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UNPUBLISHED REPORT
RAPPORT NON PUBLIÉ

This manuscript has been submitted to
the Canadian Society for Civil Engineering
5th Canadian Hydrotechnical Conference
May 26 and 27, 1981, Fredericton, N. B.
for publication and the contents are
subject to change.

SURGES FROM ICE JAM RELEASES: A CASE STUDY

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December 1980

MANAGEMENT PERSPECTIVE

The discharges and levels reached downstream of an ice jam when it breaks is not easily analyzed hydraulically because of the unknown effects from the ice. In this report, certain assumptions were made and an adaptation of the MOBED model was applied to compute stages and discharge as a function of time. The results are compared with an observed collapse of an ice jam.

The method is applicable to other situations but at this time the reliability of the computation is uncertain. This study represents a significant advance in the hydraulics of ice jams but more field data must be analyzed to ascertain the reliability of the computation method in a statistical sense.

T. M. Dick
Chief, Hydraulics Division
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January 15, 1981

PERSPECTIVE-GESTION

Il est difficile d'analyser hydrauliquement les débits et les niveaux atteints ev aval à cause des effets inconnus de la glace. Dans ce rapport, on a fait certaines suppositions et appliqué une adaptation du modèle MOBED pour calculer les hauteurs d'eau et le débit en fonction du temps. Les résultats sont comparés avec ceux d'une débâcle observée.

La méthode est applicable à d'autres situations, mais, pour le moment, la fiabilité des calculs est incertaine. La présente étude représente un progrès important pour l'hydraulique des embâcles, mais il faut analyser davantage de données recueillies sur le terrain pour établir la fiabilité de la méthode de calcul au sens statistique.

Le chef de la Division de l'hydraulique
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Le 15 janvier 1981

SURGES FROM ICE JAM RELEASES:

A CASE STUDY

S. Beltaos[1] and B. G. Krishnappan[2]

SYNOPSIS

Witness accounts of spring ice breakup in rivers often mention violent ice runs with extreme water speeds and rapidly rising water levels. Such events are believed to follow the releases of major ice jams. However, the associated dynamic aspects are little known. To gain preliminary understanding of this problem, it is attempted to "reconstruct" a partially documented jam release case on the Athabasca River at Fort McMurray, reported recently by others. The equations of the ice-water flow that occurs after the release of an ice jam are formulated. It is shown that the problem may be approximately treated as one-dimensional, unsteady, water-only flow of identical total depth and average velocity. The retarding effect of the frequently encountered intact ice cover below the jam is considered implicitly, that is, by adjusting the friction factor so as to match predicted and observed downstream stages. Predicted velocities are then shown to agree with those estimated by site observers. The effects of jam length are considered next by assuming longer jams of the same maximum depth. Peak surge velocities are only slightly influenced by jam length but the duration of surging velocities increases with length and so does the peak stage. Less than two hours after the jam release, the surge was arrested and a new jam formed causing further stage increases. Present capabilities of modelling the reformation process are discussed and the major unknowns identified.

KEYWORDS

breakup; flooding; ice; ice jam; release; river; surge; unsteady flow

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ÉLÉVATIONS SUBITES DU NIVEAU CAUSÉES PAR DES DÉBÂCLES

ÉTUDE DE CAS

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RÉSUMÉ

Les récits de témoins de débâcles printannières dans des rivières font souvent mention d'afflux violents de glaces accompagnés de très grandes vitesses du courant et d'élévations rapides du niveau de l'eau. On croit que ces phénomènes font suite à des débâcles importantes. Toutefois, on connaît mal les aspects dynamiques connexes. Afin d'acquérir une compréhension préliminaire du problème, on essaie de reconstituer un cas de débâcle partiellement documenté survenu sur la rivière Athabaska à Fort McMurray et récemment signalé par l'autres. Les équations de l'écoulement d'eau et de glace qui se produit après une débâcle sont données. On montre que le problème peut être traité en gros comme s'il s'agissait d'un phénomène unidimensionnel, instable, dont le courant ne se compose que d'eau de profondeur totale et de vitesse moyenne identiques. L'effet retardateur de la glace intacte souvent rencontrée en aval de l'embâcle est considéré de façon implicite, c'est-à-dire en ajustant le facteur de friction pour qu'il corresponde aux hauteurs d'eau prédites et observées en aval. On montre ensuite que les vitesses prédites concordent avec celles que des observateurs présents sur les lieux ont estimées. On examine ensuite les effets de la longueur de l'embâcle en supposant des embâcles plus longues de même profondeur maximale. Les vitesses maximales d'élévation ne sont que légèrement influencées par la longueur de l'embâcle, mais la durée des vitesses de crue augmente avec la longueur et il en va de même pour la hauteur d'eau maximale. Moins de deux heures après la débâcle, la poussée a été stoppéé et une nouvelle embâcle s'est formée, provoquant d'autres accroissements du niveau des eaux. Il est question des capacités actuelles d'établissement d'un modèle du processus de reformation et les principales inconnues sont déterminées.

INTRODUCTION

Witness accounts of spring ice breakup in rivers often mention violent ice runs with extreme water speeds and rapidly rising water levels. Gerard (1979) quoted several accounts of such events and suggested that they can only be explained by the action of surges caused by the release, and possibly the reformation, of major ice jams. This is plausible since an ice jam causes a significant local perturbation on the stage profile of a stream with very large gradients near its toe or downstream end. Failure of the jam releases a large water wave that results in high speeds and rapid stage rises at downstream locations.

There are several practical problems that are related to surges from ice jam releases, such as short and long term forecasting of peak water levels near a populated area located downstream of a major jamming site; possible bed scour and bank erosion due to relatively brief but intense ice runs; peak stages during reformation of a released jam. Such dynamic aspects of ice jamming phenomena are poorly understood at present, especially in quantitative terms. The writer is only aware of two pertinent investigations: an application of an open-water unsteady flow model to assess surge effects on bed scour (Mercer and Cooper 1977) and a theoretical investigation of surging and new jamming that is now in progress (Henderson and Gerard 1980).

The lack of understanding of ice jam dynamics is very likely due to the lack of pertinent quantitative data; indeed one can easily imagine the difficulties involved in obtaining adequate documentations of jam release events. First, the longitudinal water level profile along and downstream of an ice jam must be known shortly before its release; second, water level-time variations at downstream locations are needed as a means of assessing the results of the surge; and third, channel geometry and flow conditions are necessary as input information prior to application of a mathematical model. Recently, a partially documented release case was reported by Doyle and Andres (1979): the 1979 breakup on the Athabasca River at Fort McMurray which was triggered by the release of a major ice jam upstream. Fortunately, it was possible to approximately determine the water level profile along this jam and to obtain the subsequent stage-time variation at a bridge site in Fort McMurray. River cross sections were surveyed later under open-water conditions. Though this information is far from complete, it does afford an opportunity for an exploratory case study, principally intended to be a means of gaining preliminary understanding of the jam surge problem.

In the following sections, it is attempted to formulate the governing differential equations of the ice-water surge phenomenon and utilize them to "reconstruct" the results reported by Doyle and Andres (1979).

UNSTEADY ICE-WATER FLOW

In this section, the unsteady flow of water and ice that results from the release of an ice jam is considered. For mathematical simplicity, the flow is assumed to be two-dimensional such as it occurs in a very wide, rectangular, prismatic channel. With proper adaptation, some of the final equations can be shown to hold for flow in a channel of arbitrary cross-sectional shape and plan view.

With reference to Fig. 1 two flow layers can be distinguished: (i) The fragmented cover of thickness t, including the water contained in its voids; if the porosity, ε, of the cover remains the same for both the region above and the region below the water surface and if the ice is floating, then the submerged thickness of the cover is equal to $s_i t$ with s_i=specific gravity of ice. (ii) The layer of thickness h that consists of water, between the bottom of the cover and the channel bed. Figure 1 shows the assumed velocity distribution across the two layers; the fragmented cover is assumed to act as a solid due to interlocking among the fragments and thence to have a uniformly distributed velocity, u_i.

Continuity Equations

Assuming that the porosity ε of the cover is constant[1], the mass conservation for ice results in (thermal effects are neglected):

$$(1-\varepsilon)\frac{\partial t}{\partial T} + \frac{\partial q_i}{\partial x} = 0 \tag{1}$$

in which T=time; x=longitudinal distance; and q_i=ice discharge per unit width, given by:

$$q_i = (1-\varepsilon)\, u_i t \tag{2}$$

Substituting Eq. 2 in Eq. 1 gives:

$$\frac{\partial t}{\partial T} + \frac{\partial (u_i t)}{\partial x} = 0 \tag{3}$$

Considering the mass conservation of water, gives:

$$\frac{\partial h}{\partial T} + \varepsilon s_i \frac{\partial t}{\partial T} + \frac{\partial q_w}{\partial x} = 0 \tag{4}$$

in which q_w = water discharge, given by

$$q_w = q' + \varepsilon u_i s_i t \tag{5}$$

with q'=water discharge in the second layer, i.e:

$$q' = \int_0^h u\,dy = Vh \tag{6}$$

where V=average velocity in the layer. Substituting Eq. 5 in Eq. 4 and taking Eq. 3 into account, gives:

[1] In reality, ε is expected to vary, but only within a narrow range.

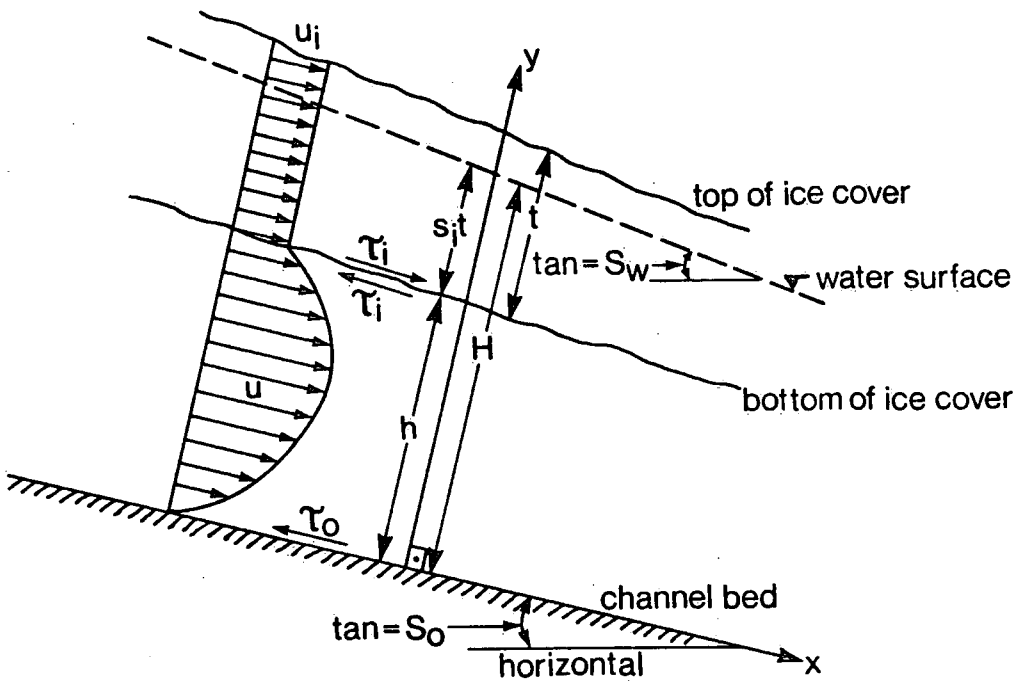

Fig. 1 Definition Sketch

(7) $$\frac{\partial h}{\partial T} + \frac{\partial q'}{\partial x} = 0$$

which may be viewed as the continuity equation for the second layer.

To write the overall mass flux equation for the ice and water flow, multiply Eq. 1 by ρ_i (=ice density) and Eq. 4 by ρ_w (= water density) and add, to find:

(8) $$\rho_w \frac{\partial H}{\partial T} + \frac{\partial \rho_w q}{\partial x} = 0$$

in which H = overall water depth, given by:

(9) $$H = h + s_i t$$

and $\rho_w q$ is the total mass flux, that is:

(10) $$\rho_w q = \rho_i q_i + \rho_w q_w$$

It is noted that Eq. 8 is identical to the continuity equation for water flow of depth H and discharge q.

Momentum Equations

The momentum equation for the water layer in a direction parallel to the channel bed is:

(11) $$\rho_w \left(\frac{\partial u}{\partial T} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \right) = \rho_w g S_o - \frac{\partial p}{\partial x} + \frac{\partial \tau}{\partial y}$$

in which u, v = velocity components in the x and y directions respectively; g=magnitude of the acceleration of gravity=9.8 m/s^2; S_o=channel bed slope; p=pressure, assumed approximately equal to the hydrostatic pressure; and τ=shear stress parallel to the x-axis, acting on a plane normal to the y-axis. The differential equation of continuity reads:

(12) $$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0$$

By virtue of Eq. 12, the bracketed term on the LHS of Eq. 11 may also be written as $(\partial u/\partial T) + (\partial u^2/\partial x) + (\partial uv/\partial y)$. Making this substitution and integrating both sides of Eq. 11 from y=o to y=h, gives:

(13) $$\rho_w \left\{ \left(\frac{\partial}{\partial T} \int_o^h u dy\right) - (u)_h \frac{\partial h}{\partial T} + \left(\frac{\partial}{\partial x} \int_o^h u^2 dy\right) - (u^2)_h \frac{\partial h}{\partial x} + (u)_h (v)_h \right\} = \rho_w g S_o h - \left(\frac{\partial}{\partial x} \int_o^h p dy \right) + (p)_h \frac{\partial h}{\partial x} - (\tau_i + \tau_o)$$

in which τ_o=bed shear stress and τ_i=shear stress on the bottom of the cover, considered positive if it tends to retard the water layer and accelerate the cover, as sketched in Fig. 1. It is noted that $(u)_h = u_i$ and $p = \rho_w g\,(H-y)$. To determine $(v)_h$, Eq. 12 may be integrated from y=o to y=h; this gives:

$$(14) \qquad (v)_h = u_i \frac{\partial h}{\partial x} - \frac{\partial}{\partial x} \int_o^h u\,dy$$

Using Eqs. 6, 7, 9 and 14, Eq. 13 may be simplified to:

$$(15) \qquad \rho_w \left(\frac{\partial q'}{\partial T} + \frac{\partial m'}{\partial x} \right) = \rho_w g h S_w - (\tau_i + \tau_o)$$

in which S_w = slope of the water surface and

$$(16) \qquad m' = \int_o^h u^2 dy$$

Consider next the momentum equation for an element of the cover of length dx. For simplicity, the equation is written in a direction parallel to the water surface so as to cancel the pressure forces; the cosines of small angles which would ordinarily appear in the equation are assumed equal to unity. If dm_i is the total, ice and water, mass of the element and a_i is its acceleration (note $a_i = du_i/dT$=constant across the element), then:

$$(17) \qquad (dm_i)\, a_i = g\,(dm_i)\, S_w + \tau_i\, dx$$

But

$$(18) \qquad a_i = \frac{du_i}{dT} = \frac{\partial u_i}{\partial T} + u_i \frac{\partial u_i}{\partial x}$$

and

$$(19) \qquad dm_i = \rho_i (1-\varepsilon)\, t dx + \rho_w\, \varepsilon s_i t\, dx = \rho_w s_i t\, dx$$

Substituting Eqs. 18 and 19 in Eq. 17 and rearranging, gives:

$$(20) \qquad \rho_w s_i t \left(\frac{\partial u_i}{\partial T} + u_i \frac{\partial u_i}{\partial x} \right) = \rho_w g s_i t S_w + \tau_i$$

A similar form may be obtained for Eq. 15 if we make the one-dimensional flow approximation $m' \approx V^2 h$ and use Eq. 7 to show that:

$$(21) \qquad \rho_w h \left(\frac{\partial V}{\partial T} + V \frac{\partial V}{\partial x} \right) = \rho_w g h S_w - (\tau_i + \tau_o)$$

Once the initially stationary cover accelerates to the full water speed, the one-dimensional approximation will indicate that $u_i \approx V$. In this case, addition of Eqs. 20 and 21 will give:

$$\rho_w H \left(\frac{\partial V}{\partial T} + V \frac{\partial V}{\partial x} \right) = \rho_w g H S_w - \tau_o \tag{22}$$

which is the same as the momentum equation for flow of depth H and average velocity V. Further, with $u_i \simeq V$, it can be shown that q (defined by Eq. 10) becomes equal to VH and Eq. 8 reduces to

$$\frac{\partial H}{\partial T} + \frac{\partial (VH)}{\partial x} = 0 \tag{23}$$

It follows that under the conditions of (i) the one-dimensional flow approximation and (ii) full development of the speed of the cover, the overall equations governing the motion of water and ice are identical to those of ordinary water flow with depth H and average velocity V. With proper boundary and initial conditions, the jam release problem could then be handled by means of existing unsteady flow models. It is noted that a more elaborate derivation for a channel of arbitrary cross-sectional shape and plan form gave the same correspondence between water-ice flow and water flow of the same overall depth and average velocity.

To estimate the time required for full development of the ice cover speed, an order-of-magnitude analysis was performed assuming constant water speed and thickness t. It was found that u_i becomes equal to 95 percent of the water speed within a few minutes. Since the acceleration time is quite small, it could, as a first approximation, be neglected and the computation based on the open-water equations from the very beginning of the process (instant of release).

FORT MCMURRAY CASE STUDY

Figure 2 is a plan of the Athabasca River in the vicinity of the town of Fort McMurray (Alberta), a site notorious for troublesome ice jamming. The 1979 breakup at this site was documented by Doyle and Andres (1979) who reported that breakup at MacEwan Bridge was triggered by the release of an ice jam that had formed a few kilometres upstream. The longitudinal stage profile of this jam was determined shortly before its release and can be used to define the initial conditions. The passage of the surge was observed at MacEwan Bridge and a few stage readings and velocity estimates are available. Channel cross sections below MacEwan Bridge have been provided by Doyle and Andres (1979); additional cross sections for the reach above the bridge were kindly provided by M. Anderson of the Transportation and Surface Water Engineering Division of Alberta Research Council.

To solve the governing differential equations, a numerical algorithm was used that has been developed by Krishnappan and Snider (1977) for one-dimensional unsteady flow with variable channel width. Though this algorithm is capable of dealing with cross sections of arbitrary shape, it was deemed sufficient for the present purpose to assume rectangular sections, as

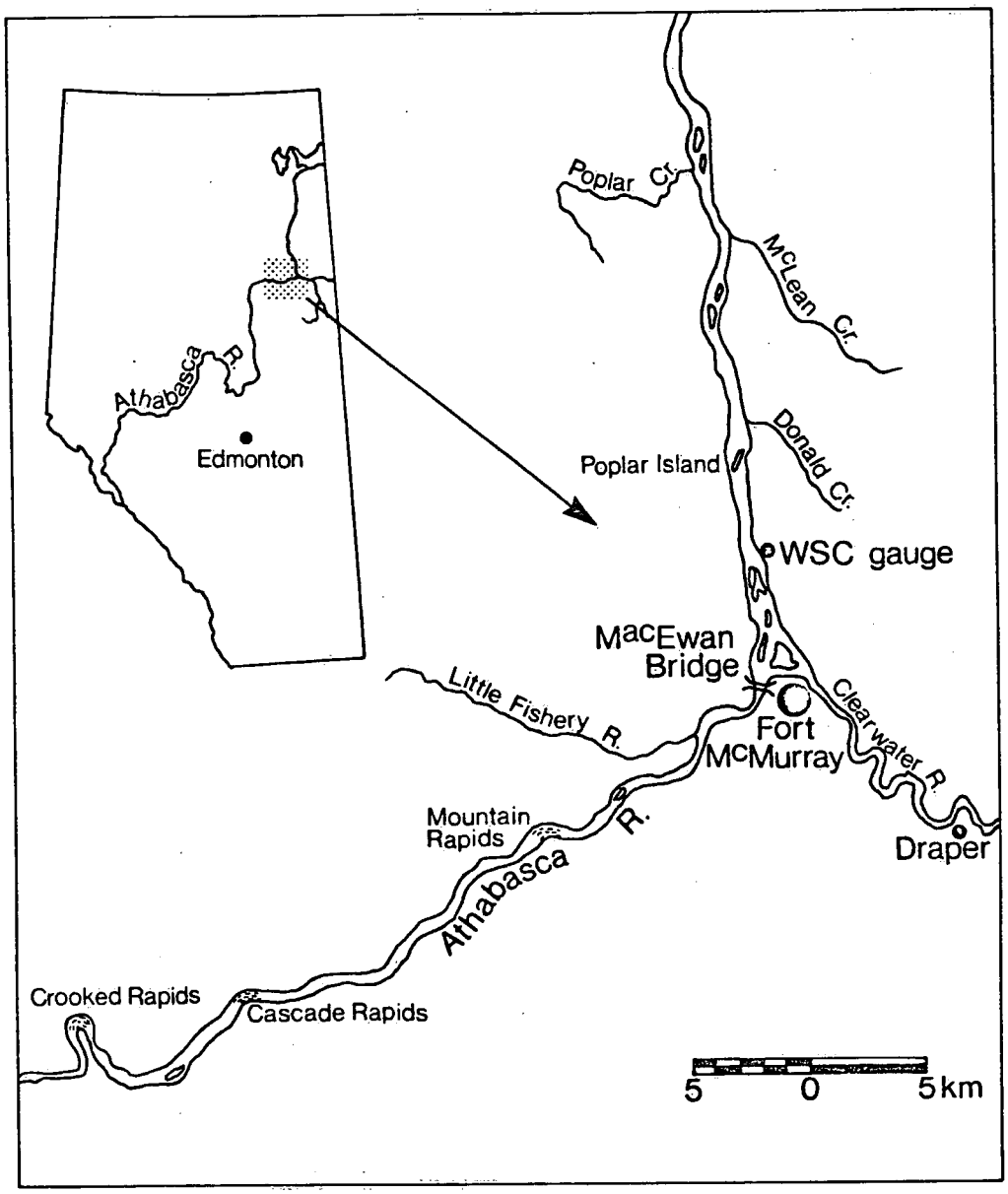

Fig. 2 Location map of study area (after Doyle and Andres, 1979 with changes).

follows. First, a cross section was approximated by a trapezoid of depth equal to the distance of the water surface from the average channel bed level. This trapezoid was then approximated by a rectangle of the same depth and width equal to the average width of the trapezoid. The channel width and depth between successive surveyed cross sections were determined by linear interpolation.

Initial conditions for the computation consist of the water surface and bed profiles as well as flow discharge along the study reach. For the jammed reach, it is assumed that flow through the voids of the jam is negligible, therefore the value of q is equal to q' which in turn is equal to the water discharge prior to release. Discharge was estimated as 900 m^3/s below the mouth of the Clearwater River (see Fig. 2) and 700 m^3/s above this site, based on Water Survey of Canada records.

In addition to the initial conditions, Krishnappan and Snider's algorithm requires depth or flow rate at the upstream and downstream boundaries of the study reach plus an estimate of the friction factor or of the ratio V/V_* (V_*=shear velocity) which is assumed independent of x and T. The boundary conditions were specified simply by choosing the boundaries sufficiently far upstream of the jam and downstream of MacEwan Bridge to ensure that surge effects do not reach these locations during the computation period. The parameter V/V_* was left "free", i.e. it was selected by a trial and error process so as to give optimum agreement between predicted and observed stages at McEwan Bridge. Though this parameter is known for open-water conditions ($V/V_* \simeq 16$) and should probably apply to unimpeded ice-water flow, this value may not be appropriate for the present case study; downstream of the ice jam, the river was not open but covered by 1m thick sheet ice with occasional open-water sections. What the friction factor should be in this reach is unknown and certainly it would be expected to change with time and distance as the surge moves in and dislodges the sheet ice cover. Because this effect is too complex, it was considered reasonable to use an "average" constant value; clearly, this value should be less than the open-water value.

Figure 3 shows the river bed profile in the study reach along with the initial water surface profile as documented by Doyle and Andres (1979). The actual data points of Doyle and Andres are also plotted so as to show the degree of "smoothening" that was applied for computational purposes. The time T=0 is fixed at 1950 h, April 28, 1979.

Figure 4 shows stage-time variations as computed for different values of V/V_* along with available observations. The best agreement between computation and observation seems to be obtained when V/V_*=9.0. Note that all computed curves have a peak and decline slowly afterwards. However, the observations show the stage to remain fairly steady after $T \simeq 50$ min. This is probably due to new jamming that occurred somewhere downstream of MacEwan Bridge. According to Doyle and Andres, ice movement at the bridge ceased at T=165 min (2235 h, April 28) and a major jam was observed in the morning of April 29 with its toe 14 km below, and head 11 km above, MacEwan Bridge. Assuming that the new jam was initiated at the above indicated location[2], it was estimated that, with V/V_*=9.0, the time of initiation would

[2]This is the furthest possible location from MacEwan Bridge; the jam might have been initiated upstream of this site and slowly moved downstream during the night of April 28 to 29 by intermittent "shoves".

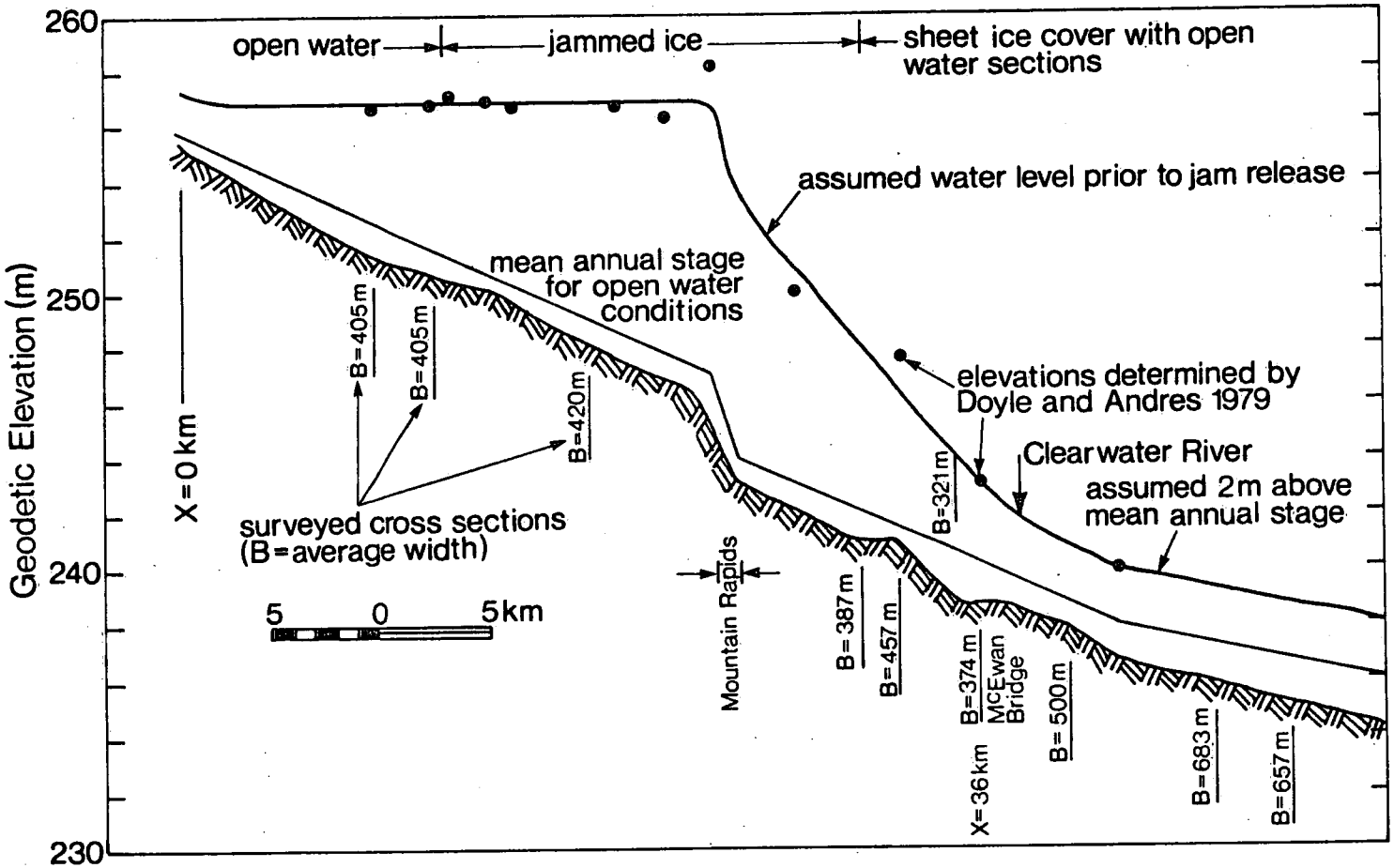

Fig. 3 Initial flow conditions and channel geometry (T=0 is taken at 1950 h, April 28, 1979).

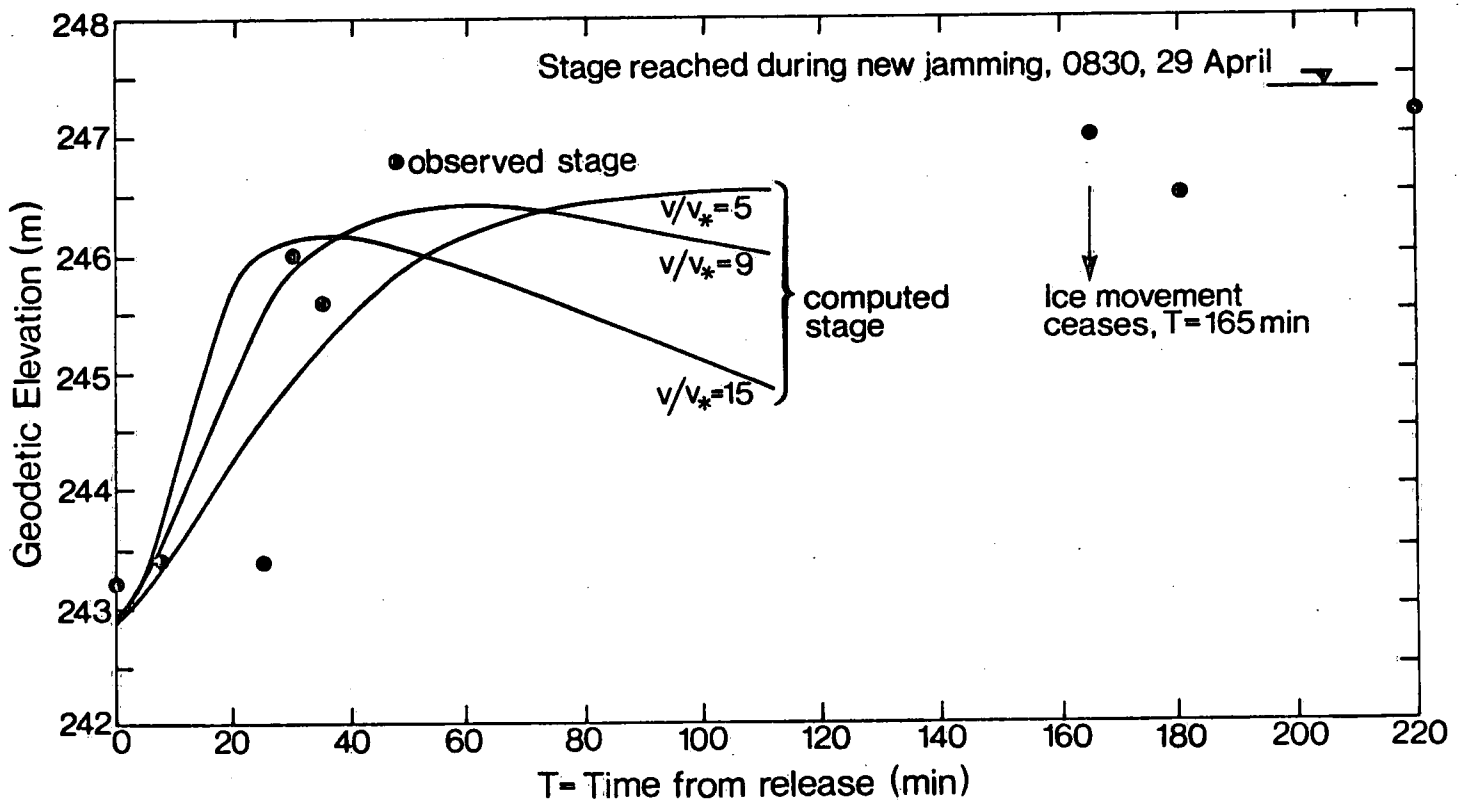

Fig. 4. Computed versus observed stage-time variations at MacEwan Bridge (origin of T: 1950 h, April 28, 1979. Observed stages are approximate).

have been $T \simeq 70$ min. For $T>70$ min, effects of the new jamming would be experienced at MacEwan Bridge.

Figure 5 shows the variation of V with time at MacEwan Bridge as computed for $V/V_* = 9.0$. At T=35 min, the computed value of V is 2.2 m/s while the surface velocity was estimated by site observers to be between 2 and 3 m/s. Considering that surface velocities are typically 15 percent larger than average values, the agreement between prediction and observation appears satisfactory.

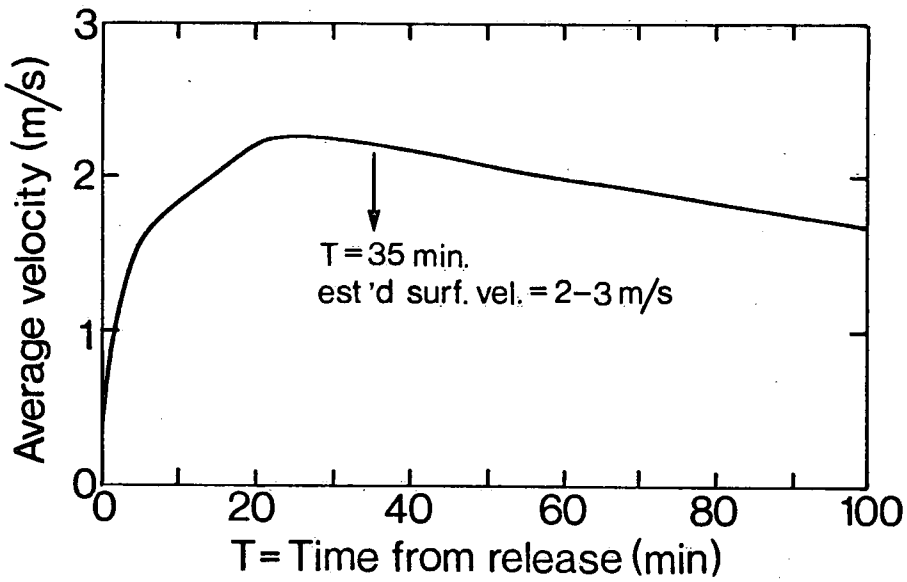

Fig. 5 Computed velocity-time variation at MacEwan Bridge ($V/V_* = 9$).

Considering again Fig. 3, it may be noticed that the stage profile of the ice jam does not include any section parallel to the normal river slope. This implies that this jam did not attain equilibrium in the sense adopted by Uzuner and Kennedy (1976). This is the result of the jam's limited length. Had the supply of ice been larger, an equilibrium section would have formed; this would have caused a longer jam than the one that actually formed, though not necessarily deeper. Considering that such an occurrence is not inconceivable, it is of interest to examine the effects of a hypothetical jam with the same maximum depth as that of the actual jam but with larger length. Figure 6 shows the assumed initial profile of the hypothetical jam: a constant depth, equal to the maximum depth of the actual jam, is assumed to occur in a reach of length L_e and a horizontal water surface transition is drawn between this reach and the uniform flow, open-water, reach upstream. Figure 7 shows the resulting peak stage at MacEwan Bridge plotted versus L_e using $V/V_* = 9.0$; for $L_e = 25$ km, this peak would have been 1.3 m higher than the one that actually occurred. The main effect of L_e on V is associated with the duration of surging velocities. For $L_e = 0$, Fig. 5 indicates a maximum of 2.3 m/s for V, while velocities in excess of 2 m/s lasted for about 45 min. For $L_e = 25$ km, the maximum value of V was calculated as 2.35 m/s but velocities larger than 2 m/s persisted for 110 min.

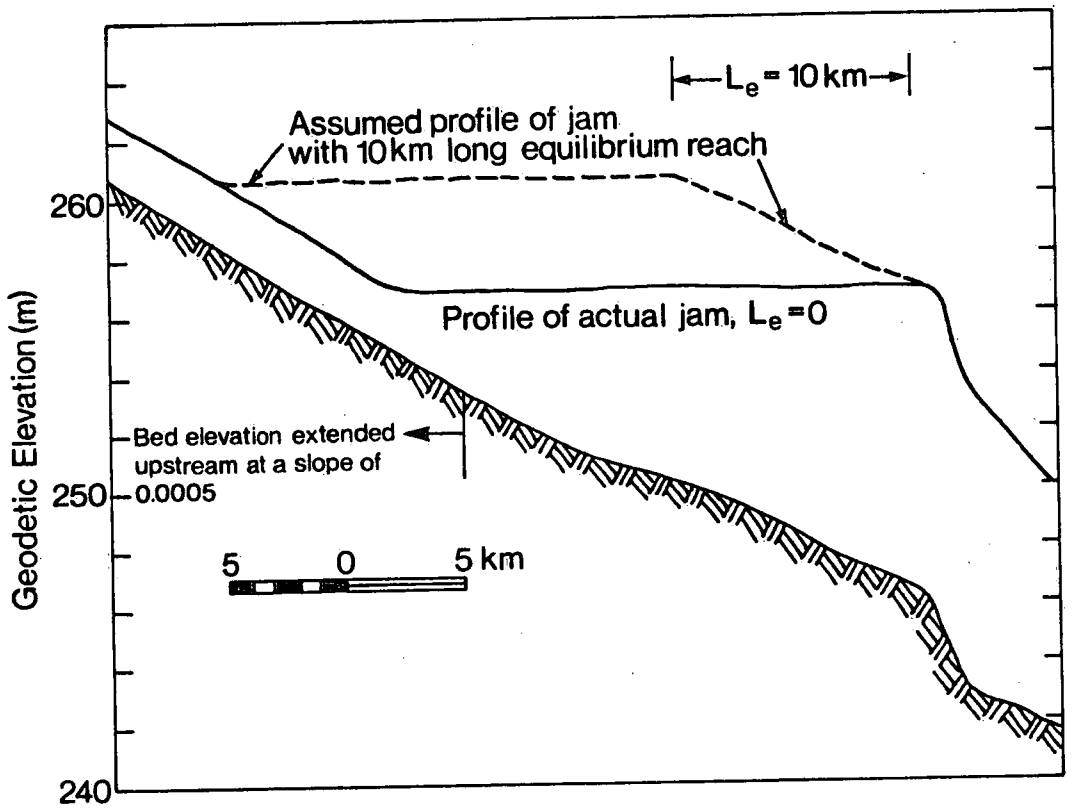

Fig. 6 Illustration of initial profile assumed for a hypothetical jam with an equilibrium reach.

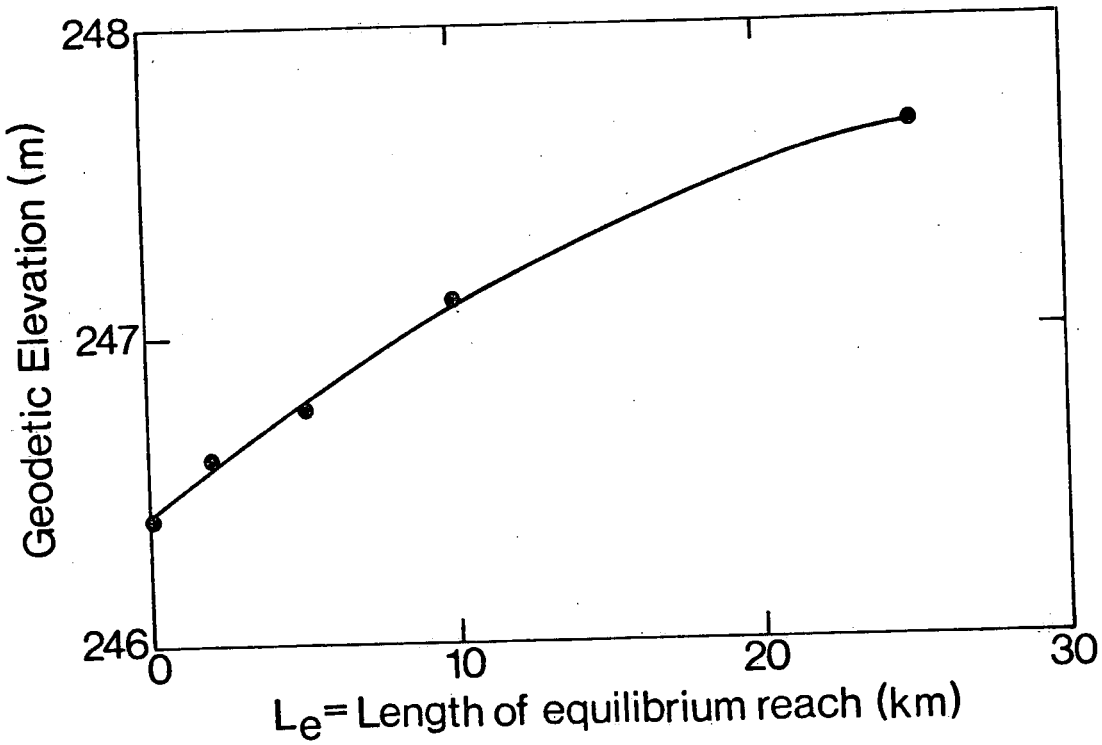

Fig. 7 Effect of jam length on peak surge stage at MacEwan Bridge (computed with $V/V_* = 9$).

DISCUSSION

From the foregoing analysis, it appears that a one-dimensional, unsteady, open-water flow model can be applied to the ice-water flow that results from the release of an ice jam using appropriate definitions of the mass and momentum fluxes. Realistic predictions can be made with this approach provided a suitable value is selected for the coefficient V/V_*. At this time, it is not known how this coefficient is to be predicted because of complications arising from the frequent existence of solid ice sheets below an ice jam. For the present study, the "best" value of V/V_* was found equal to 9 which is between the open-water value ($\simeq 16$) and the apparent value for flow under a 1m thick ice cover (=5). The apparent value of V/V_* is defined as the ratio of the apparent V(=q/H) to the apparent $V_*(=\sqrt{gHS_f}$; S_f =energy slope). The apparent V/V_* applies when the cover is stationary but has to increase when the cover is set in motion. Additional case studies would help to develop a method for predicting suitable values of V/V_*.

The possible effects of the jam's length on downstream flow conditions were investigated using $V/V_*=9$. It was found that jams of the same maximum depth as that of, but longer than, the actual jam would have resulted in increased peak stages and durations of surging velocities.

The peak value of V at MacEwan Bridge was calculated as 2.3 m/s, occurring at T≃23 min. At T=35 min, the calculated V had dropped slightly to 2.2 m/s and this is in accord with an estimated surface velocity of 2-3 m/s, reported by site observers. It is of interest to note here that surface velocities of 5-6 m/s occurred at this site during the 1977 breakup (Doyle 1977); this implies corresponding average velocities of 4.3-5.2 m/s which are about twice those of 1979. The difference could be produced by one or more of several factors such as a deeper jam; a jam located closer to the observation site than the 1979 jam; a steeper toe slope; and a higher initial discharge. Unfortunately, the origin of the 1977 surge is unknown but chances are that the released jam was located at about the same distance above MacEwan Bridge as did the 1979 jam. The 1977 discharge was about 1300 m^3/s which may account for a part but not for all of the difference in surging velocities. It can be shown that, other things being equal, surge speeds are approximately proportional to the square root of the initial water surface slope at the jam toe. Figure 3 indicates an initial toe slope of about 10^{-3}; hence, it is estimated that the toe slope of the jam responsible for the 1977 surge would be close to but not greater than 4×10^{-3}. This value is not uncommon for ice jams in the vicinity of Fort McMurray (see Doyle 1977; Doyle and Andres 1978, 1979).

It has been pointed out that predictions cannot be expected to be realistic beyond T=70 min, due to new jamming that occurred at a location no farther than 14 km below MacEwan Bridge. The toe of the new jam was observed at this location about 12 hours after the surge; it is thus possible that jamming first occurred at a distance less than 14 km from MacEwan Bridge and the toe advanced by "shoves" during the intervening time. The probability of this occurrence is enhanced if it is considered that in 1977 a much more violent ice run was arrested at Poplar Island (9 km from MacEwan Bridge - Fig. 2; Doyle 1977). If this had also been the case in 1979, it is estimated that prediction would only apply until T≃40 min.

Regardless of the actual timing and location of the new jamming, prediction of subsequent flow conditions depends on several factors, as indicated below:

(i) Upstream surge characteristics
(ii) Unsteady flow equations under a stationary fragmented ice cover (new ice jam)
(iii) Mechanisms of upstream propagation and vertical growth (thickening) of an ice jam.
(iv) Downstream boundary condition, that is, discharge or depth variation with time at the jam toe.
(v) Stability of the jam toe.

Item (i) can be dealt with using the approach presented herein while item (ii) has already been discussed in an earlier section where continuity and momentum equations were developed (see also similar equations derived by Uzuner and Kennedy 1976 for flow under a stationary cover). For a situation where an ice jam lengthens in the upstream direction, two flow models must be applied simultaneously: a model of ice-water flow for the region upstream of the jam head and a model for flow under a stationary cover for the region downstream of the jam head. The location of the boundary between these regions depends on time in a manner dictated partly by item (iii) and partly by the incoming ice discharge which is related to item (i). Item (iii) can be

formulated so as to be consistent with generally accepted theoretical developments to date (see for example R. J. Kennedy 1958; Pariset et al 1966; Uzuner and J. F. Kennedy 1976). Some atempts to formulate mathematically the propagation and thickening of an ice jam have already been made (Uzuner and Kennedy 1976; Mercer and Cooper 1977); these are considered of theoretical interest as the resulting models have not been tested against laboratory or field data.

Items (iv) and (v), that is, flow and stability conditions at the jam toe are, to a large degree, unknown. For example, Uzuner and Kennedy (1976) did not attempt to solve their time-dependent equations largely because the downstream boundary conditions were unknown. On the other hand, Mercer and Cooper (1977) assumed a floating toe with equilibrium thickness which permits one to consider the water surface along the jam an M2 curve. Though floating toes have been observed occasionally, grounded toes seem to be commonplace (Beltaos 1980). Evidence for the latter situation can be either direct (water surface located farther below the top of the jam than one tenth of the available channel depth) or indirect (mode of failure of an ice jam, locally very steep slope of water surface). For the second ice jam at Fort McMurray in 1979, the results of Doyle and Andres indicate a toe slope of 0.005[3] over a distance of 500 m; this is 26 times the normal channel slope at the same location. To withstand the resulting forces (streamwise weight component plus bottom shear stress) an ice jam would have to be much thicker than the available flow depth.

When a jam toe is grounded, the downstream boundary condition may be formulated in terms of a seepage-type equation which relates the discharge to the water depths upstream and downstream of the grounded portion. If it is assumed that at the time of formation of the toe, the flow is stopped completely, i.e. discharge becomes zero momentarily, the upstream depth will subsequently increase and the downstream depth will decrease; this will establish a hydraulic gradient which, in turn, will cause the discharge to increase. This concept could be formulated mathematically and incorporated in an overall model of the jam formation process; however, there is an additional consideration that requires investigation. As the hydraulic head across the jam toe increases, the seepage force also increases while the jam's ability to resist this force may decrease if increased water stages cause partial floatation of the grounded ice. Therefore, there must be a limit of stability beyond which the jam would fail and move downstream but it is not known how a pertinent criterion should be expressed quantitatively. It would thus appear that research on the mechanics of grounded jams is necessary before a complete model of ice jam formation processes can be produced.

SUMMARY AND CONCLUSIONS

The results of a preliminary investigation into the mechanics of surges due to ice jam releases have been reported in the previous sections. The

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[3]Note that similar toe slopes for ice jams near Fort McMurray have also been reported regarding the 1977 and 1978 breakup periods (Doyle 1977; Doyle and Andres 1978).

investigation was prompted by a recent report (Doyle and Andres 1979) that includes a partially documented case of ice jam release.

The differential equations for the ice-water flow that occurs subsequent to the release of an ice jam were formulated and it was shown that, with plausible approximations, the problem may be treated as one-dimensional, open-water flow of identical total depth H and average velocity V. This applies to situations where the river is free of ice downstream of the released ice jam. Though this does occur in nature occasionally, the downstream reach is often covered with an undisturbed or deteriorated ice sheet. Arrival of the surge lifts, breaks and sets in motion this ice sheet; this phenomenon is too complex to model but its main effect is to retard the advance of the surge. For practical purposes, it was assumed that this effect may be handled by an increased friction factor or a reduced ratio V/V_*.

The data provided by Doyle and Andres pertaining to the release of an ice jam on the Athabasca River above Fort McMurray were reprocessed to define the initial and boundary conditions necessary for the computation. Stream geometry was defined on the basis of several surveyed cross sections; each cross section was approximated by a rectangle of average width and depth for simplicity. The computation was carried out by means of an algorithm developed by Krishnappan and Snider (1977) for unsteady, one-dimensional, open-water flow. This algorithm uses a constant value of V/V_* which, in view of previous comments, appears to be the weakest assumption of the present study. The value $V/V_*=9$ was found to adequately reproduce available stage and velocity estimates at a downstream location. This value is between the corresponding open-water value (16) and the apparent value (5) for flow under a solid ice sheet.

Using $V/V_*=9$, it was found further that, if the jam had been of the same maximum depth but longer than the one that actually occurred, the peak surge stages and durations of surge velocities would increase.

From the data of Doyle and Andres (1979), it appears that the surge was arrested at a location no more distant than 14 km below MacEwan Bridge and the present computation cannot be expected to be realistic for $T > 70$ min due to changed downstream conditions. Preliminary considerations on mathematical modelling of jam reformation indicated that the major unknowns are the flow and stability conditions at the toe of an ice jam, especially in cases where the toe is grounded.

ACKNOWLEDGMENTS

River cross sections upstream of MacEwan Bridge were provided by M. Anderson of the Transportation and Surface Water Engineering Division of Alberta Research Council. Unpublished gauge data were made available to the authors by the Calgary office of the Water Survey of Canada. Review comments by T. M. Dick and Y. L. Lau are appreciated.

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