression  $x_p$  during  $\Delta t$  is computed using

$$B h_o x_p (1 - e_c) = V_c + \bar{\alpha}_c x_p \bar{C}_p \bar{A}_p \quad (157)$$

which gives

$$x_p = \frac{V_c}{B h_o (1 - e_c) - \bar{\alpha}_c \bar{C}_p \bar{A}_p} \quad (158)$$

where  $h_o$  = average thickness of the initial cover

$\bar{B}$  = width of the new ice cover

$A$  = average cross-sectional area of the river section

$e_c$  = effective porosity of the ice cover

$\bar{C}_p$  = average ice concentration

$p$  = subscript representing the reach containing the newly formed ice cover.

The case  $B h_o (1 - e_c) \leq \bar{\alpha}_c \bar{C}_p \bar{A}_p$  implies that a large amount of surface ice exists in the river. This large quantity of surface ice is more than enough to form the initial cover of thickness  $h_o$ . Although rare in the field, this case is treated by assuming that the initial cover in the reach will form at a thickness larger than  $h_o$ . This thickness depends on the ice supply and is calculated by

$$h' = \frac{V_c + \bar{\alpha}_c x_p' \bar{C}_p \bar{A}_p}{B x_p' (1 - e_c)} \quad (159)$$

where  $x_p' = x_k - x_Y$ .

When the volume  $V_c$  is removed from the concentration profile  $C'(x, t^{n+1})$ , the remaining concentration profile cannot be correctly represented by only the remaining nodal point concentrations. An interpolation procedure is used to obtain equivalent nodal concentrations for  $C(x, t^{n+1})$ , so that the total volume of the ice and the center of mass of the ice contained in each reach remain unchanged. The latter restriction is introduced to avoid artificial advection. This procedure was discussed in detail earlier.

The ice cover cannot progress beyond a cross section where the maximum local Froude number along the width of the cross section exceeds the critical Froude number. In a one-dimensional model a considerable amount of geometrical information is lost during the schematization. It is possible for a reach having an average Froude number less than  $F_{rc}^*$  to have a cross section that has a local Froude number exceeding  $F_{rc}^*$ . In the present model, locations of possible critical cross sections, and the ratio between the local Froude number and the average Froude number of the schematized reach, are determined from the hydrographic charts. When these values are provided as input conditions, local Froude numbers can be computed from the average Froude number of the river reach. The control on the progression of ice cover can then be modeled correctly.

## UNDERCOVER DEPOSITION AND EROSION

Transport and deposition of ice particles in ice-covered reaches are discussed in this section. The transport of ice particles in an ice-covered channel is similar to the transport of sediments in an alluvial river, except that ice particles move up due to buoyancy whereas sediment particles settle down. Based on this analogy, the trans-

port of ice along the river can be considered to consist of a cover load and a suspended load. Ice particles traveling on the underside of the cover at a given location can have a number of origins. They include:

- Frazil ice produced in the upstream reaches remaining in suspension (this ice will add to the undercover accumulation when it floats to the ice cover);
- Ice particles eroded from an undercover accumulation of an upstream reach or from the collapse of an existing ice cover; and
- Surface ice discharge that was swept under the ice cover at the leading edge due to unfavorable conditions for progression.

Laboratory and field studies of frazil ice transport are difficult due to the effect of thermal regimes and adverse field conditions. The present understanding of the undercover transport is rather limited. Large accumulations of ice particles on the underside of an ice cover are commonly known as hanging dams. Hanging dams may be classified into two categories depending on the formation process (Shen et al. 1983). The first type of hanging dams, referred to as surface ice hanging dams, are accumulations of large plates or frazil ice pans, which were surface ice particles overturned at the leading edge during ice cover progression. Surface ice hanging dams are formed near the leading edge of an ice cover, usually during its upstream progression. The second type of hanging dams, referred to as frazil ice hanging dams or frazil ice jams, are formed by the accumulation of suspended frazil ice particles on the underside of an ice cover.

There are two mechanisms by which surface ice hanging dams are formed. In the first mechanism, ice floes that are released from upstream reach the leading edge of an ice cover, submerge, move along the undersurface of the cover, and get arrested at some point downstream. These floes can accumulate until a hanging dam is formed. In the second mechanism the external forces acting on the ice cover exceed its strength, causing the ice cover to collapse upon itself and subsequently thicken. This second type of hanging dam is essentially a localized ice jam. Both mechanisms can occur either at the beginning of the winter during the formation of the new cover or during the spring break-up period when ice floes are generated from the fragmentation of ice covers.

Frazil ice particles suspended in a river are subjected to the buoyancy force, which tends to move ice particles upward, and turbulent mixing, which tends to disperse the particles and effectively move them from high- to low-concentration regions. Undercover deposition requires a net upward movement of particles to bring them to the underside of the cover. As the effective size of ice particles increases with time, buoyancy overcomes turbulent mixing to create a net upward movement. This upward movement can also take place when ice particles move into a slow-flow area where turbulence intensity is low. Deposition or erosion of frazil ice particles on the underside of the ice cover can change the size of a frazil ice jam.

A critical velocity, or Froude number, criterion has long been accepted as a means of determining frazil ice jam or hanging dam thicknesses (Kivislid 1959, Michel and Drouin 1975, Tesaker 1975, Ashton 1986). In the critical velocity concept, ice particles will be deposited on the underside of the ice cover if the local flow velocity is low. Changes in hanging dam size can change the flow condition. When the thickness of a hanging dam increases, the flow velocity will also increase due to the reduction in flow cross section. Deposition will cease when the velocity is high. Ice erodes from frazil ice deposits when the flow velocity is high and ice particles can no longer resist the hydrodynamic force exerted on them.

Field observations have indicated that the critical velocity varies not only from river reach to river reach but also from time to time in a given reach. We believe that the undercover deposition or erosion is governed by the ice transport capacity of the flow. Deposition will occur when the ice discharge exceeds the transport capacity of the river flow. Similarly, deposits that are not frozen onto the cover will erode when the ice discharge is less than the ice transport capacity of the flow. Unfortunately a theory does not exist that can determine the ice transport capacity of a river. Simplified critical deposition and erosion criteria will be used in the study. Erosion and deposition can take place in different parts of a river at the same time. This section describes the one-dimensional formulation used in the present model.

### **Deposition and erosion criteria**

Because of the lack of understanding of the detailed mechanics of undercover transport and hanging dam formation, a simple critical velocity criterion is used to determine the location and size of hanging dams. Both

critical velocity and critical Froude number criteria have been suggested in the literature (Kivislid 1959, Tesaker 1975). Using the laboratory data of Filippov (1974), Ashton (1975) obtained empirical relationships for under-cover travel distance of ice floes:

$$\frac{L}{l} = f\left(\frac{V}{\sqrt{gl \frac{\Delta\rho}{\rho}}}, n_i, \frac{t_i}{l}\right) \quad (160)$$

where  $L$  = undercover travel distance of an ice floe before its rest  
 $V$  = mean velocity upstream of the cover  
 $\Delta\rho = \rho - \rho_i$   
 $l$  = length of an ice block  
 $\rho$  and  $\rho_i$  = densities of ice and water.

Equation 160 gives some insight into the possible patterns of ice deposition near the leading edge.

Particle stability theories can also be used to calculate accumulation thickness by determining the critical conditions for local velocity, depth and ice characteristics that allow the deposition or erosion of ice particles. Tatinclaux and Gogus (1981) carried out a laboratory study aimed at determining the re-entrainment criterion of an ice block resting under the ice cover. The following equation was developed by considering the rotational stability of an ice floe along with laboratory experiments using simulated floes:

$$F_e \equiv \frac{V_c}{\left[\left(1 - \frac{\rho_i}{\rho}\right)gh\right]^{1/2}} = \frac{1}{\left[c_1\left(\frac{h}{L}\right)^2 + c_2\left(\frac{h}{L}\right) + c_3\right]} \quad (161)$$

where  $F_e$  = densimetric Froude number  
 $V_c$  = critical velocity under the cover for erosion  
 $\rho$  = density of water  
 $\rho_i$  = density of ice  
 $h$  = block thickness  
 $L$  = length of the block  
 $c_1 = -2.26$   
 $c_2 = 2.14$   
 $c_3 = 0.015$ .

At velocities greater than  $V_c$ , thickening should not occur. These laboratory studies are not sufficient to make them usable in field applications.

Field studies in the upper St. Lawrence River suggested that the critical velocity for ice deposition is around  $3.0 \text{ ft s}^{-1}$  (Shen et al. 1983). These studies also show that hanging dams are affected by the transverse distribution of the river flow. Once deposited in a hanging dam, frazil ice undergoes morphological and structural changes, and the critical velocity can change accordingly.

Based on observations in the LaGrande River, Michel and Drouin (Ashton 1986) suggested the following critical velocities for ice deposition:

- In narrow sections the critical velocity ranges from  $0.9 \text{ m s}^{-1}$  at the beginning of the winter to  $0.5 \text{ m s}^{-1}$  at the end.
- In wide sections the critical velocity ranges from  $0.8 \text{ m s}^{-1}$  at the beginning of the winter to  $0.5 \text{ m s}^{-1}$  at the end.
- After making corrections for back eddies and low flows, the local velocity of deposition is  $0.9 \text{ m s}^{-1}$ .

In a recent study in the Yellow River near Hequ, Sun and Shen (1988) obtained an empirical relationship between the thickness of frazil accumulation and the Froude number. They pointed out the inadequacy of the critical velocity criterion. However, in view of the lack of a better method, the critical velocity criterion is used in the present model.



### Simulation of undercover deposition

Both ice cover progression and undercover erosion and deposition depend on the frazil ice concentration distribution in the river. As explained earlier, progression takes place when surface ice reaches the leading edge of an existing ice cover. Similarly, deposition can take place when frazil ice reaches the bottom surface of an ice cover. Surface ice that is swept under the cover at the leading edge will also contribute to the undercover deposition.

The rate of undercover deposition depends on the rate of ice supply reaching the underside of the ice cover. The distribution of frazil ice under an ice cover can be represented by the following transport equation, which is similar to eq 126:

$$\frac{\partial c}{\partial t} + u \frac{\partial c}{\partial x} = \frac{\partial}{\partial x} \left( \epsilon'_x \frac{\partial c}{\partial x} \right) + \frac{\partial}{\partial y} \left( \epsilon'_y \frac{\partial c}{\partial y} - V_b c \right) \quad (162)$$

where  $\epsilon'_x$  and  $\epsilon'_y$  are turbulent mixing coefficients for flow under the ice cover. The source term is not considered in this equation, assuming short-wave radiation does not penetrate the ice cover. The boundary condition at the top boundary is

$$\epsilon_y \frac{\partial c}{\partial y} - v_s (1 - \alpha) c = 0 \quad \text{at } y = d_w \quad (163)$$

where  $d_w$  is the depth of flow measured from the channel bottom to the bottom of the ice accumulation and  $\alpha$  is an adsorption coefficient or the probability that a particle reaching the top surface will remain at the surface. At the channel bottom, assuming there is no bottom heat exchange or anchor ice formation,

$$\epsilon_y \frac{\partial c}{\partial y} - v_s c = 0 \quad \text{at } y = 0. \quad (164)$$

In a one-dimensional model, only depth-averaged concentrations can be considered.

#### Transport of frazil ice

The ice volume that can deposit in a given reach can be computed in a manner similar to the case of progression. The fraction of the total ice discharge  $\beta$  that can contribute to deposition in a given reach is determined first. Since the total ice discharge is known at the end of a Lagrangian step, the volume of ice supply to the deposit can be obtained by multiplying it by  $\beta$ .

In one-dimensional form the mass conservation of the ice in suspension can be written as

$$\frac{\partial}{\partial t} (C_v A) + \frac{\partial}{\partial x} (C_v A u) = B \left[ \left( \epsilon_y \frac{\partial c}{\partial y} - V_b c \right)_{y=d_w} - \left( \epsilon_y \frac{\partial c}{\partial y} - V_b c \right)_{y=0} \right] \quad (165)$$

where  $C_v$  is the depth-averaged ice concentration. Assuming no exchange at the bed and replacing the square bracket term on the right side by  $\theta_1 V_b C_v$ , eq 165 becomes

$$\frac{\partial}{\partial t} (C_v A) + \frac{\partial}{\partial x} (C_v A u) = -B \theta_1 V_b C_v \quad (166)$$

where  $\theta_1 V_b C_v$  represent the rate of deposition of the frazil ice accumulation. Assuming steady uniform flow, this equation can be approximated by

$$A u \frac{dC_v}{dx} = -B \theta_1 V_b C_v. \quad (167)$$

The variation of  $C_v$  along the river becomes

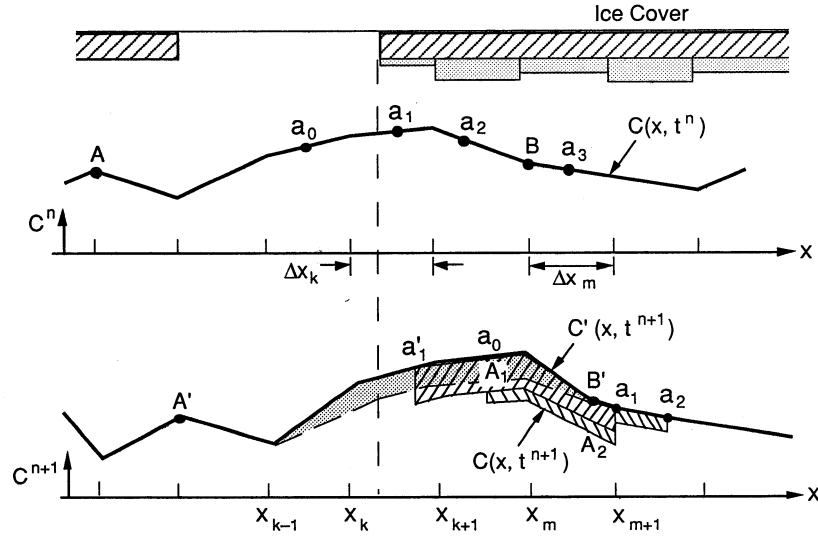


Figure 20. Definition sketch for the computation of under-cover deposition.

$$C_v(x) = C_0 \exp\left(-\frac{\theta_1 V_b B x}{Au}\right) + \text{constant} \quad (168)$$

where  $C_0$  is the concentration at the upstream end. For a reach of length  $\Delta x$ , eq 168 gives

$$C_2 = C_1 \exp\left(-\frac{\theta_1 V_b \Delta x}{Au}\right) \quad (169)$$

where  $C_1$  is the average concentration at the beginning of the reach and  $C_2$  is the average concentration at the end of the reach. Let the coefficient  $\beta$  represent the factor of the reduction of ice concentration over the distance  $\Delta x$ . Then

$$\beta = \frac{C_1 - C_2}{C_1} \quad (170)$$

and

$$\beta = 1 - \exp\left(\frac{\theta_1 V_b B \Delta x}{Au}\right). \quad (171)$$

The concentration of ice remaining in the suspension after deposition is  $(1 - \beta)C_1$ . The parameter  $\theta_1 V_b$  is to be determined by calibration against field data.

Computation of the ice supply that is available for deposition is similar to that for ice cover progression explained earlier. Consider the longitudinal frazil ice concentration profiles at times  $t^n$  and  $t^{n+1}$  as shown in Figure 20. Parcels A and B at time  $t^n$  move to A' and B' at time  $t^{n+1}$ . In the computation, ice particles are first transported to new positions at the end of the time step, as shown by  $C'(x, t^{n+1})$ , ignoring deposition. The volume of ice that is available for deposition to each reach is then calculated and deducted from  $C'(x, t^{n+1})$  at the end of the time step.

Concentrations at the center of each reach, as represented by  $a_0, a_1, a_2$ , and  $a_3$  in Figure 20, are considered to be representative for the reach. The volumes of ice that are available for deposition in reaches  $\Delta x_k$  and  $\Delta x_{k+1}$  are represented by  $A_1$  and  $A_2$  in Figure 20. The volume corresponding to  $A_1$  is

$$V_d = \frac{t_k}{2} \beta_k \bar{C}_k \bar{A}_k \Delta x_k + \sum_{i=k+1}^{m-1} \beta_i \bar{C}_i \bar{A}_i \Delta x_i + \Delta t' \bar{u}_m \bar{C}_m \quad (172)$$

where

$$\Delta t' = \Delta t - \left( \frac{1}{2} \frac{r_k \Delta x_k}{\bar{u}_k} + \sum_{i=k+1}^{m-1} \frac{\Delta x_i}{\bar{u}_i} \right) \quad (173)$$

and  $r_k$  = fraction of ice covered length in the  $k^{th}$  reach

$\beta_i$  = fraction of total ice discharge in the reach that can contribute to deposition

$\bar{u}_i$  = average flow velocity in the  $i^{th}$  reach

$\bar{C}_i$  = average volumetric concentration of ice in the  $i^{th}$  reach determined by interpolation

$A_i$  = average cross-sectional area of reach

$\Delta x_i$  = length of the  $i^{th}$  reach

$k$  = number of the reach that contains the leading edge at time  $t^n$

$m$  = number of the reach where the particle  $B$  (starting at the leading edge at time  $t^n$ ) would be located at the end of the time step.

Ice deposition proceeds reach by reach from the leading edge of the ice cover downstream. A particle moving under the ice cover is to be deposited under the first ice-covered reach it meets unless the flow velocity is above the critical limit. If all the ice volume available cannot be accommodated in a given reach due to the limitation imposed by the critical velocity criterion, the excess ice will remain in the concentration profile and be deposited downstream. The final concentration profile  $C(x, t^{n+1})$  at the end of the time step is obtained by removing all the ice deposited under the ice-covered section from the ice concentration profile  $C'(x, t^{n+1})$ . This final concentration profile is shown by the unshaded area in Figure 20. Conversion of the final shape into a shape that can be expressed using single nodal values is done by interpolation.

#### Thickness of a frazil deposit

The thickness of a frazil deposit under an ice cover depends on the amount of frazil ice that is available for deposition and the hydraulic conditions that determine the limiting condition for deposition. As ice deposits under the ice cover, the hydraulic conditions are affected due to the reduction in the flow cross section. The following derivation assumes that the hydraulic conditions do not change significantly during a time step and that the water level remains unchanged before and after deposition. Based on the critical velocity criterion, the condition for deposition of frazil ice under the ice cover can be expressed as

$$\frac{Q}{A} \leq v_d \quad (174)$$

where  $v_d$  = critical velocity for ice deposition

$A$  = net area of flow

$Q$  = flow rate in the river.

Assuming that the ice cover is floating freely, the following expression is obtained by equating the river cross-sectional areas under the water level before and after deposition during a time step:

$$A_o + B \left( \frac{\rho_s}{\rho_w} h_s + \frac{\rho_i}{\rho_w} h_i + \frac{\rho_n}{\rho_w} h_n + \frac{\rho_f}{\rho_w} h_{fo} \right) = A + B \left( \frac{\rho_s}{\rho_w} h_s + \frac{\rho_i}{\rho_w} h_i + \frac{\rho_n}{\rho_w} h_n + \frac{\rho_f}{\rho_w} h_f \right) \quad (175)$$

where  $\rho_w, \rho_s, \rho_i, \rho_n$  and  $\rho_f$  = densities of water, snow, solid ice, initial cover formed by ice fragments and frazil ice layers, respectively, as defined earlier

$h_{fo}$  and  $h_f$  = frazil ice thicknesses before and after deposition

$B$  = average width of the river.

Substituting  $A$  of eq 174 into eq 175, the following equation is obtained for the maximum thickness of frazil deposition:

$$h_{fc} = h_{fo} + \frac{\rho_f}{\rho_w} \left( \frac{A_o}{B} - \frac{Q}{Bv_d} \right) \quad (176)$$

where  $h_{fc}$  is the maximum allowable thickness of a frazil deposit. The maximum volume of ice that can be deposited with a uniform thickness is given as

$$V_{fm} = B r \Delta x (1 - e_f) (h_{fc} - h_{fo}) \quad (177)$$

where  $r$  is the fraction of the ice-covered length in  $\Delta x$  and  $e_f$  is the porosity of the frazil deposit. Since a simplified formulation is used in treating the interaction between the ice deposition and hydraulics during a time step, the allowable frazil deposit thickness calculated is not precise. The interaction effect can be considered more accurately by a formulation similar to that for ice cover formation described earlier. However, since the rate of deposition is generally small and the interaction will be accounted for in the next time step, the simplification used in the present model is acceptable.

### Erosion of frazil ice

Frazil ice is assumed to erode when the local flow velocity over frazil ice increases beyond a critical value  $v_e$ . The critical velocity of erosion can be expected to be higher than the critical velocity of deposition because the bond between frazil particles in the deposit may have been increased due to sintering and freezing. The incipient condition for frazil ice erosion has some similarity with the incipient conditions for sediment erosion, but modeling in the case of frazil ice is more difficult because of the effects of metamorphism of ice. The model needs  $v_e$  as an input, which can be obtained from field observations or calibrated using observed flow and ice conditions.

As shown in Figure 20, if the  $k^{th}$  reach is subjected to erosion, eroded ice particles will be re-entrained into the flow and distributed between  $a'_1$  and  $a_1$  during the time step  $\Delta t$ . Since there is no theory available for estimating the rate of erosion, the current model assumes a rate of erosion that will add a concentration equal to the current concentration in the river.

When the water temperature is above freezing, part of the eroded frazil ice will first absorb the latent heat of water to melt and bring the water temperature down to  $0^\circ\text{C}$ . This concentration can be computed by distributing the remaining volume of ice in the flow passing over the section. Erosion in warm water (above freezing) is possible during break-up.

The maximum thickness of a frazil deposit that can remain after erosion  $h_{fc}$  is governed by the following critical velocity criterion:

$$\frac{Q}{A} \leq v_e \quad (178)$$

where  $v_e$  is the critical velocity of erosion. An expression for  $h_{fc}$  is

$$h_{fc} < h_{fo} + \frac{\rho_f}{\rho_w} \left( \frac{A_o}{B} - \frac{Q}{Bv_e} \right). \quad (179)$$

The volume of ice that enters the stream as a result of erosion is given by

$$V_{fm} = B r \Delta x (h_{fo} - h_{fc}) (1 - e_f). \quad (180)$$

### THERMAL GROWTH AND DECAY OF ICE COVERS

The thickness of an ice cover can be changed by thermal growth and decay as a result of heat exchange with the atmosphere and the river water. Determining the ice cover thickness is important in river ice modeling. The ice cover thickness can affect the flow cross-sectional area and hence the velocity of the flow. The ice cover thickness is also important in determining the stability of an ice cover and its break-up. A solid ice cover can

consist of layers of black ice and white ice in addition to a snow cover and frazil accumulations. When repeated cycles of heavy snowfalls and cooling take place, the formation of slush layers sandwiched between white ice layers can occur.

The rate of thermal growth and decay of an ice cover is governed by the heat exchange at the top and bottom surfaces and the heat conduction in the ice cover. Heat exchange across the top surface depends on air temperature, wind velocity, humidity, short- and long-wave radiation, albedo and cloud cover. Heat exchange at the bottom surface depends on water temperature and flow velocity. Heat conduction depends on the thermal conductivities of ice, snow or white ice, which are functions of porosity. Since the horizontal extent of the ice cover is much larger than its thickness, the growth or decay of an ice cover can be treated as a one-dimensional problem. Many methods have been used to model the thickness of ice covers.

Although a numerical solution of the full unsteady equations is possible, simple steady-state solutions are commonly used in river ice problems because the thicknesses involved are small. The current model assumes a quasi-steady state and uses a finite-difference method to model the variation of black and white ice thicknesses with time. A linear heat transfer model is used to compute the heat exchange at the top surface, and a turbulent heat exchange coefficient is used to model the heat exchange at the bottom.

The degree-day method (Stefan 1891) has long been used for predicting ice cover thicknesses in lakes and rivers. In this method the ice thickness  $h$  is given as

$$h = \alpha_h \sqrt{S} \quad (181)$$

where  $S = \int_{t_0}^t (T_m - T_a) dt$  = cumulative freezing degree-days of air temperature since the formation of the cover at time  $t_0$

$t$  = time from the formation of the ice cover  
 $T_m$  = freezing temperature of water  
 $T_a$  = air temperature  
 $\alpha_h$  = empirical degree-day constant.

Typical values of  $\alpha$  that have been used for ice covers of different conditions are shown in Table 5. Equation 181 cannot be used to simulate the melting of an ice cover.

**Table 5. Typical values of  $\alpha_h$  (from Michel 1971).**

<i>Ice cover condition</i>	<i><math>\alpha_h</math> (cm °C<sup>-1/2</sup> day<sup>-1/2</sup>)</i>
Windy lake with no snow	2.7
Average lake with snow	1.7–2.4
Average river with snow	0.4–0.5
Sheltered small river with rapid flow	0.7–1.4

Bilello (1980) suggested the use of accumulated thawing degree-days to describe the decay and break-up of ice covers. In his model the ice thickness is given as

$$h = h_{\max} - \alpha_o S_T \quad (182)$$

where  $h_{\max}$  = maximum ice thickness at the beginning of decay  
 $S_T$  = accumulated thawing degree-days  
 $\alpha_o$  = empirical constant.

Shen and Yapa (1985) developed a unified degree-day method for simulating the thermal growth, decay and break-up of river ice covers in the St. Lawrence River. In this model the variation of the ice cover thickness was related to the ambient air temperature using the formula

$$h = (h_0^2 + \alpha_t S)^{1/2} - \beta t^0 \quad (183)$$

where  $h$  = ice cover thickness  
 $h_0$  = initial ice cover thickness  
 $\alpha_i, \beta, \theta$  = empirical constants.

The coefficient  $\alpha_i$  is a function of time.

A number of finite-difference models were developed to determine the ice cover thickness in lakes and seas. Maykut and Untersteiner (1971) presented a one-dimensional model for sea ice that included the effect of snow cover, salinity and internal heating due to short-wave radiation. The model was applied to the central Arctic to simulate the thicknesses of young sea ice when the thickness was on the order of a few meters. In the model, heat exchange at the top surface, ocean heat flux, snow accumulation, ice salinity and albedo were treated as inputs that change with time.

Simulation models for sea or lake ice covers have undergone many improvements over the years (Maykut 1978, Sydor 1978, Wake and Rumer 1979, Miller 1980, Gabison 1987). Ashton (1979) developed models describing the suppression of river ice by a thermal effluent. In Ashton's model the convection of thermal energy in the river water was simulated numerically using a Lagrangian scheme. Surface heat exchange across the air/ice interface and the air/water interface were expressed using simple equations and heat transfer coefficients. Greene (1981) developed a numerical model and an analytical model. These models were used to simulate ice cover thicknesses in the upper St. Lawrence River. A detailed numerical model by Shen and Chiang (1984) treated the river as a coupled air-ice-water-bed system by taking into account the heat exchange at all the interfaces in the system. Simulated results for the St. Lawrence River were in good agreement with observed data.

Leppäranta (1983) and Bengtsson (1984) considered the effect of snow ice formation. In Leppäranta's model, snow slush was directly transformed to snow ice, completely neglecting the thermal process. The snow surface temperature was assumed to be the same as the air temperature. The packing of the snow layer was included using an empirical formulation. Bengtsson considered the effect of the surface thermal resistance and the conductivity of white ice in calculating black ice growth. Unfrozen snow slush between layers of white ice was directly converted to snow ice. The existence of the capillary fringe in the snow slush was neglected in both these studies.

Shen and Lal (1986) included the effects of capillary rise and the layered formation of white ice in a thermal growth and decay simulation model. The insulating effect of the snow layer and the presence of frazil ice were also considered in this model. Heat exchanges at the top and bottom surfaces were computed using a linear model and a turbulent heat exchange coefficient, respectively. The model can simulate thickness variations of black ice, white ice, snow and slush, if the weather conditions, snowfall, frazil thickness and porosities are given. The simulation of thermal growth and decay in the present study is based on a simplified version of the model of Shen and Lal (1986). It is capable of simulating both black and white ice thickness as well as snow and snow slush thicknesses.

All of the previous river ice models assumed quasi-steady-state thermal conditions in the cover. This approximation is justified in view of the small river ice thickness with respect to the time step used in the computation (Greene 1981).

### Surface heat exchanges

The turbulent heat exchange at top and bottom surfaces of an ice cover governs the rate of change of the ice cover thickness. In the present model the heat exchange at the top surface is expressed using a linearized model similar to eq 43 proposed by Dingman and Assur (1969):

$$\phi_T = -\phi_R + \phi_n \quad (184)$$

and

$$\phi_n = \alpha_i + \beta_i(T_s - T_a) \quad (185)$$

where  $\phi_T$  = net heat flux to the atmosphere from ice or snow surface ( $\text{W m}^{-2}$ )  
 $\phi_n$  = net heat flux to the atmosphere excluding short-wave radiation ( $\text{W m}^{-2}$ )  
 $\phi_R$  = net short-wave radiation ( $\text{W m}^{-2}$ )

$T_s$  = surface temperature (°C)  
 $T_a$  = air temperature (°C)  
 $\alpha_i$  and  $\beta_i$  = coefficients that can be derived from the complete heat exchange process including long-wave radiation, evaporation and sensible heat transfer.

To further simplify the analysis, the solar radiation is sometimes lumped into the linear model:

$$\phi_T = \alpha'_i + \beta'_i (T_s - T_a). \quad (186)$$

Since  $\phi_R$  varies with latitude,  $\alpha'_i$  and  $\beta'_i$  values depend on the latitude. Coefficients  $\alpha_i$ ,  $\beta_i$ ,  $\alpha'_i$  and  $\beta'_i$  are functions of wind velocity, cloud cover and relative humidity. The following heat exchange models were obtained from multiple linear regression analysis of weather data at Massena, New York, for five years (Lal and Shen 1990b).

For heat exchange from the snow surface to the atmosphere,

$$\phi_n = 25.21(T_s - T_a) \quad (s = 128.1, r = 0.72) \quad (187)$$

$$\phi_n = 83.65 + 18.64(T_s - T_a) \quad (s = 108.8, r = 0.72) \quad (188)$$

$$\phi_T = 23.36(T_s - T_a) \quad (s = 75.7, r = 0.84) \quad (189)$$

$$\phi_T = 54.92 + 19.64(T_s - T_a) \quad (s = 69.4, r = 0.84). \quad (190)$$

For heat exchange from the ice surface to the atmosphere,

$$\phi_n = 17.38(T_s - T_a) \quad (s = 77.9, r = 0.79) \quad (191)$$

$$\phi_n = 64.41 + 12.32(T_s - T_a) \quad (s = 58.0, r = 0.79) \quad (192)$$

$$\phi_T = 13.31(T_s - T_a) \quad (s = 44.0, r = 0.85) \quad (193)$$

$$\phi_T = 32.55 + 12.19(T_s - T_a) \quad (s = 36.9, r = 0.87). \quad (194)$$

Statistical indicators obtained from linear regression are the standard error estimate  $s$  and the coefficient of correlation  $r$ .

Turbulent heat transfer from the flowing river water to the ice cover has been studied by Ashton (1973), Calkins (1984) and Marsh and Prowse (1987), among others. In the present study the formulation by Ashton (1973) is used. In this formulation the turbulent heat transfer is expressed as

$$\phi_{wi} = h_{wi}(T_w - T_m) \quad (195)$$

where  $\phi_{wi}$  = heat exchange ( $\text{W m}^{-2}$ )  
 $T_w$  = water temperature (°C)  
 $T_m$  = freezing point of water.

The coefficient  $h_{wi}$  can be evaluated by the formula (Ashton 1973)

$$h_{wi} = C_{wi} U_w^{0.8} D_w^{-0.2} \quad (196)$$

where  $C_{wi} = 1662 (\text{W s}^{0.8} \text{m}^{-2.6} \text{°C}^{-1})$   
 $D_w$  = flow depth (m)  
 $U_w$  = average flow velocity ( $\text{m s}^{-1}$ ).

The coefficient  $C_{wi}$  may be increased by up to 50% when relief features form on the underside of the cover.

### Simulation of thermal growth and decay

The equation governing the one-dimensional temperature distribution in an ice cover is given by (Shen and Chiang 1984)

$$\rho_i C_i \frac{\partial T}{\partial t} = \frac{\partial}{\partial z} \left( k_i \frac{\partial T}{\partial z} \right) + \phi_v(z, t) \quad (197)$$

where  $t$  = time

$z$  = distance measured downward from the top surface

$T$  = temperature in the cover

$k_i$  = conductivity of ice

$\rho_i$  = density of ice

$C_i$  = specific heat of ice

$\phi_v(z, t)$  = rate of internal heating of the ice cover due to the absorption of penetrated short-wave radiation.

The boundary condition at the top surface is

$$\rho_i L \frac{dh}{dt} = \phi_T - k_i \frac{\partial T}{\partial z} \quad \text{at } z = 0 \quad (198)$$

where  $L$  = latent heat of fusion of ice

$h$  = thickness of the ice cover

$\phi_T$  = net heat loss at the air/ice interface.

Any water on the top surface of the ice cover is assumed to drain through the cover. In this study  $dh/dt = 0$  when the cover surface temperature is below freezing. The boundary condition at the bottom boundary is

$$\rho_i L \frac{dh}{dt} = -\phi_{wi} + k_i \frac{\partial T}{\partial z} \quad \text{at } z = h \quad (199)$$

where  $\phi_{wi}$  is the net heat flux from the water to the ice cover.

#### *Rate of growth in the absence of frazil accumulation*

Time-dependent ice growth and decay can be determined by assuming one-dimensional quasi-steady-state calculations at each time step. At steady state the solution to the governing eq 197 gives a linear temperature distribution if the rate of internal heating of the ice cover  $\phi_v(z, t)$  is neglected. The quasi-steady-state assumption has been shown to be acceptable for river ice covers because of the relatively small thickness (Greene 1981, Ashton 1982). At steady state the heat flux across different layers of the ice cover is the same if there is no water in the snow cover. As shown in Figure 21, the heat flux in each layer can be obtained using  $\phi = k(\partial T/\partial z)$ :

$$\phi_i = \frac{k_i}{h_i} (T_m - T_2) = \phi_s \quad (200)$$

$$\phi_s = \frac{k_w}{h_w} (T_2 - T_1) = \phi_a \quad (201)$$

$$\phi_a = \frac{k_s}{h_s} (T_1 - T_s) \quad (202)$$

where  $k$  = thermal conductivity

$h$  = thickness

$i, w$  and  $s$  = subscripts denoting black ice, white ice and snow, respectively.

The boundary condition at the ice/water interface is obtained using eq 199 and 200:



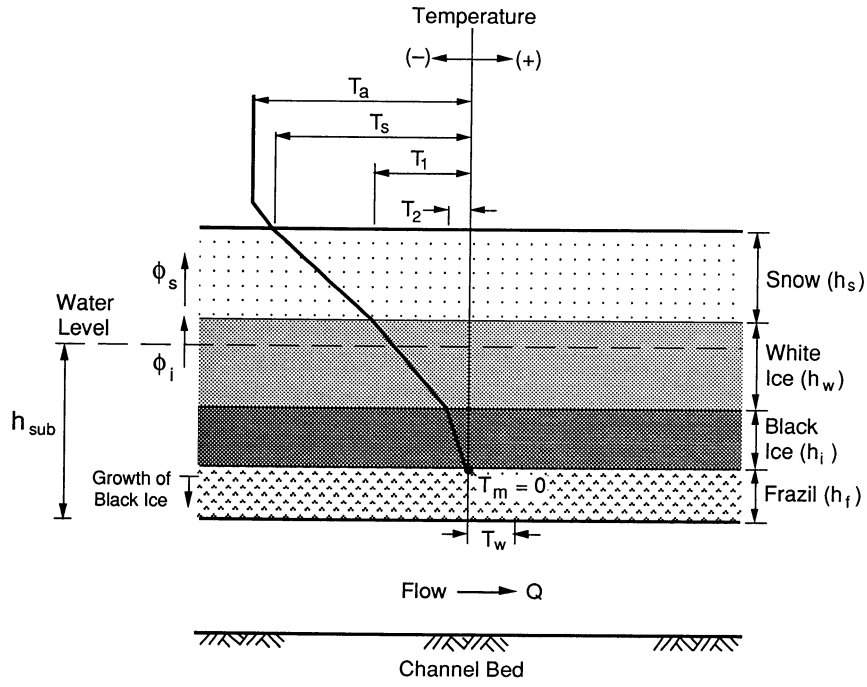


Figure 21. Thermal growth of an ice cover.

$$\phi_a - \phi_{wi} = \rho_i L \frac{dh_i}{dt} \quad (203)$$

where  $\phi_{wi}$  is the heat flux from water to ice. In the absence of melting, the boundary condition at the top surface is obtained using eq 198 and 202:

$$\phi_a = \phi_T \quad (204)$$

where  $\phi_T$  is the net heat exchange at the top surface. The rate of growth  $dh_i/dt$  given by eq 203 can be determined if  $\phi_a$  or  $\phi_T$  is known. The latter can be determined in terms of  $T_s$  using eq 184 or 186, and eq 195.

$$\rho_i L \frac{dh_i}{dt} = -\phi_R + \alpha + \beta(T_s - T_a) - h_{wi}(T_w - T_m). \quad (205)$$

$T_s$  can be determined explicitly when a linear heat exchange model is used. Linear models given by eq 184 or 186 can be used with eq 200, 201 and 202 to eliminate  $T_1$ ,  $T_2$  and  $\phi_a$  and obtain the following expression for  $T_s$ :

$$T_s = \frac{\left[ \left( T_a + \frac{\phi_R - \alpha}{\beta} \right) \left( \frac{h_s}{k_s} + \frac{h_w}{k_w} + \frac{h_i}{k_i} \right) + \frac{T_m}{\beta} \right]}{\frac{h_s}{k_s} + \frac{h_w}{k_w} + \frac{h_i}{k_i} + \frac{1}{\beta}}. \quad (206)$$

Equation 205 is valid only if  $T_s < T_m$ . When there is no frazil ice accumulation, black ice growth takes place on the underside of the ice cover at a rate given by this equation.

An alternative expression for the rate of growth can be obtained by substituting eq 206 into eq 205:

$$\frac{dh_i}{dt} = \frac{-\phi_R + \alpha + \beta(T_m - T_a)}{\rho_i L \beta \left( \frac{1}{\beta} + \frac{h_s}{k_s} + \frac{h_w}{k_w} + \frac{h_i}{k_i} \right)} - \frac{h_{wi}}{\rho_i L} (T_w - T_m). \quad (207)$$

Equation 207 is also valid only when  $T_s < T_m$ . Under steady-state conditions,  $T_s < T_m$  is synonymous with  $T_a < T_m$  except when  $\phi_R < \alpha$ . This shows that ice cover growth takes place on most occasions when the air temperature is below freezing. Equation 207 gives the steady-state governing equation for the rate of growth of black ice. Equation 207 is valid only for  $-\phi_R + \alpha + \beta (T_m - T_a) > 0$ .

The condition for ice cover growth starting from skim ice of nearly zero thickness can be determined by obtaining the limiting condition for  $dh_i/dt$  to be positive as  $h_i \rightarrow 0$ . Using eq 207, it can be shown that the latter condition is equivalent to

$$T_a < T_m - \frac{[\phi_R + h_{wi}(T_w - T_m)]}{\beta}. \quad (208)$$

This shows that skim ice growth can start in turbulent water only if the condition in eq 208 is satisfied.

It can also be shown from eq 207 that for a given set of weather conditions, there exists an equilibrium thickness  $h_{ie}$  that has zero growth and decay or  $dh_i/dt = 0$ :

$$h_{ie} = -k_i \left( \frac{1}{\beta} + \frac{h_w}{k_w} + \frac{h_s}{k_s} \right) + \frac{k_i}{\beta} \frac{[-\phi_R + \alpha + \beta (T_m - T_a)]}{h_{wi}(T_w - T_m)}. \quad (209)$$

Analytical expressions for ice thickness can be obtained by solving eq 207 for  $h$  as shown below. Neglecting bottom heat transfer, the rearranged equation can be integrated as shown below:

$$\int_{h_i^0}^{h_i} \left( \frac{1}{\beta} + \frac{h_s}{k_s} + \frac{h_w}{k_w} + \frac{h_i}{k_i} \right) dh_i = \frac{1}{\rho L \beta} S \quad (210)$$

where

$$S = \int_t^{t+\Delta t} [-\phi_R + \alpha + \beta (T_m - T_a)] dt \quad (211)$$

and  $h_i^0$  and  $h_i$  are the black ice thicknesses at times  $t$  and  $t+\Delta t$ , respectively. The solution of the quadratic equation obtained from eq 210 is

$$h_i = -\left( \frac{k_i}{\beta} + \frac{k_i h_s}{k_s} + \frac{k_i h_w}{k_w} \right) + \left[ \left( \frac{k_i}{\beta} + \frac{k_i h_s}{k_s} + \frac{k_i h_w}{k_w} + h_i^0 \right)^2 + \frac{2k_i S}{\rho_i L_i \beta} \right]^{1/2}. \quad (212)$$

The degree-day formula is a simplified form of eq 212 (Shen and Yapa 1985). This explicit expression for  $h_i$  is possible only when the heat transfer from the river flow is neglected. When heat transfer at the ice/water interface is included, eq 207 can be solved to obtain the following expression:

$$\Delta t = \frac{b}{d} (h_i - h_o) + \frac{ad - bc}{d^2} \ln \left( \frac{c + dh_i}{c + dh_i^0} \right) \quad (213)$$

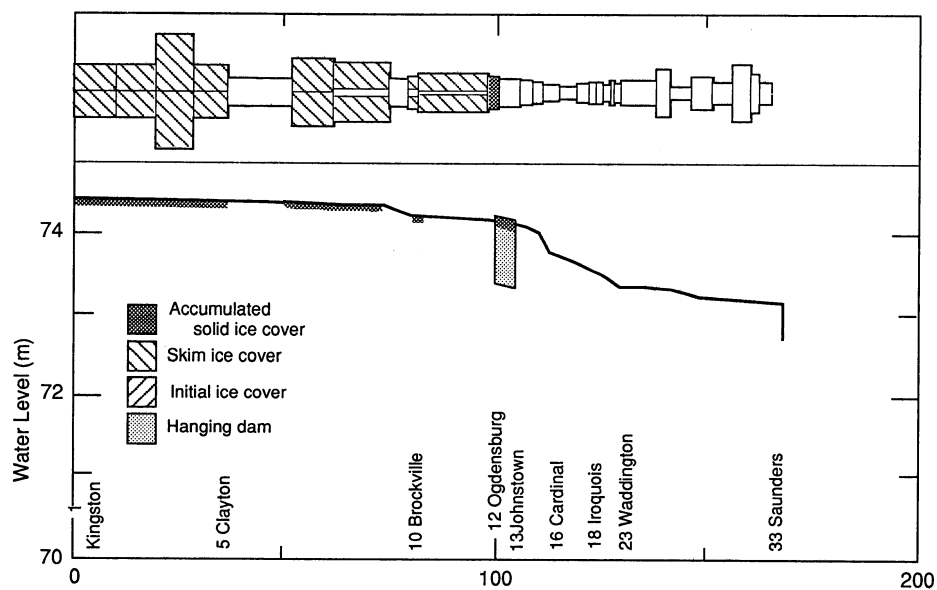
where

$$a = \rho_i L \left( 1 + \frac{\beta h_s}{k_s} + \frac{\beta h_w}{k_w} \right) \quad (214)$$

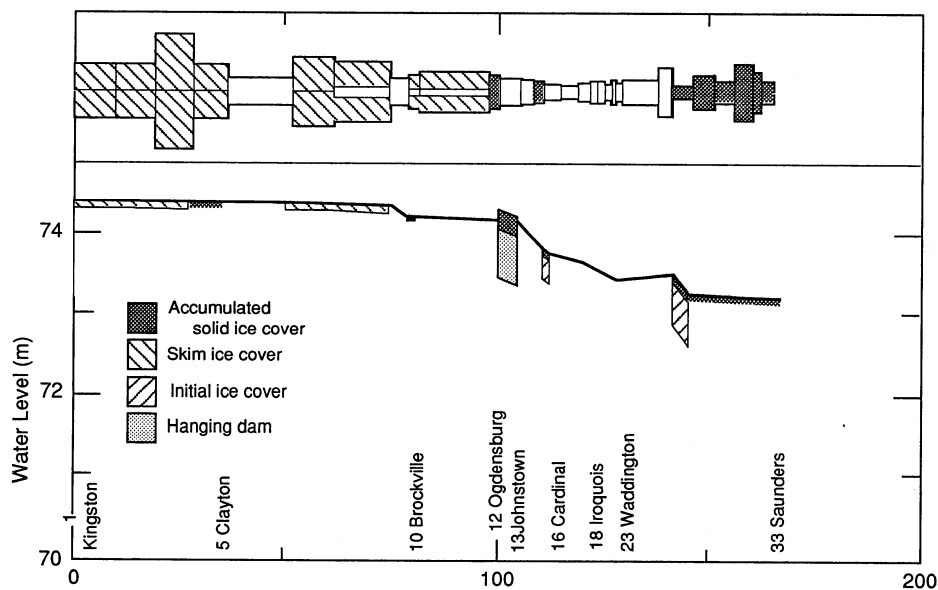
$$b = \frac{\rho_i L \beta}{k_i} \quad (215)$$

$$c = \phi_R + \alpha + \beta (T_m - T_a) - h_{wa} (T_w - T_m) \left( 1 + \frac{\beta h_s}{k_s} + \frac{\beta h_w}{k_w} \right) \quad (216)$$

b. 24 January 1980.



c. 26 January 1980.



d. 28 January 1980.

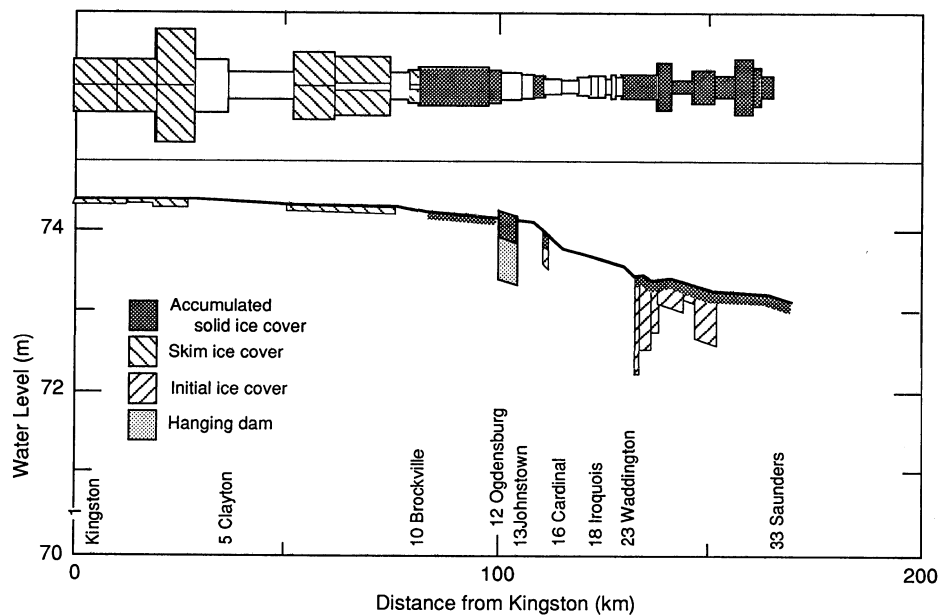


Figure 31 (cont'd).

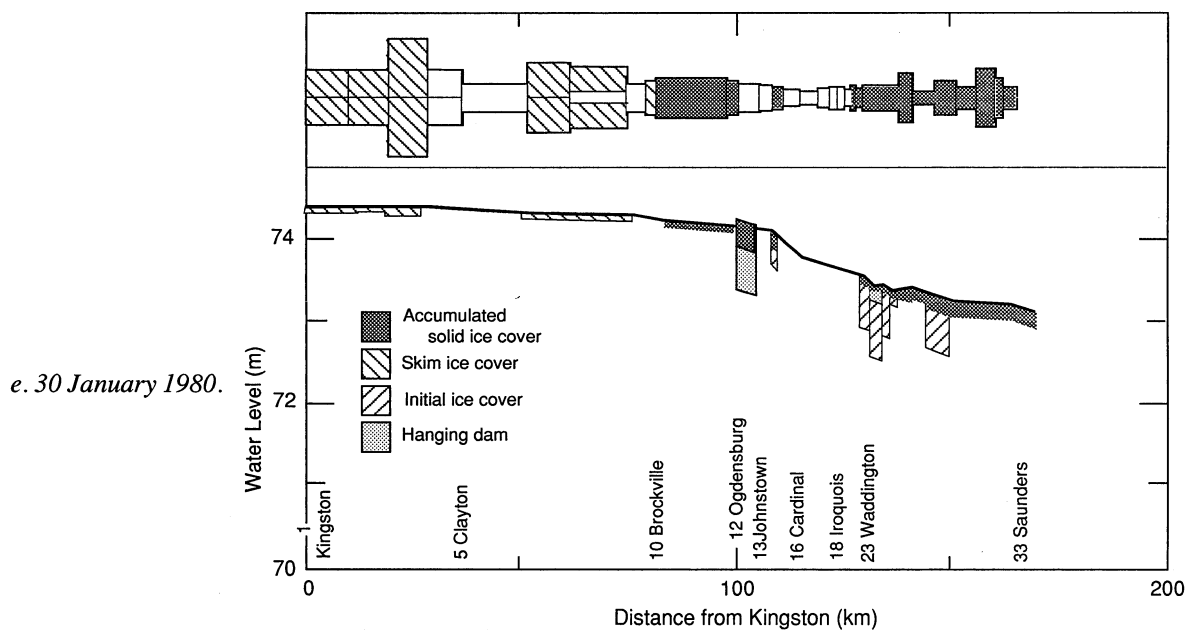


Figure 31 (cont'd). Plan view of schematized channel surface, channel bottom profile and water surface profile.

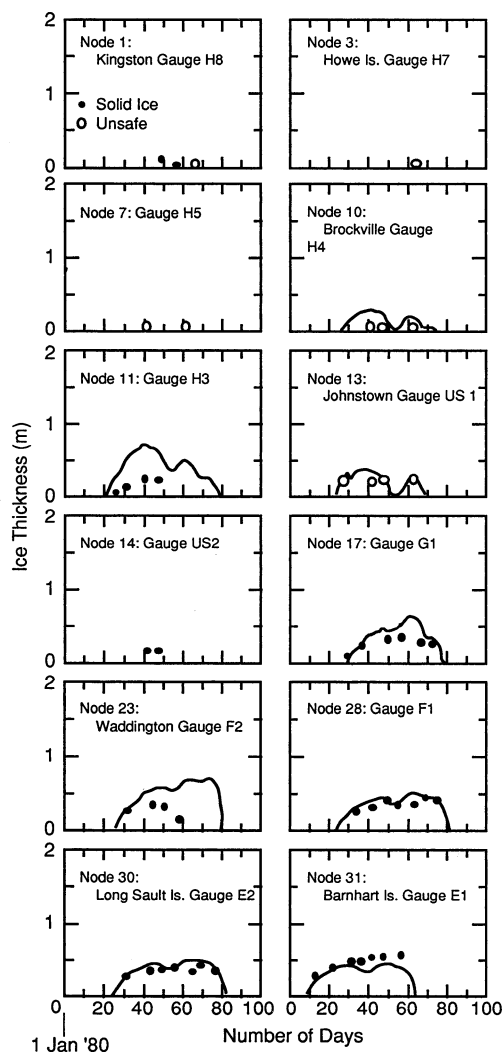


Figure 32. Observed and simulated ice cover thickness.

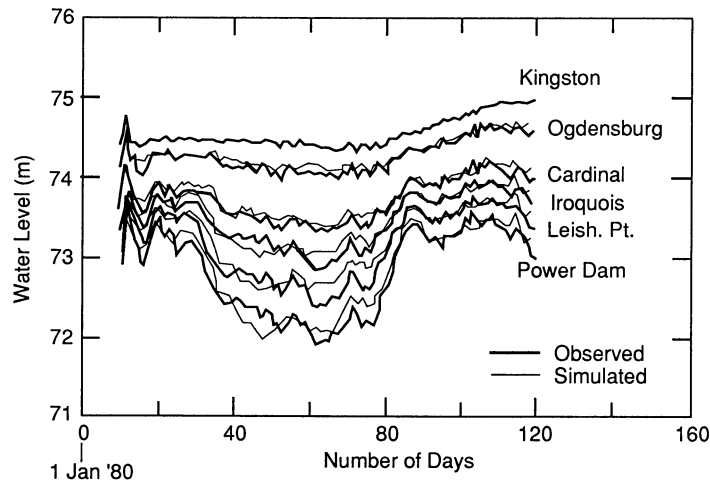


Figure 33. Observed and simulated water levels for the winter of 1980.

of simulating water level variations during both open water and ice-covered conditions. The error in the simulation of water levels is larger during the ice-covered period because of the cumulative effect of errors in the simulated ice cover area, the under-cover thickness and the resistance coefficients. Figure 34 shows the water level differences between gauging stations. Simulated and observed head losses agree well except in the Cardinal–Leishman Point region. Water level differences in this region can be due to inaccuracies in the simulation of the undercover ice accumulation.

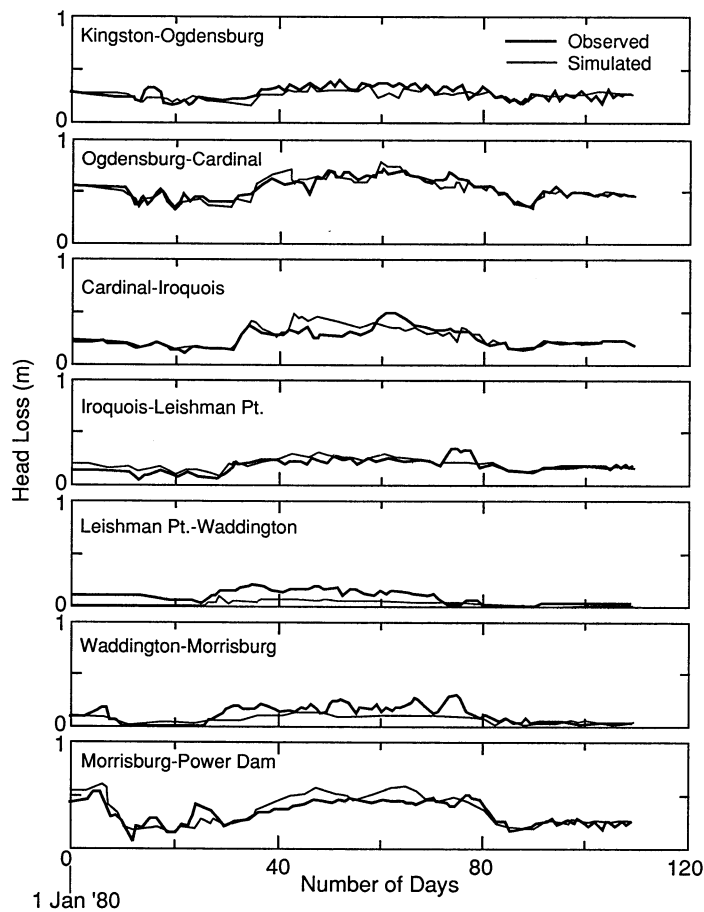


Figure 34. Variations of head losses.

Figure 35 shows the distribution of simulated water temperatures along the river between 17 January and 30 January 1980. Subzero values represent the existence of ice concentration.

### Error analysis

Figure 36 shows the relationship between the observed and simulated daily water levels. Graphical comparisons of observed and simulated average daily data provide qualitative information on the accuracy of the simulation. The statistical parameters developed in Appendix A can be used to quantify errors of the simulation. Table 6 gives the definitions of these parameters, which are useful in interpreting the results.

Table 7 compares all the statistical indicators at all the water level observation stations. Values of the standard deviation of observed water levels  $\sigma_Y$  show that water levels fluctuate more at locations closer to the power dam at the downstream end. This is because of variations in the power demand and the relatively stable level at the outlet of Lake Ontario. The error terms  $e_a$ ,  $e_b$ ,  $e_p$ ,  $e_n$  and  $e_s$  are also larger at the downstream end. However, values of the nondimensionalized parameters  $V$  and  $\rho$  remain essentially constant along the river, showing that the model is capable of simulating water level fluctuations along the entire river effectively.

In Table 7,  $e_s$  and  $e_a$  values are on the order of 0.1 m, indicating that the error involved in simulating water levels is relatively small. Correlation coefficients between the observed and simulated water levels are on the order of 0.97. Values of the parameter  $V$  are on the order of 0.9, indicating that the model is capable of simulating variations of water levels accurately. Values of  $b$  and  $\sigma_y$  are approximately equal to 1.0 and  $\sigma_y$ , respectively.

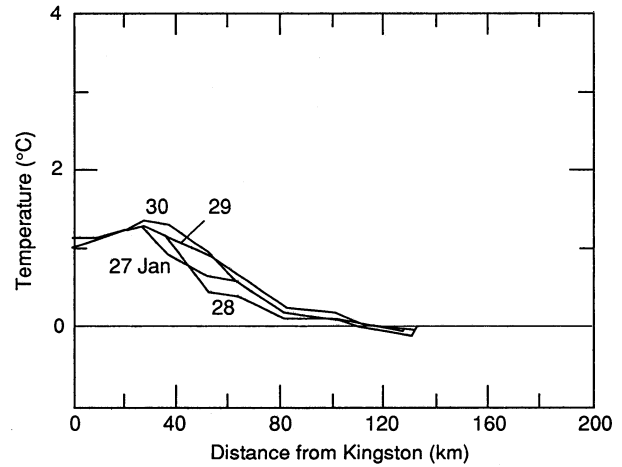
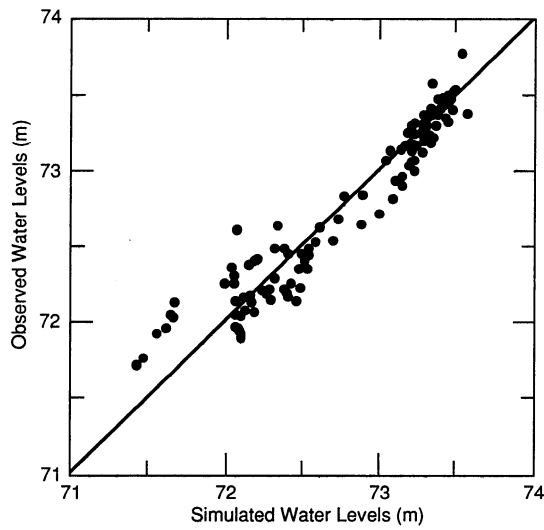


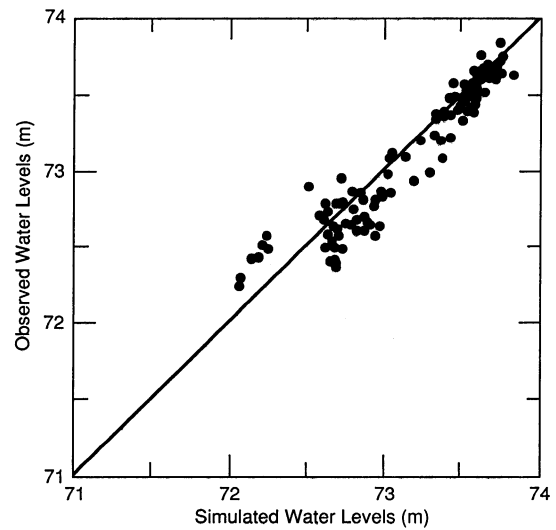
Figure 35. Water temperature profiles along the river from 27 January to 30 January 1980.

**Table 6. Definitions of statistical parameters for the error analysis.**

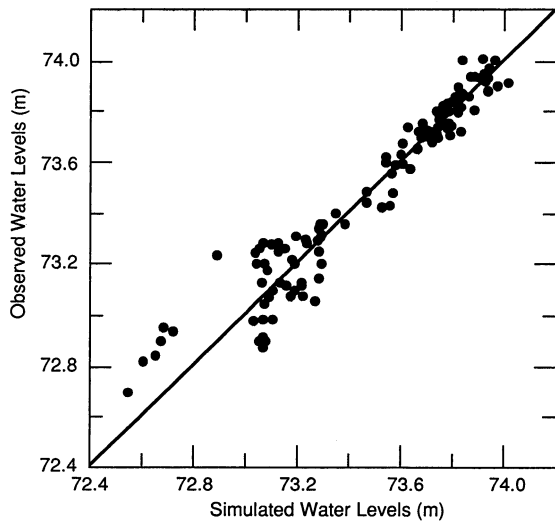
Variable	Definition
$n$	Number of time steps ( $i = 1, 2, \dots, n$ ) or number of observations
$Y_i$	Observed value at time step $i$
$y_i$	Simulated value at time step $i$
$\bar{Y}$	Average of observed values, $\frac{\sum_{i=1}^n Y_i}{n}$
$\sigma_Y$	Standard deviation of observed values, $\sqrt{\frac{\sum_{i=1}^n (Y_i - \bar{Y})^2}{n}}$
$\bar{y}$	Averaged of simulated values, $\sum_{i=1}^n y_i$
$\sigma_y$	Standard deviation of simulated values, $\sqrt{\frac{\sum_{i=1}^n (y_i - \bar{y})^2}{n}}$
$Y_{\min}$	Minimum observed value
$Y_{\max}$	Maximum observed value
$y_{\min}$	Minimum simulated value
$y_{\max}$	Maximum simulated value
$e_b$	Bias, $\sum_{i=1}^n \frac{y_i - Y_i}{n}$
$e_a$	Average absolute error, $\frac{\sum_{i=1}^n  y_i - Y_i }{n}$
$e_p$	Maximum positive error, $\text{Max}(y_i - Y_i)$ for all $i$
$e_n$	Maximum negative error, $\text{Max}(Y_i - y_i)$ for all $i$
$e_s$	Standard error estimate, $\sqrt{\frac{\sum_{i=1}^n (y_i - Y_i)^2}{n}}$
$r$	Correlation coefficient between $Y_i$ and $y_i$ for all $i$
$V$	$1 - \frac{e_s^2}{\sigma_Y^2}$
$a, b$	Constants in a $Y_i = a + by_i$ type model for all $i$



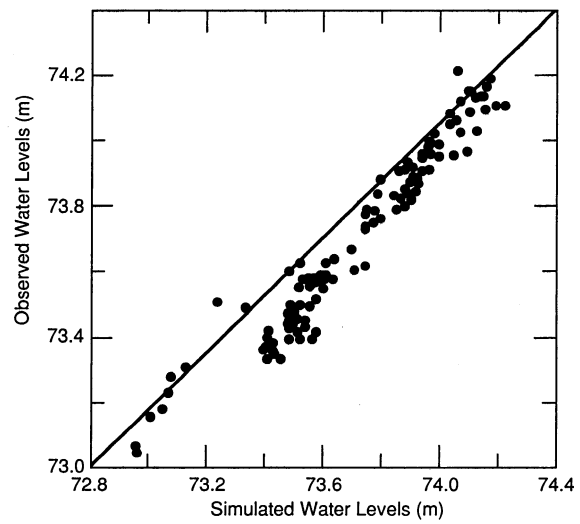
*a. Power Dam.*



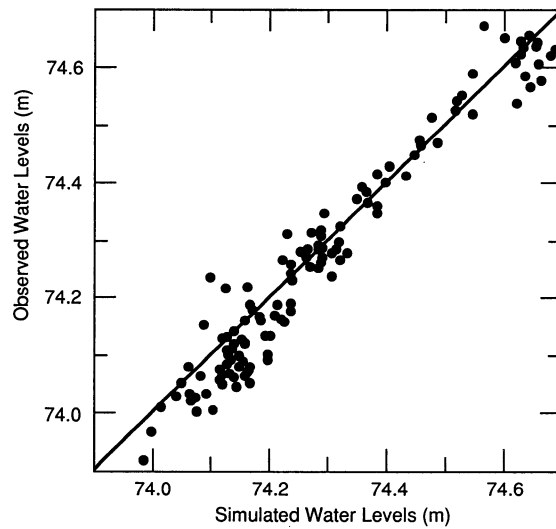
*b. Morrisburg.*



*c. Iroquois Dam.*



*d. Cardinal.*



*e. Ogdensburg.*

Figure 36. Simulated and observed water levels, 1 January to 30 April 1980.

**Table 7. Statistical parameters for the water level simulations.**

Variable	Power Dam	Morrisburg	Iroquois	Cardinal	Ogdensburg
$n$	120	120	120	120	120
$\bar{Y}$ (m)	72.793	73.131	73.453	73.713	74.268
$\sigma_Y$ (m)	0.5591	0.4649	0.3540	0.2859	0.1977
$\bar{y}$ (m)	72.812	73.147	73.437	73.7114	74.278
$\sigma_y$ (m)	0.5961	0.4629	0.3762	0.2981	0.1875
$Y_{\min}$ (m)	71.726	72.256	72.689	73.042	73.920
$Y_{\max}$ (m)	73.798	73.868	73.996	74.219	74.673
$y_{\min}$ (m)	71.428	72.046	72.524	72.9503	73.977
$y_{\max}$ (m)	73.574	73.808	74.018	74.218	74.685
$e_b$ (m)	0.0107	-0.0016	0.0175	0.0016	-0.0104
$e_a$ (m)	0.1214	0.1091	0.0759	0.0526	0.0368
$e_p$ (m)	0.3094	0.3434	0.2085	0.1606	0.1075
$e_n$ (m)	0.5285	0.4120	0.3474	0.2727	0.1416
$e_s$ (m)	0.1623	0.1425	0.1035	0.0710	0.0463
$r$	0.9624	0.9531	0.9624	0.9710	0.9736
$V$	0.9150	0.9053	0.9137	0.9377	0.9446
$a$	7.0969	3.1215	6.9595	5.0654	-1.9690
$b$	0.9026	0.9571	0.9054	0.9313	1.0263

**Table 8. Statistical parameters for simulated water levels at the power dam during ice-covered and open water conditions.**

Variable	Combined	Ice-covered	Open water
$n$	120	62	36
$\bar{Y}$	72.792	72.470	73.352
$\sigma_Y$	0.5591	0.4029	0.1326
$\bar{y}$	72.701	72.5084	73.3504
$\sigma_y$	0.6504	0.4336	0.0979
$Y_{\min}$	72.726	71.924	73.015
$Y_{\max}$	73.798	73.368	73.579
$y_{\min}$	71.428	72.000	73.1741
$y_{\max}$	73.576	73.339	73.5741
$e_b$	0.0913	-0.0383	0.0018
$e_a$	0.11382	0.125	0.0665
$e_p$	0.2193	0.3094	0.2171
$e_n$	0.5252	0.3225	0.2307
$e_s$	0.1954	0.1499	0.0850
$r$	0.9701	0.9416	0.7604
$V$	0.8768	0.8593	0.5776
$a$	12.1631	9.0234	-2.1746
$b$	0.8339	0.8750	1.0297

**Table 9. Statistical parameters for simulated water temperatures.**

Variable	Clayton	Waddington	Power Dam
$n$	55	55	55
$\bar{Y}$ (°C)	3.1465	2.1908	2.1994
$\sigma_Y$ (°C)	1.8148	1.9107	1.9879
$\bar{y}$ (°C)	2.9982	2.0108	1.8915
$\sigma_y$ (°C)	1.5956	1.6550	1.6521
$Y_{\min}$ (°C)	0.5000	0.0000	1.1100
$Y_{\max}$ (°C)	6.5500	6.0500	0.3278
$y_{\min}$ (°C)	0.8614	0.0000	0.0000
$y_{\max}$ (°C)	5.8814	5.2456	4.9903
$e_b$ (°C)	0.1484	0.1799	0.3078
$e_a$ (°C)	0.4176	0.5817	0.6113
$e_p$ (°C)	0.8438	1.0181	0.9239
$e_n$ (°C)	1.0723	2.1914	2.6079
$e_s$ (°C)	0.5119	0.7753	0.8792
$r$	0.9661	0.9187	0.9120
$V$	0.9189	0.8322	0.3007
$a$	-0.1478	0.0579	0.1236
$b$	1.0987	1.0606	1.0974

This indicates that simulated variations are close to the observed variations. Values of  $b < 1$  and  $\sigma_y < \sigma_Y$  indicate that simulated variations are slightly larger than the observed variations.

Table 8 shows a comparison of water level predictabilities during open water and ice-covered conditions. As expected, this table shows that simulations during ice-covered conditions are not as good as those during open water conditions.

Table 9 summarizes the error indicators for water temperature simulations. Values of  $e_s$  and  $e_a$  show that the error involved in the simulation is about 0.5°C. Because water temperature stations are often located near the bank and because of the accuracy of the temperature data, this error is acceptable. Values of  $r$  and  $V$  suggest that the model can simulate variations of water temperature satisfactorily.

The error indicators could not be developed for variables such as ice cover area and thickness because of the limited availability of observed data. Errors in simulations as measured by the error variance  $e_s^2$  can be due to a number of sources. The component due to observational error can only be reduced through accurate instrumentation and better observation station locations. One of the major error sources is the water temperature data. The



effect of stratification in the upstream region, which cannot be modeled accurately due to lack of data, can also have an important effect on the ice condition. A second component of error variance due to bias  $e_s^2$  can usually be eliminated by proper calibration. The part of the error due to the one-dimensional approximation is also unavoidable. A significant part of the error is due to shortcomings in the existing theories on river ice processes. These can be improved only when better theories become available. These shortcomings cover almost all of the river ice processes, including formation of frazil and skim ice, ice jam formation, shore ice accumulation, hanging dam formations or undercover erosion and deposition, and ice cover resistance coefficient.

### Summary and conclusion

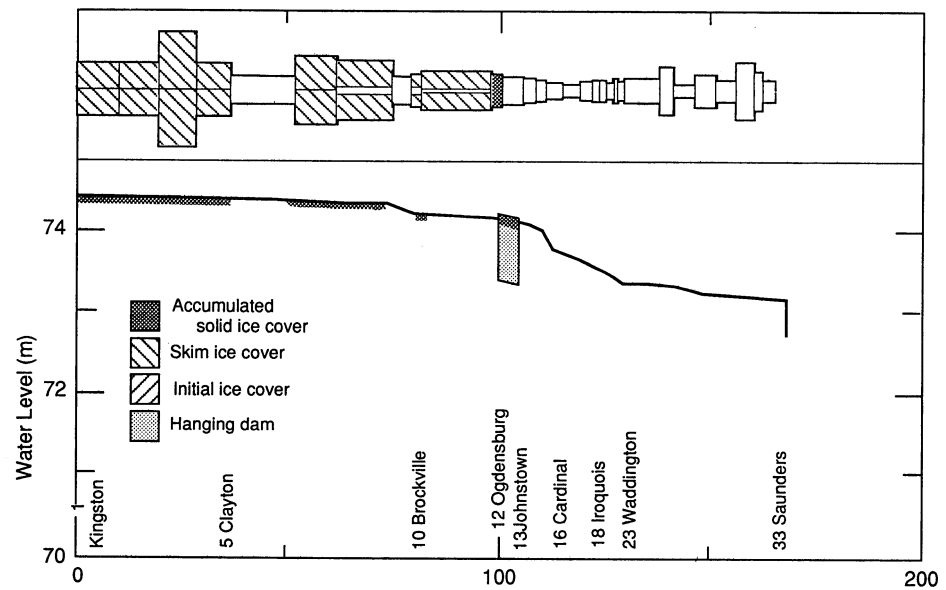
In this study a one-dimensional model is developed for simulating river ice processes. The model is capable of simulating time-dependent conditions of the river hydraulics, water temperature and ice concentration, formation of an ice cover, undercover accumulation, thermal growth and decay of the ice cover, and mechanical stability of the cover. In the river hydraulics component, the flow condition is determined by an implicit finite-difference solution of one-dimensional unsteady flow equations. In the thermal component, distributions of water temperature and ice concentration are determined by a Lagrangian–Eulerian solution scheme for equations of transport of thermal energy and ice. A two-layer formulation is introduced to model the ice transport. In this formulation the total ice discharge is considered to consist of the surface ice discharge and the discharge of suspended ice distributed over the depth of the flow. The effect of surface ice on ice production as well as formations of skim ice and border ice are included. The dynamic formation and stability of the ice cover is formulated according to existing equilibrium ice jam theories with due consideration to the interaction between the ice cover and the flow. The undercover ice accumulation is formulated according to the critical velocity criterion. The growth and decay of the ice cover is simulated using a finite-difference formulation applicable to composite ice covers consisting of snow, ice and frazil layers. The model has been applied to the upper St. Lawrence River and the Ohio River system with good results. With the generic nature of the model structure, the model can be applied to other rivers. In view of the limited current understanding on river ice processes, further improvements of the model should be made when improved theoretical formulations become available. The model can assist the development of new formulations by analyzing field data in a comprehensive manner. Further improvements on modeling techniques are also possible by including some important two-dimensional aspects of ice processes.

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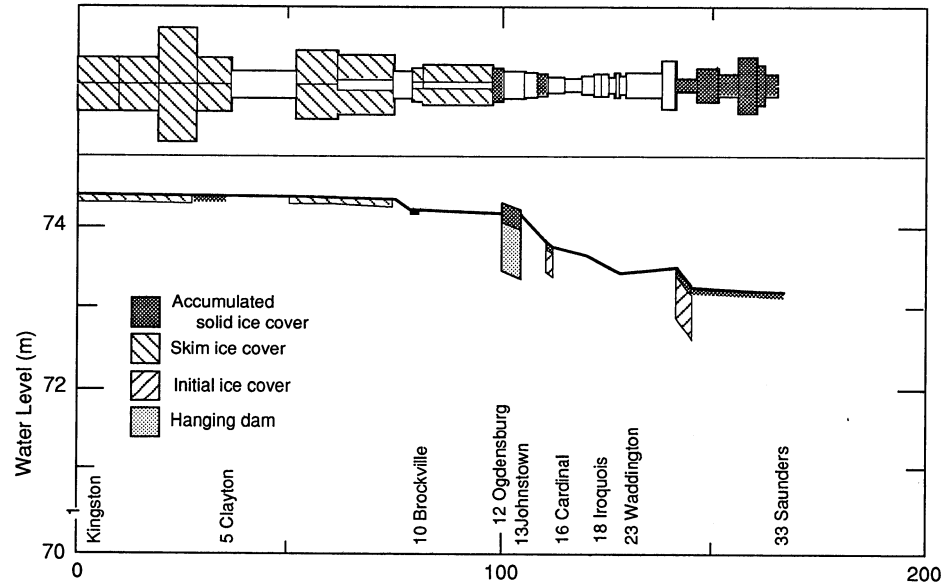
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b. 24 January 1980.



c. 26 January 1980.



d. 28 January 1980.

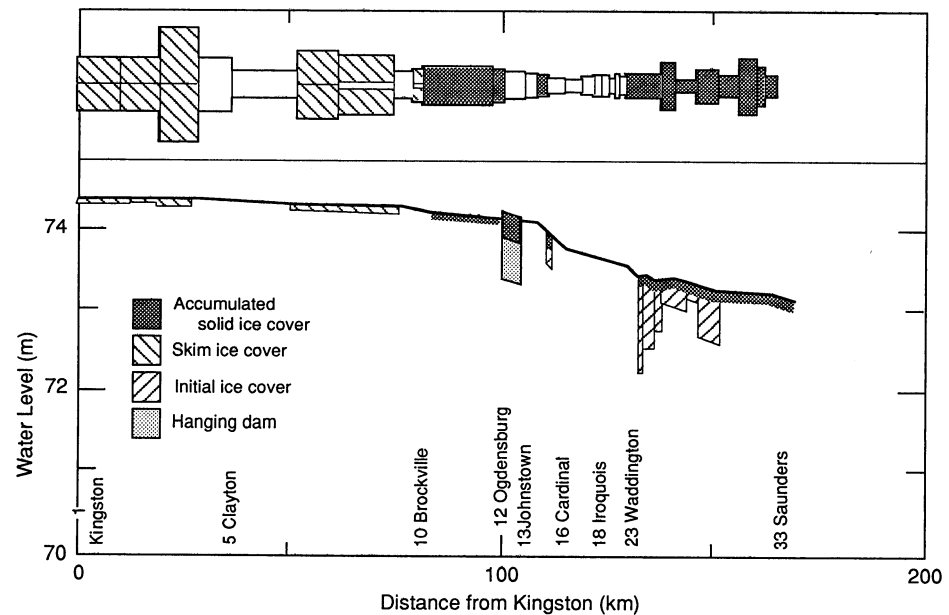


Figure 31 (cont'd).

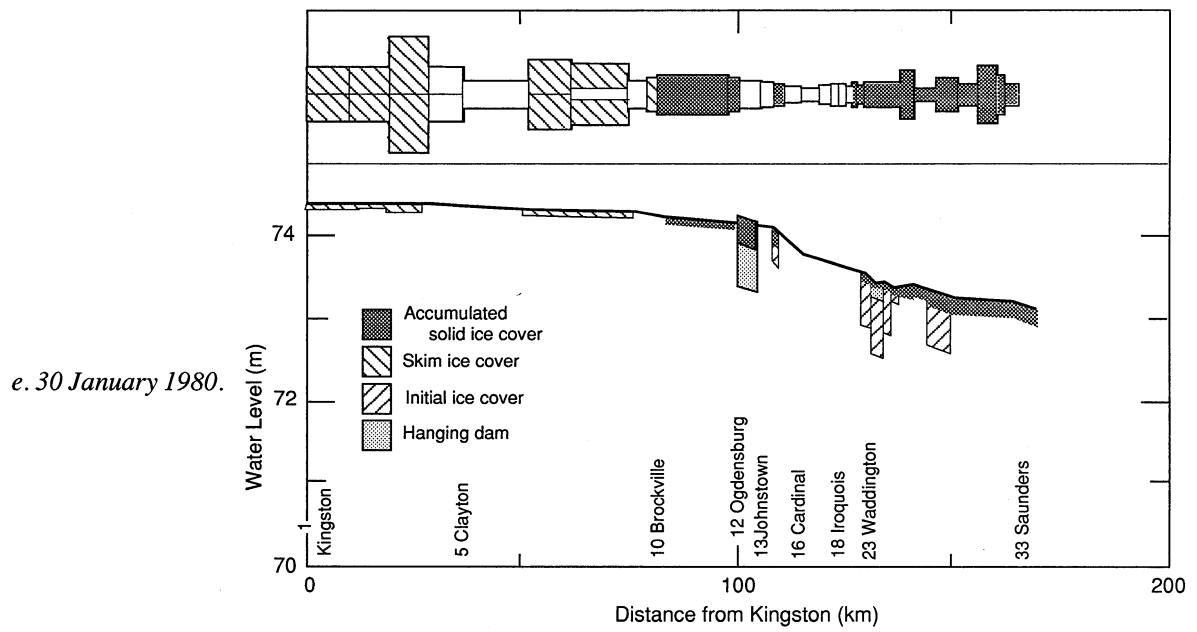


Figure 31 (cont'd). Plan view of schematized channel surface, channel bottom profile and water surface profile.

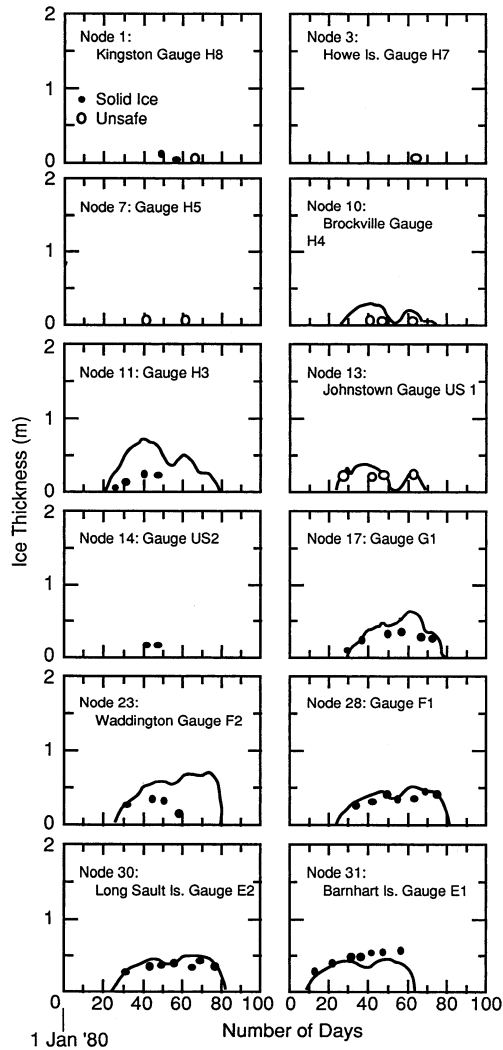


Figure 32. Observed and simulated ice cover thickness.

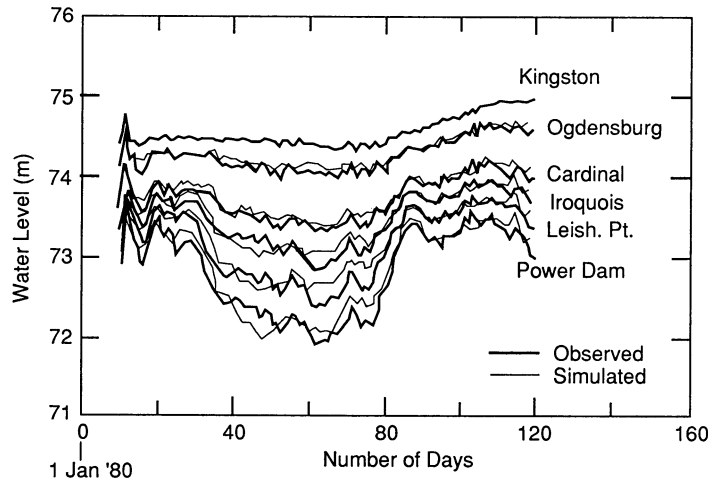


Figure 33. Observed and simulated water levels for the winter of 1980.

of simulating water level variations during both open water and ice-covered conditions. The error in the simulation of water levels is larger during the ice-covered period because of the cumulative effect of errors in the simulated ice cover area, the under-cover thickness and the resistance coefficients. Figure 34 shows the water level differences between gauging stations. Simulated and observed head losses agree well except in the Cardinal–Leishman Point region. Water level differences in this region can be due to inaccuracies in the simulation of the undercover ice accumulation.

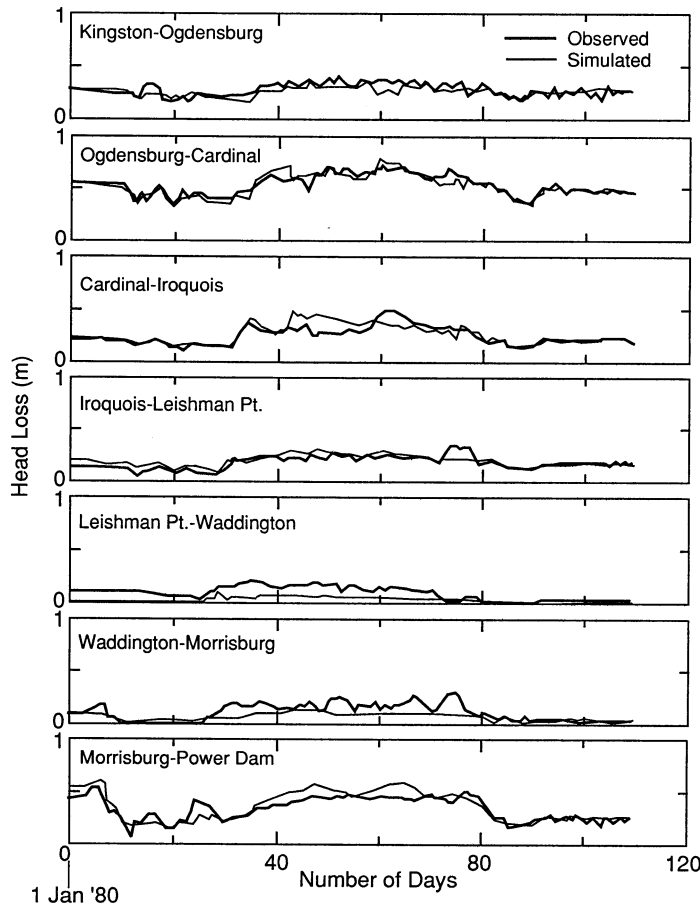


Figure 34. Variations of head losses.

Figure 35 shows the distribution of simulated water temperatures along the river between 17 January and 30 January 1980. Subzero values represent the existence of ice concentration.

### Error analysis

Figure 36 shows the relationship between the observed and simulated daily water levels. Graphical comparisons of observed and simulated average daily data provide qualitative information on the accuracy of the simulation. The statistical parameters developed in Appendix A can be used to quantify errors of the simulation. Table 6 gives the definitions of these parameters, which are useful in interpreting the results.

Table 7 compares all the statistical indicators at all the water level observation stations. Values of the standard deviation of observed water levels  $\sigma_Y$  show that water levels fluctuate more at locations closer to the power dam at the downstream end. This is because of variations in the power demand and the relatively stable level at the outlet of Lake Ontario. The error terms  $e_a$ ,  $e_b$ ,  $e_p$ ,  $e_n$  and  $e_s$  are also larger at the downstream end. However, values of the nondimensionalized parameters  $V$  and  $\rho$  remain essentially constant along the river, showing that the model is capable of simulating water level fluctuations along the entire river effectively.

In Table 7,  $e_s$  and  $e_a$  values are on the order of 0.1 m, indicating that the error involved in simulating water levels is relatively small. Correlation coefficients between the observed and simulated water levels are on the order of 0.97. Values of the parameter  $V$  are on the order of 0.9, indicating that the model is capable of simulating variations of water levels accurately. Values of  $b$  and  $\sigma_y$  are approximately equal to 1.0 and  $\sigma_y$ , respectively.

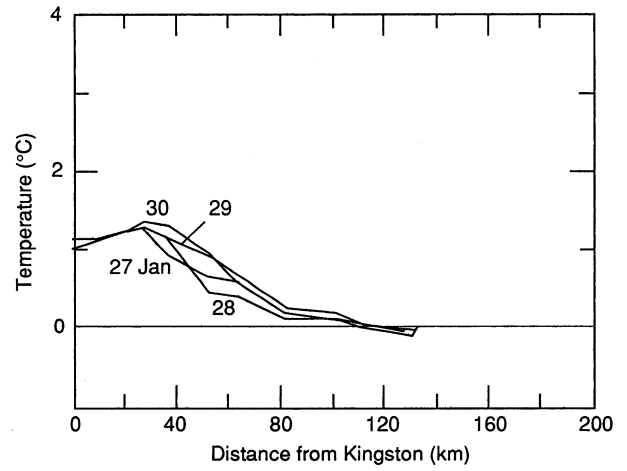
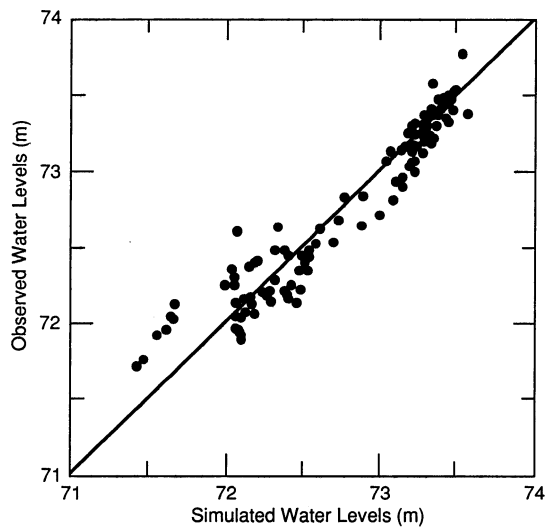


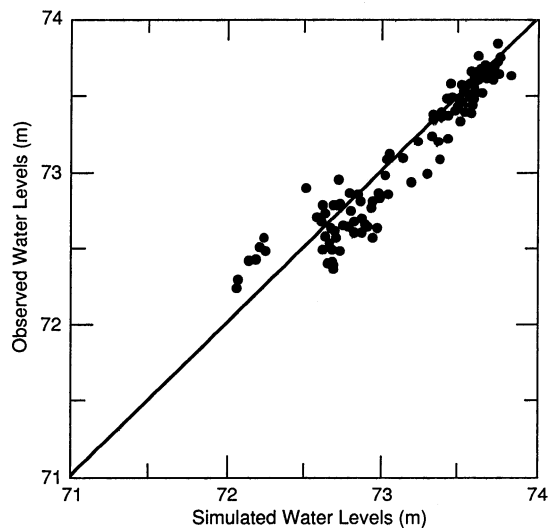
Figure 35. Water temperature profiles along the river from 27 January to 30 January 1980.

**Table 6. Definitions of statistical parameters for the error analysis.**

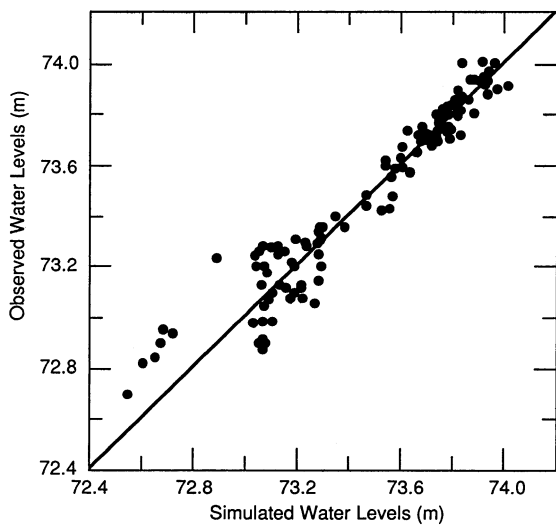
Variable	Definition
$n$	Number of time steps ( $i = 1, 2, \dots, n$ ) or number of observations
$Y_i$	Observed value at time step $i$
$y_i$	Simulated value at time step $i$
$\bar{Y}$	Average of observed values, $\frac{\sum_{i=1}^n Y_i}{n}$
$\sigma_Y$	Standard deviation of observed values, $\sqrt{\frac{\sum_{i=1}^n (y_i - \bar{y})^2}{n}}$
$\bar{y}$	Averaged of simulated values, $\frac{\sum_{i=1}^n y_i}{n}$
$\sigma_y$	Standard deviation of simulated values, $\sqrt{\frac{\sum_{i=1}^n (Y_i - \bar{Y})^2}{n}}$
$Y_{\min}$	Minimum observed value
$Y_{\max}$	Maximum observed value
$y_{\min}$	Minimum simulated value
$y_{\max}$	Maximum simulated value
$e_b$	Bias, $\sum_{i=1}^n \frac{y_i - Y_i}{n}$
$e_a$	Average absolute error, $\frac{\sum_{i=1}^n  y_i - Y_i }{n}$
$e_p$	Maximum positive error, $\text{Max}(y_i - Y_i)$ for all $i$
$e_n$	Maximum negative error, $\text{Max}(Y_i - y_i)$ for all $i$
$e_s$	Standard error estimate, $\sqrt{\frac{\sum_{i=1}^n (y_i - Y_i)^2}{n}}$
$r$	Correlation coefficient between $Y_i$ and $y_i$ for all $i$
$V$	$1 - \frac{e_s^2}{\sigma_Y}$
$a, b$	Constants in a $Y_i = a + by_i$ type model for all $i$



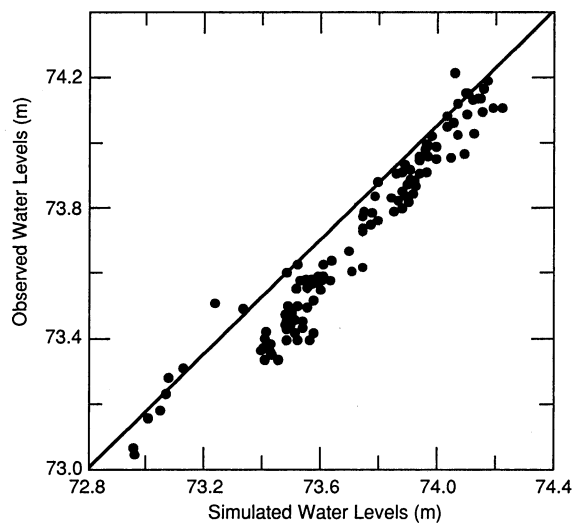
*a. Power Dam.*



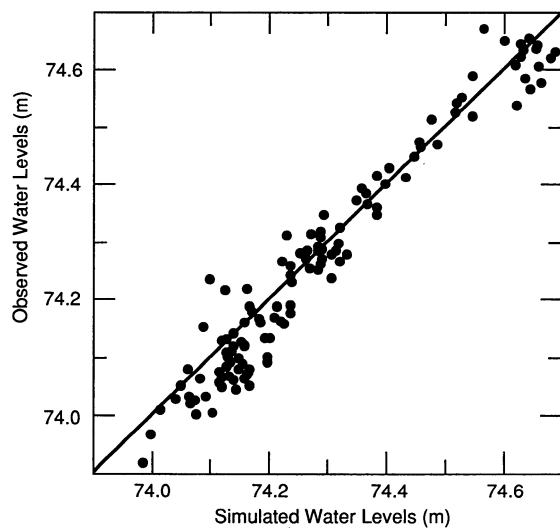
*b. Morrisburg.*



*c. Iroquois Dam.*



*d. Cardinal.*



*e. Ogdensburg.*

Figure 36. Simulated and observed water levels, 1 January to 30 April 1980.

**Table 7. Statistical parameters for the water level simulations.**

<i>Variable</i>	<i>Power Dam</i>	<i>Morrisburg</i>	<i>Iroquois</i>	<i>Cardinal</i>	<i>Ogdensburg</i>
<i>n</i>	120	120	120	120	120
$\bar{Y}$ (m)	72.793	73.131	73.453	73.713	74.268
$\sigma_Y$ (m)	0.5591	0.4649	0.3540	0.2859	0.1977
$\bar{y}$ (m)	72.812	73.147	73.437	73.7114	74.278
$\sigma_y$ (m)	0.5961	0.4629	0.3762	0.2981	0.1875
$Y_{\min}$ (m)	71.726	72.256	72.689	73.042	73.920
$Y_{\max}$ (m)	73.798	73.868	73.996	74.219	74.673
$y_{\min}$ (m)	71.428	72.046	72.524	72.9503	73.977
$y_{\max}$ (m)	73.574	73.808	74.018	74.218	74.685
$e_b$ (m)	0.0107	-0.0016	0.0175	0.0016	-0.0104
$e_a$ (m)	0.1214	0.1091	0.0759	0.0526	0.0368
$e_p$ (m)	0.3094	0.3434	0.2085	0.1606	0.1075
$e_n$ (m)	0.5285	0.4120	0.3474	0.2727	0.1416
$e_s$ (m)	0.1623	0.1425	0.1035	0.0710	0.0463
$r$	0.9624	0.9531	0.9624	0.9710	0.9736
$V$	0.9150	0.9053	0.9137	0.9377	0.9446
$a$	7.0969	3.1215	6.9595	5.0654	-1.9690
$b$	0.9026	0.9571	0.9054	0.9313	1.0263

**Table 8. Statistical parameters for simulated water levels at the power dam during ice-covered and open water conditions.**

<i>Variable</i>	<i>Combined</i>	<i>Ice-covered</i>	<i>Open water</i>
<i>n</i>	120	62	36
$\bar{Y}$	72.792	72.470	73.352
$\sigma_Y$	0.5591	0.4029	0.1326
$\bar{y}$	72.701	72.5084	73.3504
$\sigma_y$	0.6504	0.4336	0.0979
$Y_{\min}$	72.726	71.924	73.015
$Y_{\max}$	73.798	73.368	73.579
$y_{\min}$	71.428	72.000	73.1741
$y_{\max}$	73.576	73.339	73.5741
$e_b$	0.0913	-0.0383	0.0018
$e_a$	0.11382	0.125	0.0665
$e_p$	0.2193	0.3094	0.2171
$e_n$	0.5252	0.3225	0.2307
$e_s$	0.1954	0.1499	0.0850
$r$	0.9701	0.9416	0.7604
$V$	0.8768	0.8593	0.5776
$a$	12.1631	9.0234	-2.1746
$b$	0.8339	0.8750	1.0297

**Table 9. Statistical parameters for simulated water temperatures.**

<i>Variable</i>	<i>Clayton</i>	<i>Waddington</i>	<i>Power Dam</i>
<i>n</i>	55	55	55
$\bar{Y}$ (°C)	3.1465	2.1908	2.1994
$\sigma_Y$ (°C)	1.8148	1.9107	1.9879
$\bar{y}$ (°C)	2.9982	2.0108	1.8915
$\sigma_y$ (°C)	1.5956	1.6550	1.6521
$Y_{\min}$ (°C)	0.5000	0.0000	1.1100
$Y_{\max}$ (°C)	6.5500	6.0500	0.3278
$y_{\min}$ (°C)	0.8614	0.0000	0.0000
$y_{\max}$ (°C)	5.8814	5.2456	4.9903
$e_b$ (°C)	0.1484	0.1799	0.3078
$e_a$ (°C)	0.4176	0.5817	0.6113
$e_p$ (°C)	0.8438	1.0181	0.9239
$e_n$ (°C)	1.0723	2.1914	2.6079
$e_s$ (°C)	0.5119	0.7753	0.8792
$r$	0.9661	0.9187	0.9120
$V$	0.9189	0.8322	0.3007
$a$	-0.1478	0.0579	0.1236
$b$	1.0987	1.0606	1.0974

This indicates that simulated variations are close to the observed variations. Values of  $b < 1$  and  $\sigma_y < \sigma_Y$  indicate that simulated variations are slightly larger than the observed variations.

Table 8 shows a comparison of water level predictabilities during open water and ice-covered conditions. As expected, this table shows that simulations during ice-covered conditions are not as good as those during open water conditions.

Table 9 summarizes the error indicators for water temperature simulations. Values of  $e_s$  and  $e_a$  show that the error involved in the simulation is about 0.5°C. Because water temperature stations are often located near the bank and because of the accuracy of the temperature data, this error is acceptable. Values of  $r$  and  $V$  suggest that the model can simulate variations of water temperature satisfactorily.

The error indicators could not be developed for variables such as ice cover area and thickness because of the limited availability of observed data. Errors in simulations as measured by the error variance  $e_s^2$  can be due to a number of sources. The component due to observational error can only be reduced through accurate instrumentation and better observation station locations. One of the major error sources is the water temperature data. The



effect of stratification in the upstream region, which cannot be modeled accurately due to lack of data, can also have an important effect on the ice condition. A second component of error variance due to bias  $e_s^2$  can usually be eliminated by proper calibration. The part of the error due to the one-dimensional approximation is also unavoidable. A significant part of the error is due to shortcomings in the existing theories on river ice processes. These can be improved only when better theories become available. These shortcomings cover almost all of the river ice processes, including formation of frazil and skim ice, ice jam formation, shore ice accumulation, hanging dam formations or undercover erosion and deposition, and ice cover resistance coefficient.

### Summary and conclusion

In this study a one-dimensional model is developed for simulating river ice processes. The model is capable of simulating time-dependent conditions of the river hydraulics, water temperature and ice concentration, formation of an ice cover, undercover accumulation, thermal growth and decay of the ice cover, and mechanical stability of the cover. In the river hydraulics component, the flow condition is determined by an implicit finite-difference solution of one-dimensional unsteady flow equations. In the thermal component, distributions of water temperature and ice concentration are determined by a Lagrangian–Eulerian solution scheme for equations of transport of thermal energy and ice. A two-layer formulation is introduced to model the ice transport. In this formulation the total ice discharge is considered to consist of the surface ice discharge and the discharge of suspended ice distributed over the depth of the flow. The effect of surface ice on ice production as well as formations of skim ice and border ice are included. The dynamic formation and stability of the ice cover is formulated according to existing equilibrium ice jam theories with due consideration to the interaction between the ice cover and the flow. The undercover ice accumulation is formulated according to the critical velocity criterion. The growth and decay of the ice cover is simulated using a finite-difference formulation applicable to composite ice covers consisting of snow, ice and frazil layers. The model has been applied to the upper St. Lawrence River and the Ohio River system with good results. With the generic nature of the model structure, the model can be applied to other rivers. In view of the limited current understanding on river ice processes, further improvements of the model should be made when improved theoretical formulations become available. The model can assist the development of new formulations by analyzing field data in a comprehensive manner. Further improvements on modeling techniques are also possible by including some important two-dimensional aspects of ice processes.

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## APPENDIX A: ERROR ANALYSIS

The validity of a simulation model can be determined by comparing the simulated results with observed data. The most common method of comparison is by graphically comparing observed and simulated results. Graphical comparison gives a qualitative estimate of the accuracy of the model. Quantitative evaluation of a simulation model can be obtained using error estimates. Some of the indicators are similar to those used in the analysis of variance.

Consider a sequence of observations where  $Y_i$  and corresponding simulated values  $y_i$  where  $i = 1, 2, \dots, n$  are the observation numbers. The error  $e_i$  of any observation  $i$  is

$$e_i = y_i - Y_i \quad \text{for } i = 1, 2, \dots, n \quad (\text{A1})$$

where  $n$  is the number of observations. The error  $e_i$  can be positive or negative. During an entire simulation,  $e_i$  can fluctuate around a nonzero value. This value differs from the actual average value by an amount called the bias:

$$\text{bias} = e_b = \frac{\sum_{i=1}^n e_i}{n} = \frac{\sum_{i=1}^n (y_i - Y_i)}{n}. \quad (\text{A2})$$

Bias can be reduced to zero by calibration. In the case of the bed roughness calibration in the present study, the objective was to obtain zero bias for all observation stations. Zero bias does not guarantee error-free simulation. The error in a simulation can also be computed as an average absolute error:

$$\text{average absolute error} = e_a = \frac{\sum_{i=1}^n |y_i - Y_i|}{n}. \quad (\text{A3})$$

A better indicator of error, which can also be used in variance analysis, is the standard error estimate:

$$\text{standard error estimate} = e_s = \left[ \frac{\sum_{i=1}^n (y_i - Y_i)^2}{n} \right]^{1/2}. \quad (\text{A4})$$

Both  $e_a$  and  $e_s$  are always nonzero, even in the case of  $e_b = 0$ , unless observations at all time steps exactly match with simulated values. During the calibration of parameters, minimization of  $e_a$  and  $e_s$  can be used as the objective functions for optimization. Minimization of the absolute value of maximum positive error  $e_p$  or negative error  $e_n$  is also a commonly used objective function. The phase lag between observed and simulated results can also be used as an error indicator.

The error term, which is measured by  $e_s$  or  $e_a$  can be due to a number of causes such as: a) physical phenomena that were not formulated correctly; b) errors introduced in the numerical procedure; c) measurement errors in the observations and d) incomplete calibration. A complete error analysis has to be able to separate contributions to the error variance  $e_s^2$  from these effects. The current analysis only computes  $e_s^2$ .

The standard error estimate  $e_s$  alone is not capable of measuring the capability of the model to simulate variations. The model should be capable of simulating a known variation of the output variable as closely as possible to its observation. A simple correlation estimation and an analysis of variance is useful in developing an indicator capable of measuring this property in a model. The correlation coefficient between observed and simulated values is defined as

$$r^2 = \frac{\left[ \sum_{i=1}^n (y_i - \bar{y})(Y_i - \bar{Y}) \right]^2}{\sum_{i=1}^n (y_i - \bar{y})^2 \sum_{i=1}^n (Y_i - \bar{Y})^2} \quad (\text{A5})$$

where

$$\bar{Y} = \sum_{i=1}^n Y_i \quad (\text{A6})$$

where

$$\bar{Y} = \sum_{i=1}^n Y_i \quad (A6)$$

and

$$\bar{y} = \sum_{i=1}^n y_i. \quad (A7)$$

The coefficient of correlation  $r$  ranges between 0 and 1. A perfect model will have  $r = 1$ . Values of  $r$  closer to 1 indicate better simulations. A value of  $r < 0.6$  indicates either that the simulation model is unsuccessful or that the data set used is not capable of supporting the hypothesis that the model is capable of simulating the variation in the observations.

In addition to determining various error terms, an analysis of the behavior of the simulated result in relation to observed results is useful. The best method to determine this behavior in a statistical sense is to obtain a relationship between simulated and observed results. The simplest relationship possible is a linear one:

$$Y_i = a + by_i \quad \text{for } i = 1, 2, \dots, n. \quad (A8)$$

The value of  $r$  obtained from eq A5 can also be obtained by simple linear regression of  $Y_i$  and  $y_i$ . Values of  $a$  and  $b$  can be used to check the effectiveness of the model. For the perfect model,  $a = 0$  and  $b = 1$ . Any deviation from these values indicates an error in the simulation. The coefficient  $b$  gives a statistical estimate for the amplification by the simulation model. Amplification  $= \partial y_i / \partial Y_i = 1/b$ . When  $1/b > 1$ , a magnification of the variation by the simulation model is indicated, while  $|a| > 0$  indicated the presence of a bias. Amplification of the standard deviation  $s_y/s_Y$  can be related to the average amplification  $1/b$ , using the following equation:

$$\frac{1}{b} = \frac{1}{r} \frac{s_y}{s_Y} = \frac{1}{r} \sqrt{\frac{\sum_{i=1}^n (y_i - \bar{y})^2}{\sum_{i=1}^n (Y_i - \bar{Y})^2}}. \quad (A9)$$

The above analysis can be extended and used to explain the overall behavior of simulated results using four statistical parameters. Parameters  $a$ ,  $b$ ,  $k$  and  $e_w$  can be used as statistical estimators of bias, amplification, lag and standard error, respectively. Figure A1 shows the effect of these parameters graphically. Using these parameters, the observed results can be related to simulated results using

$$Y_i = a + by_{i+k} w_i e_w \quad (A10)$$

where  $k = \text{lag}$

$e_w = \text{standard deviation of remaining error}$

$w_i = \text{white noise.}$

By fitting this model, any error term that cannot be explained using a gross amplification or a bias is assumed as a random error. Lag  $k$  for the best fit of eq A10 is the lag for maximum cross correlation between  $y_i$  and  $Y_i$ :

$$r_k = \sqrt{\frac{[\sum_{i=1}^n (Y_i - \bar{Y})(y_{i+k} - \bar{y})]^2}{\sum_{i=1}^n (y_i - \bar{y})^2 \sum_{i=1}^n (Y_i - \bar{Y})^2}}. \quad (A11)$$

Parameters  $a$  and  $b$  can be determined by linear regression. Both  $a \neq 0$  and  $b \neq 1$  indicate mostly improper calibration or weaknesses of the model. Their values can suggest the type of parameters to be improved. The parameter  $e_w$  is the remaining standard error to be explained by improving the model.



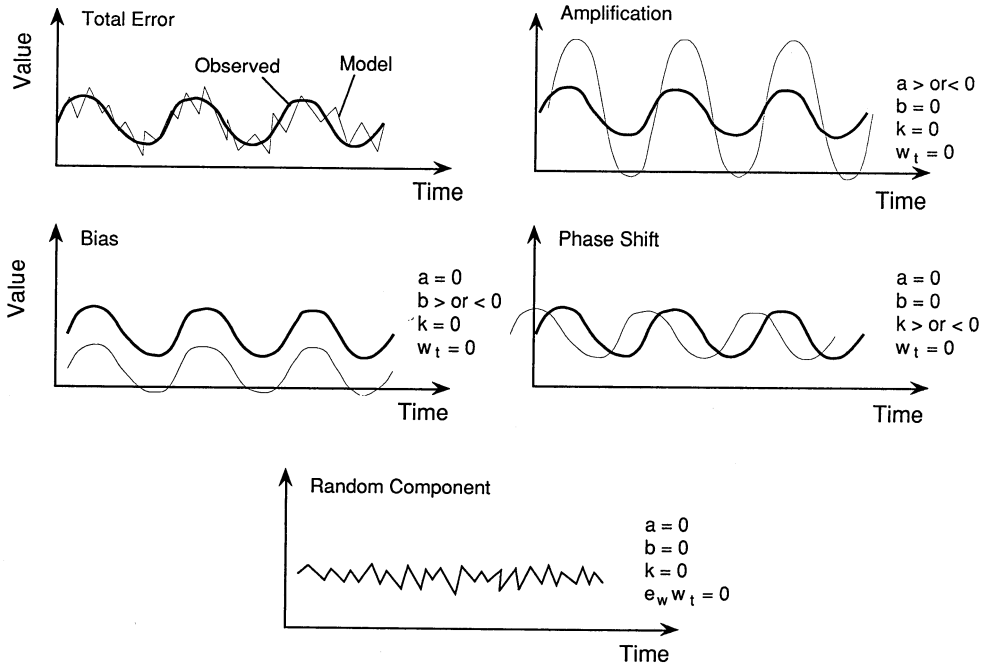


Figure A1. Components of the simulation error.

The effectiveness of the model can also be determined using methods similar to those used in the analysis of variance. Considering that the purpose of the model is to explain the total variation of the observations, components of the variation can be expressed using the following terms:

$$\text{Total sum of squares} = \text{SST} = \sum_{i=1}^n (Y_i - \bar{Y})^2 = n\sigma_Y^2 \quad (\text{A12})$$

and

$$\text{SSE} = \text{error sum of squares} = \sum_{i=1}^n (Y_i - y_i)^2 = ne_s^2. \quad (\text{A13})$$

If the model is calibrated to make the bias  $e_b = 0$ , then  $\bar{y} = \bar{Y}$ , and the percentage variation explained for  $b = 1$  is

$$\begin{aligned} V &= \frac{\text{SST} - \text{SSE}}{\text{SST}} \\ &= \frac{\text{variation explained by the model}}{\text{total variation}} \end{aligned} \quad (\text{A14})$$

where  $V$  is the percentage variation explained ( $r^2$ ). Although the bias has not been eliminated in the current model and  $a \neq 0$  and  $b \neq 1$ , the same indicator can be used to explain the effectiveness of the model:

$$V = \frac{\text{SST} - \text{SSE}}{\text{SST}} = 1 - \frac{\sum_{i=1}^n (y_i - Y_i)^2}{\sum_{i=1}^n (Y_i - \bar{Y})^2} = 1 - \frac{e_s^2}{\sigma_Y^2} \quad (\text{A15})$$

where  $\sigma_Y$  is the variance of  $Y_i$ ,  $i = 1, 2, \dots, n$ .  $V$  is defined as in eq A15 because it is similar to the definition of percentage variation explained in the linear regression analysis.

The part of SSE due to parameter estimation errors can be easily separated if the parameter sensitivities are known. Let the simulated result of  $k^{th}$  system variable as a function of the parameters  $\theta_1, \theta_2, \dots, \theta_m$  be  $y_k(\theta_1, \theta_2, \dots, \theta_m) = 0$ . A small change in the variables can be related to the changes in parameters as

$$\Delta y_k = \frac{\partial y_k}{\partial \theta_1} \Delta \theta_1 + \frac{\partial y_k}{\partial \theta_2} \Delta \theta_2 + \dots + \frac{\partial y_k}{\partial \theta_m} \Delta \theta_m \quad (A16)$$

where  $\frac{\partial y_k}{\partial \theta_1}, \frac{\partial y_k}{\partial \theta_2}$  are the sensitivities of the variable to the parameters. Equation A16 can be used to write an expression for the variance of  $y_k$ , assuming that the above partial differentials or sensitivities are finite and have constant values for a given set of parameters:

$$\sigma_{y_k}^2 = \left( \frac{\partial y_k}{\partial \theta_1} \right)^2 \sigma_{\theta_1}^2 + \left( \frac{\partial y_k}{\partial \theta_2} \right)^2 \sigma_{\theta_2}^2 + \dots + \left( \frac{\partial y_k}{\partial \theta_m} \right)^2 \sigma_{\theta_m}^2. \quad (A17)$$

If the standard error variances of the parameters and the sensitivities are known, the error variance of the simulated result due to parameter errors can be determined using eq A17. This is the part of the total error variance  $e_s^2$  explained by parameter errors. The remaining error variance is due to errors in the observations and the general model structure itself.

# REPORT DOCUMENTATION PAGE

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1. AGENCY USE ONLY (Leave blank)		2. REPORT DATE May 1993		3. REPORT TYPE AND DATES COVERED	
4. TITLE AND SUBTITLE  A Mathematical Model for River Ice Processes				5. FUNDING NUMBERS  DACA 89-84-K-0008	
6. AUTHORS  A.M. Wasantha Lal and Hung Tao Shen					
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)  Clarkson University Potsdam, New York				8. PERFORMING ORGANIZATION REPORT NUMBER  CRREL Report 93-4	
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)  U.S. Army Cold Regions Research and Engineering Laboratory 72 Lyme Road Hanover, New Hampshire 03755-1290				10. SPONSORING/MONITORING AGENCY REPORT NUMBER	
11. SUPPLEMENTARY NOTES					
12a. DISTRIBUTION/AVAILABILITY STATEMENT  Approved for public release; distribution is unlimited.  Available from NTIS, Springfield, Virginia 22161.				12b. DISTRIBUTION CODE	
13. ABSTRACT (Maximum 200 words)  River ice processes are complex phenomena that are affected by many factors, including meteorological conditions, thermal inputs, hydraulic conditions and channel geometry. In this study a one-dimensional model called RICE is developed for simulating ice processes in rivers. In the river hydraulics component, the flow condition is determined by an implicit finite-difference solution of one-dimensional unsteady flow equations. In the thermal component, distributions of water temperature and ice concentration are determined by a Lagrangian-Eulerian solution scheme for equations of transport of thermal energy and ice. A two-layer formulation is introduced to model the ice transport. In this formulation the total ice discharge is considered to consist of the surface ice discharge and the discharge of suspended ice distributed over the depth of the flow. The effect of surface ice on ice production, as well as the formation of skim ice and border ice, is included. The dynamic formation and stability of the ice cover is formulated according to existing equilibrium ice jam theories with due consideration to the interaction between the ice cover and the flow. The undercover ice accumulation is formulated according to the critical velocity criterion. The growth and decay of the ice cover is simulated using a finite-difference formulation applicable to composite ice covers consisting of snow, ice and frazil layers. The model has been applied to the St. Lawrence River and the Ohio River system, with simulated results comparing favorably with field observations. Future improvements on the mathematical model as well as theoretical formulations on various ice processes are discussed.					
14. SUBJECT TERMS  Ice                      Ohio River              St. Lawrence River Mathematical models      River ice				15. NUMBER OF PAGES 85	
				16. PRICE CODE	
17. SECURITY CLASSIFICATION OF REPORT  UNCLASSIFIED		18. SECURITY CLASSIFICATION OF THIS PAGE  UNCLASSIFIED		19. SECURITY CLASSIFICATION OF ABSTRACT  UNCLASSIFIED	
20. LIMITATION OF ABSTRACT					

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