
Investigation of River Ice Cover Formation Processes at Freeze-up

by

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Abstract

The formation of a river ice cover at freeze-up has several implications on society. In fast flowing rivers, the formation of an ice cover is highly dynamic; frontal progression is interrupted by sudden collapses which cause the cover to mechanically thicken. The attendant flow restriction can lead to rapid flooding and threats to human safety. Thick, rough ice covers also limit the energy production of hydroelectric generating stations through the winter season. It is important for engineers to understand the processes affecting ice cover formation at freeze-up, and the associated impacts on river hydraulics.

This thesis investigates this topic by way of field monitoring, laboratory experiments, and numerical modeling. A field monitoring program was established on the Dauphin River in central Manitoba, and comprehensive datasets were obtained for three freeze-up periods from 2015-2018. The data and observations constitute one of the most detailed freeze-up monitoring programs to date, and aid in the understanding of the processes of ice cover progression and consolidation. Laboratory experiments were conducted to investigate the effect of freezing air temperatures on the consolidation resistance of a floating rubble ice cover; the link between thermal and dynamic ice processes is not well understood, and is often neglected in numerical models. New formulations were added to the CRISSP2D model to account for effects of thermal strengthening on mechanical thickening of a forming ice cover. The CRISSP2D model was applied to the case study of the Dauphin River to evaluate its ability to simulate ice cover formation and associated water levels, and to highlight potential areas for future model development.

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Nomenclature

a	dimensionless coefficient relating average water depth to water surface elevation above a known datum
B	channel width
c	empirical coefficient in calculation of atmospheric emissivity
C	cloudiness of sky measured in tenths
C	celerity of surge wave
C_i	cohesion between ice cover and river bank
C_l	celerity of surge wave leading edge
C_o	coefficient relating bank shear stress to average vertical effective stress in an ice cover
C_p	celerity of surge wave peak
c_a	wind drag coefficient at ice surface
c_w	water drag coefficient at underside of ice cover
d	empirical coefficient in calculation of atmospheric emissivity
e	vapour pressure
F	maximum internal force per unit width on ice parcel
$\vec{F}_a$	force due to wind shear
F_f	friction force at model land boundary
F_N	normal force to land boundary
$\vec{F}_w$	force due to water shear
G	shear modulus
$\vec{G}$	force due to gravity
g	gravitational acceleration
H_{ia}	linearized heat transfer coefficient at the ice-air interface
H_{wa}	linearized heat transfer coefficient at the water-air interface
H_{wi}	linearized heat transfer coefficient at the water-ice interface
h	average water depth

h	sheet ice thickness (Section 3.2)
h_{crit}	critical solid crust thickness in CRISSP2D model
h_{crust}	thickness of the solid crust layer
h_i	level ice thickness
h_{por}	thickness of the porous ice layer
h_s	snow depth
K	elastic modulus
K_p	passive earth pressure coefficient
K_x	ratio of maximum streamwise stress to average vertical stress in an ice cover
k_0	ratio of shear to normal stress on an ice cover at the bank
k_1	ratio of lateral to streamwise stresses in an ice cover
k_i	thermal conductivity of ice
L_i	latent heat of fusion of ice
l	smoothing length used in SPH method
M	mass density of ice parcel
m	dimensionless coefficient to modify variation in celerity along surge waveform (Section 2.3)
m	mass of ice parcel
N	areal concentration of ice parcel
n_b	Manning roughness of channel bed
n_c	composite Manning roughness
n_i	Manning roughness of underside of ice cover
P_0	vertical buoyant stress
P_r	horizontal hydrostatic pressure
p_j	porosity of ice rubble
Q	water discharge
q	water discharge per unit width
q_o	water discharge per unit width of unperturbed flow
R	total resistance force per unit width
R	hydraulic radius

$\vec{R}$	force due to internal resistance
R_b	hydraulic radius of bed-affected portion of flow
R_i	hydraulic radius of ice-affected portion of flow
$\vec{r}$	position of ice parcel
S	cumulative degree-days of freezing
S_f	friction slope
S_o	water surface slope prior to passage of surge wave
S_w	water surface slope
s_i	specific gravity of ice
T_a	air temperature
T_m	melting temperature of ice
T_s	temperature at the ice surface
T_w	water temperature
t	ice cover thickness
t	time
t_i	ice parcel thickness
t_{intvl}	coupling time interval in CRISSP2D
U	average streamwise water velocity
$\vec{V}_i$	velocity of ice parcel = $u\hat{i} + v\hat{j}$
$\vec{V}_w$	water velocity vector
v_{crit}	critical ice parcel velocity in numerical model
W	interpolation kernel used in SPH method
$\vec{W}$	wind velocity vector at 10 m height
y	water surface elevation
y_o	water surface elevation prior to arrival of a surge wave
y_p	peak water surface elevation during passage of a surge wave
α	coefficient in Stefan's Law
α_i	albedo of ice cover
β	stability number for juxtaposed ice covers
γ_i	specific weight of ice

ε	strain
$\dot{\varepsilon}$	strain rate
ζ_v	bulk viscosity
η	water surface elevation
η_v	shear viscosity
λ	seepage flow coefficient
μ	strength coefficient for ice cover
ξ_a	clear sky atmospheric emissivity
ξ_c	cloudy sky emissivity
ξ_s	surface emissivity
ρ	density of water
ρ_a	density of air
ρ_i	density of ice
σ	Stefan-Boltzmann constant ($5.67\text{e-}8 \text{ W/m}^2/\text{K}^4$)
$\sigma_{1,2,3}$	principal stresses
σ_C	convergent stress
σ_{crust}	compressive strength of the solid crust layer
σ_D	divergent stress
σ_{ij}	normal stress ($i = j$) or shear stress ($i \neq j$)
$\bar{\sigma}_x$	average streamwise effective stress
$\bar{\sigma}_y$	average vertical effective stress
$\bar{\sigma}_z$	average lateral effective stress
τ	dummy variable used in calculation of cumulative degree-days of freezing
τ_b	bed shear stress
τ_{cons}	consolidation force per unit area of ice cover
$\bar{\tau}_i$	average shear stress on underside of ice cover caused by flowing water
τ_{xz}	shear stress between ice cover and river bank
ϕ	internal friction angle for rubble ice
ϕ^*	net heat flux out of the water body
ϕ_L	heat flux due to longwave radiation

ϕ_b	friction angle between ice and channel bank
ϕ_E	heat flux due to latent heat transfer due to evaporation
ϕ_H	heat flux due to sensible heat transfer
ϕ_R	heat flux due to shortwave radiation
ϕ_S	heat flux due to precipitation
ϕ_{wi}	heat flux at the ice-water interface
χ	constant used in linearized heat transfer approach

Chapter 1 Introduction

1.1 Background and Motivation

The presence of ice in rivers as air temperatures drop is a sure sign of the onset of winter in northern regions. The transition of rivers from open water to an ice-covered condition may seem relatively insignificant at first thought, but river ice has many societal implications. Many of these implications are positive; a smooth, competent ice cover allows for outdoor recreational activities such as ice fishing and skating. Ice roads constructed across rivers allow for more efficient transportation. A strong ice cover can also serve as a platform for off-shore construction activities. However, there are also several drawbacks to river ice. Ice covers can make ship transportation impossible without the use of icebreakers. Ice can create damaging stresses on hydraulic structures such as bridge piers and dams. River ice also negatively impacts hydroelectric operations; the reduced channel conveyance with the presence of an ice cover limits electricity production. Finally, and perhaps most tangible for Manitobans, river ice jams choke the river and create rapid water level rises that cause flooding and threats to human safety. As such, it is important for engineers to understand processes related to river ice cover formation so that they can better predict and/or mitigate negative consequences of river ice.

The characteristics of a river ice cover are inherently related to the conditions that exist as a river transitions from open water to an ice covered state. Meteorological conditions determine the rate of cooling of the water and, together with hydraulic

conditions, define the type of ice formations that develop. The following sections summarize the major processes that occur at the onset of winter, and their impacts on the formation of a river ice cover.

1.1.1 Ice Formation

Ice formation in any water body is facilitated by the net loss of heat from the water to its environment. Heat can be gained and lost through the water-bed interface or through the water-air interface; however, the energy exchange is typically dominated by the latter. Heat exchange at the water-air interface is governed mainly by shortwave radiation, longwave radiation, sensible heat flux, and energy exchange due to precipitation and evaporation processes. By quantifying each of these components, one can solve the full energy budget equation to determine the net heat flux out of a water body, as shown in Equation [1.1].

$$\phi^* = \phi_R + \phi_L + \phi_H + \phi_E + \phi_S \quad [1.1]$$

where ϕ^* is the net heat flux out of the water body, ϕ_R is the heat flux due to shortwave radiation, ϕ_L is the heat flux due to longwave radiation, ϕ_H is the heat flux due to sensible heat transfer, ϕ_E is the heat flux due to latent heat transfer due to evaporation, and ϕ_S is the heat flux due to precipitation. Other modes of heat transfer such as the frictional heat flux and energy exchange with bottom sediments can be added to Equation [1.1] if deemed significant for a particular location. As applying a full energy budget to a water body is often impractical due to data constraints, a common alternative is to assume that the heat flux at the water-air interface is a linear function of the difference in water and air temperatures, shown in Equation [1.2].

$$\phi^* = H_{wa}(T_w - T_a) \quad [1.2]$$

where H_{wa} is the linearized heat transfer coefficient, T_w is the water temperature, and T_a is the air temperature. The value of H_{wa} is site specific and can be determined with measured data. Values of H_{wa} for a given site may vary due to effects of wind speed and other factors, but typical values are in the range of 15-25 W/m²/°C (Prowse, 1995). During open-water conditions prior to freeze-up, the main heat fluxes are caused by sensible and latent heat transfers, as outgoing longwave radiation approximately cancels out incoming shortwave radiation over the course of one day (Ashton, 2013). As the onset of winter brings colder air temperatures, the rate of heat transfer out of the water body increases and causes water temperatures to decrease. When the water temperature drops below the freezing point (0°C for freshwater), ice begins to form.

Border ice is often the first type of ice to form on a river. The shallow water near the banks of a river promotes low water velocities and the presence of a thin, supercooled layer near the water surface (Clark, 2013). Ice grows laterally from the banks in thin sheets that are roughly parallel to the direction of flow. In calm rivers, the velocity in the middle portion of the channel may be sufficiently low for a similar process to occur; a thin layer of supercooled water forms at the surface, allowing the growth of ice particles. If the vertical turbulence intensity of the flow is low enough to allow the ice particles to remain at the surface, they continue to grow laterally to form skim ice (Clark, 2013). Skim ice can be transported downstream with the flow, which is called a skim ice run. If hydraulic and meteorological conditions are favourable (low turbulence and high rate of heat transfer at air-water interface), a stationary skim ice cover can form over a river or lake in a single night.

In turbulent supercooled flow, frazil ice will form rather than skim ice. Frazil ice typically appears as discoids with a diameter less than 1 mm and thickness varying between 0.5 mm and 0.05 mm (Ashton, 1986). Frazil ice can be entrained and remain suspended in the flow if vertical turbulence forces overpower the buoyant forces on the frazil particles. While the water remains supercooled and frazil production continues, the ice particles are in an active state. Active frazil ice is very sticky, and can adhere to the channel bed to form anchor ice or to other frazil particles to form porous frazil flocs that can range in size from approximately 5-100 mm (Prowse, 1995). As flocs grow, the buoyant force acting upon them increases, making them less influenced by vertical turbulent eddies. Buoyant frazil flocs reach the water surface and are exposed to the freezing ambient conditions. At this stage of development, the frazil ice particles are still only loosely connected to each other and can easily be broken apart by hand. This type of ice is termed frazil slush (Prowse, 1995), and is often compared to the consistency of a Slurpee drink. Once at the surface, flocs can combine and grow in size. Water in the interstices of the frazil particles freezes, creating a solid ice layer at the surface. At this stage, the ice forms are more competent and are termed frazil pans. Frazil pans collide with each other as they are transported downstream and take on a roughly circular shape with upturned edges. Circular frazil pans are sometimes referred to as pancake ice. Frazil pans remaining in close contact at the water surface can freeze together, forming larger competent surface ice formations known as ice floes. Various forms of frazil ice are often observed together at the water surface early in the freeze-up period, shown in Figure 1.1.



Figure 1.1. Aerial photograph of frazil slush, pans, and floes forming during freeze-up on the Dauphin River. View is looking downstream. Photograph taken on the Dauphin River on December 14, 2016.

1.1.2 Formation of a River Ice Cover

As mentioned previously, slow-moving rivers can develop a thin, skim ice cover overnight if conditions are favourable. Conversely, fast-moving rivers have sufficient water velocities that prevent a stationary skim ice cover from forming over the entire river width. Instead, skim and frazil ice pans are transported downstream with the flow. Once ice is generated, major engineering issues can result as subsequent dynamic processes cause ice floes to jam and create massive accumulations that obstruct the river. Hereafter, the terms “jam” and “ice cover” will be used interchangeably to describe accumulations of ice pans at freeze-up. In the most general sense, an ice cover is initiated when ice moving downstream is impeded by some means. If the ice transport capacity of a river is exceeded, congestion can occur as ice floes bridge across the river and become unable to move

downstream. Congestion of ice floes is exacerbated by natural channel constrictions and meanders, or, during freeze-up, by lateral growth of border ice which effectively reduces the top width of the channel. In practice, field data are relied upon to predict likely bridging locations. Alternatively, if a river culminates at a lake or hydroelectric reservoir with a stable ice cover, incoming floes may be halted as they contact the competent, thermally grown ice.

Once ice transport is stopped, incoming ice floes can build up and the accumulation will begin to progress upstream if hydraulic conditions permit. The division between free-drifting ice floes to a stationary ice cover is often called the ice front, or leading edge of the ice cover. Incoming ice floes can either remain at the surface or submerge and be transported underneath the cover. Submergence typically occurs by overturning, but can also occur by sinking for very thick or very thin floes (Dow Ambtman et al., 2011). Submerged blocks may be transported and resurface downstream, or, if the under-ice water velocity is not sufficiently high, may be deposited on the underside of the cover. If hydraulic conditions allow incoming floes to remain at the surface, a surface jam is established. The cover quickly progresses upstream through juxtaposition and surface packing of incoming ice floes and does not thicken appreciably. Typically, this type of ice cover has a hummocky appearance, and the frazil pans and floes comprising the cover are still identifiable as shown in Figure 1.2. Surface jams can shift downstream intermittently as they grow in length. Shear lines develop between the surface jam and the bank, where competent border ice exists and may remain stationary, as shown in Figure 1.3.



Figure 1.2. Surface jamming of frazil pans and floes at the upstream region of a forming ice cover. View is looking upstream. Note the transition between ice cover and open water (ice front) at the top of the photograph. Photograph taken on the Lower Dauphin River on November 22, 2017.



Figure 1.3. Shear line between surface ice cover (right) and competent border ice (left). View is looking downstream. Photograph taken on the Lower Dauphin River on November 21, 2017.

Thickened jams, which consist of multiple layers of floes, have a greater effect on upstream water levels than surface jams due to the keel of the jam and the increased hydraulic roughness of the underside of the jam caused by the disorderly arrangement of floes. Thickened jams are classified as either narrow-channel jams or wide-channel jams based on the mechanisms by which they thicken (Pariset et al., 1966). Narrow-channel jams are thickened via the submergence and immediate deposition of incoming floes near the leading edge. Typically, a critical Froude number approach is used to determine if an ice floe will entrain or juxtapose, with the critical Froude number ranging from 0.06-0.12 (Michel, 1978). The thickness of narrow-channel jams is therefore controlled by the hydraulic conditions present near the leading edge of the cover. The thickness of wide-channel jams is governed by the flow shear stress, channel width and slope, and internal strength properties of the ice. Wide-channel jams mechanically thicken in response to water shear forces on the underside of the cover and the downstream component of the weight of the ice cover itself. Note that “mechanical” thickening refers to the macro-scale thickening of an ice cover as ice floes rearrange in response to applied forces, while “thermal” thickening refers to thickening of an ice sheet as water freezes due to heat exchange processes. Mechanically thickened ice covers consist mainly of ice floes that form a porous accumulation of discrete blocks, with water and slush filling the voids. The floes can consist of both solid ice and frazil slush, and are collectively called the rubble ice portion of the jam. The large accumulations of ice can occupy a significant portion of primary flow area, which decreases the conveyance capacity of the channel. The hydraulic regime is altered within the jammed reach itself, but also to considerable lengths upstream and downstream. As a jam forms, the keel (portion of the ice cover below the water

surface) and additional rough upper boundary decrease discharge underneath the jam, increasing water levels upstream as water goes into storage. The net result is an increase in water levels with respect to open water conditions under the same flow. A schematic representation of typical freeze-up river ice processes leading to the formation of an ice cover is shown in Figure 1.4.

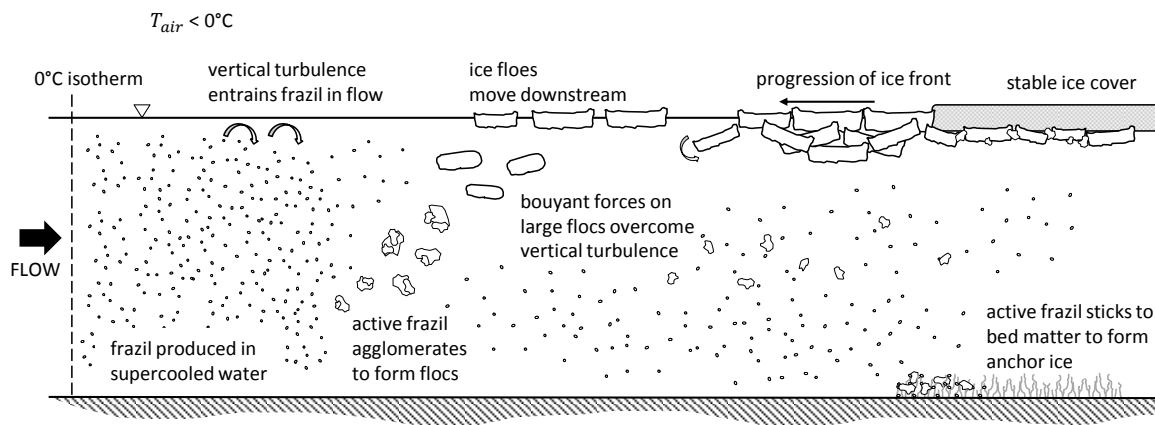


Figure 1.4. Illustration of the processes leading to the formation of a river ice cover at freeze-up.

1.1.3 Growth of Solid Crust Layer

Freeze-up jams are concurrent with cold air temperatures (below 0°C) that permit the continual transfer of heat from the water to the atmosphere. As an ice front is progressing upstream, a portion of the cover downstream will be stable and stationary. The stationary cover gains strength as the interstitial water between ice flocs at the surface freezes to form a solid crust that grows vertically through the rubble accumulation. The growth of the solid crust layer in a rubble ice cover is similar to the growth of a thermal lake ice cover, except that the solid ice is growing through a porous accumulation of ice blocks rather than only water.

The most basic analytical model for ice growth is Stefan's law (Stefan, 1891). This model is formulated around the idea that latent heat released by freezing water at the water-ice interface is conducted upward through the ice layer by a constant temperature gradient (Leppäranta, 1993). The simplest version of the model assumes: (i) no thermal inertia within the ice cover, (ii) no internal sources of heat, (iii) a known surface temperature, and (iv) negligible heat flux from the water. Assuming initially open water (i.e. no ice thickness at initial time), the analytical solution to the ice growth equation is expressed as Equation [1.3].

$$h_i = \alpha\sqrt{S} \quad [1.3]$$

with

$$\alpha \leq \sqrt{\frac{2k_i}{\rho_i L_i}} \quad [1.4]$$

$$S = \int_0^t (T_m - T_s(\tau)) d\tau \quad [1.5]$$

where h_i is the level ice thickness, S is the cumulative degree-days of freezing (also called the freezing index), k_i is the thermal conductivity of ice, ρ_i is the density of ice, L_i is the latent heat of fusion of ice, T_m is the melting temperature of ice (0°C for freshwater ice), T_s is the temperature at the ice surface, and τ is a dummy variable for the integration. In practice, the ambient air temperature is commonly used in lieu of the surface temperature for calculating S . This substitution, along with other errors inherent in the assumptions of Stefan's law, result in the use of a calibrated coefficient α that is typically in the range of 50-100% of its theoretical value (Leppäranta, 1993).

More reliable predictions of thickness with time can be achieved by considering various effects that are neglected in Stefan's law. First, the temperature at the underside of the ice layer is assumed to be at the melting temperature, T_m . By equating the heat flux through the ice layer to the heat flux from the surface to the atmosphere, and applying a linear heat transfer approach to the latter, the surface temperature, T_s , can be eliminated. The growth rate of an ice layer can then be calculated with Equation [1.6], following Ashton (1986).

$$\frac{dh_i}{dt} = \frac{1}{\rho_i L_i} \left(\frac{H_{ia}(T_m - T_a) - \phi_R}{\frac{H_{ia}h_i}{k_i} + 1} \right) \quad [1.6]$$

where H_{ia} is the linearized heat transfer coefficient at the ice-air interface. Equation [1.6] can be modified to include the insulating effects of a snow layer on the surface of the ice cover.

When a solid ice layer grows through an accumulation of rubble ice (ice fragments and blocks), the growth rate is increased due to the fact that only water in the interstices must freeze. Compared to the growth of a level ice sheet (in open water), the growth rate increases by a factor of $\frac{1}{p_j}$ (Ashton and Beltaos, 2013; Calkins, 1979), shown in Equation [1.7].

$$p_j \frac{dh_{crust}}{dt} = \frac{1}{\rho_i L_i} \left(\frac{H_{ia}(T_m - T_a) - \phi_R}{\frac{H_{ia}h_{crust}}{k_i} + 1} \right) \quad [1.7]$$

where p_j is the porosity of the rubble layer and h_{crust} is the thickness of the solid ice crust. Typical rubble ice cover porosities range from 0.4-0.6, thus the growth rate of the solid crust layer can be up to 1.7-2.5 times greater than the growth rate of a level ice sheet.

Beltaos (2013a) noted that freezing at the scale of voids within an ice cover is complex, and that Equation [1.7] represents the spatially averaged crust thickness over a large area. The solid crust layer is important to consider during the period of ice cover progression because it adds non-frictional resistance to collapse and mechanical thickening.

1.1.4 River Ice Cover Consolidation

The term “river ice cover consolidation” refers to the rapid collapse and mechanical thickening of a previously stable cover, which could be a surface jam or thickened cover from a previous consolidation event. Consolidation events often occur on fast flowing rivers during the progression of the ice cover at freeze-up. The continuous supply of ice generated upstream packs against the leading edge of the ice cover. As the cover lengthens, the external forces applied to it grow; the shear force of the water flowing beneath the ice cover and the downstream component of the weight of the ice are transferred in the downstream direction. Shear stresses caused by wind also act on the surface of the ice cover. If the strength of the ice cover is exceeded, the cover will collapse and the leading edge will “shove” downstream. The ice cover will mechanically thicken as the rubble ice from the previously stable cover rearranges to form a thicker accumulation with higher strength. The newly stable ice cover will again lengthen upstream as ice floes pack against the leading edge. The cycle of ice front progression and consolidation can occur several times over the freeze-up period. Consolidation events are unpredictable, and cause rapid changes to water levels. Upstream, water levels can drop rapidly as the flow resistance provided by the ice cover is suddenly removed. In the consolidated reach, the sudden mechanical thickening of the ice cover causes water levels to increase almost

instantaneously. The water released from storage during a consolidation event moves downstream in the form of a surge, temporarily increasing water levels and discharge.

Consolidation events are problematic because they often result in very thick, rough ice accumulations that constrict channel flow and cause flooding (see Figure 1.5). Furthermore, they occur rapidly and without warning, making it difficult to issue flood warnings or plan emergency evacuations until it is too late. Despite their significance, factors affecting the timing and severity of consolidation events are poorly understood. This is likely due, in part, to the lack of field data and first-hand observations of the conditions during consolidation events. Because of the knowledge gap, numerical models are not fully capable of simulating the process of ice cover consolidation, limiting their applicability and usefulness in predicting the freeze-up regime of a river.



Figure 1.5. Consolidated river ice cover on the Lower Dauphin River. Photograph taken on November 22, 2017. Note the rough surface caused by the disorderly arrangement of ice fragments comprising the cover.

1.2 Review of Pertinent Literature

1.2.1 Theory of Rubble Ice Strength

It has been established that mechanically thickened ice covers can constrict flow and cause increased water levels. Therefore, it is of interest to understand how the strength of an ice cover relates to its thickness. Note that in this context, “strength” refers to the resistance to mechanical thickening from forces applied in a plane coplanar to the ice surface, and not to the bearing strength which refers to the ability of an ice cover to withstand forces applied orthogonally to its surface. As the length of a thickened cover increases, the forces attempting to push the cover downstream, caused by water shear and gravity, increase. Ice rubble is traditionally thought to behave as a floating, granular material, with strength generated by frictional interlocking of ice fragments within the cover. This idea was first proposed by Kennedy (1958) with reference to the strength of floating pulpwood. Pariset et al. (1966) developed the concept that an ice cover is subject to collapse and mechanical thickening when the forces exceed the strength. As mentioned previously, they coined the terms “narrow-channel jam” and “wide-channel jam” based on the mode of thickening. They assumed that the strength of an ice jam formed from ice rubble is independent of the rheological properties of the ice itself. By considering the forces on an ice cover in a rectangular channel, they developed an equation relating the change in streamwise stress within the ice cover to the external forces. They suggested that the ice cover collapses and thickens when the streamwise stress exceeds the maximum internal stress of the ice cover. The form of this equation most often found in current literature is shown in Equation [1.8] (Beltaos, 1995a).

$$\frac{d}{dx}(\bar{\sigma}_x t) + \frac{2t}{B}(\tau_{xz}) = s_i \rho g S_w t + \bar{\tau}_i \quad [1.8]$$

where $\bar{\sigma}_x$ is the average streamwise effective stress, t is the ice cover thickness, B is the channel width, τ_{xz} is the shear stress developed between the ice cover and the bank (or grounded ice near the bank), s_i is the specific gravity of ice, ρ is the density of water, g is gravitational acceleration, S_w is the water surface slope, and $\bar{\tau}_i$ is the average shear stress caused by water flowing beneath the cover. This force balance is shown schematically in Figure 1.6. An assumption of Equation [1.8] is that the ice thickness is constant across the channel, which is a working hypothesis applicable to a one-dimensional case.

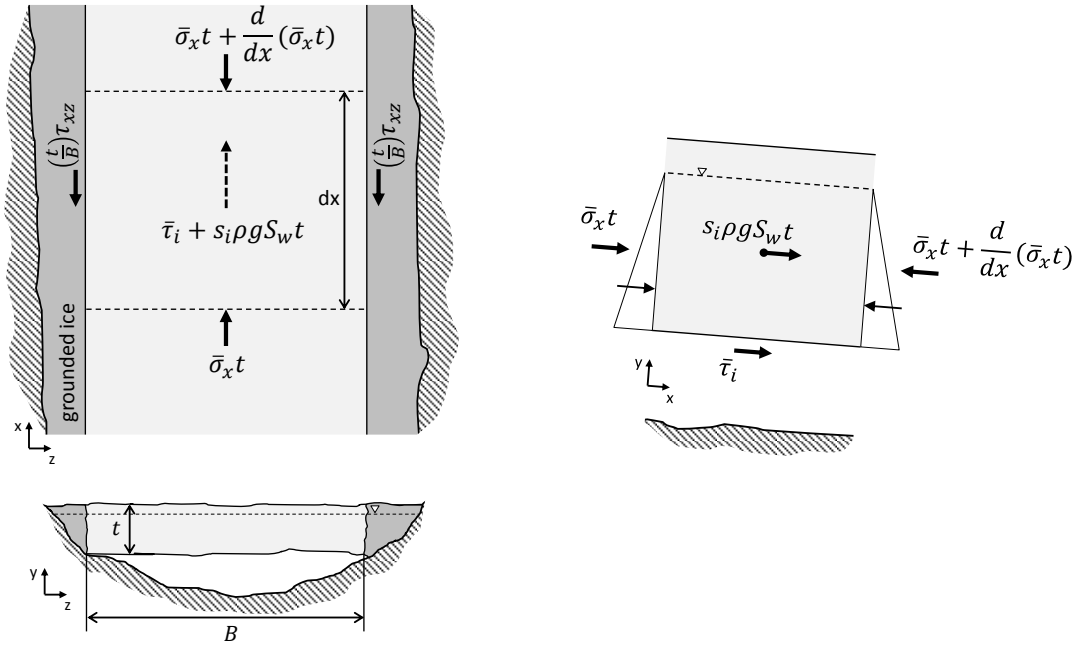


Figure 1.6. Forces on a section of ice cover in a rectangular prismatic channel. Adapted from Beltaos (1996).

Pariset et al. (1966) used theory of soil mechanics to develop equations for the limiting lateral and shear stresses acting on the cover, shown in Equations [1.9] and [1.10] (Beltaos, 1995a).

$$\tau_{xz} = C_i + k_0 \bar{\sigma}_z \quad [1.9]$$

$$\bar{\sigma}_z = k_1 \bar{\sigma}_x \quad [1.10]$$

where C_i is the cohesion between the ice cover and the bank, k_0 is the ratio of shear to normal stress applied at the bank, often assumed to be related to the friction angle of ice, ϕ , as $k_0 = \tan \phi$, and k_1 is the ratio of lateral to streamwise stresses in the ice cover. Furthermore, Pariset et al. (1966) assumed that the maximum streamwise compressive stress of the ice cover was governed by the confining stress caused by buoyancy through Equation [1.11] (Beltaos, 1995a).

$$(\bar{\sigma}_x)_{max} = K_x \bar{\sigma}_y = K_p \bar{\sigma}_y = \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \bar{\sigma}_y \quad [1.11]$$

where K_x is the ratio of streamwise to vertical stress at the point of failure, assumed to be equal to the passive pressure coefficient, K_p , which is a function of the angle of internal friction, and $\bar{\sigma}_y$ is the thickness averaged vertical stress caused by buoyancy, given by Equation [1.12].

$$\bar{\sigma}_y = \frac{1}{2} s_i (1 - s_i) (1 - p_j) \rho g t \quad [1.12]$$

The above analysis follows a Mohr-Coulomb failure criterion for the case in which the vertical stress caused by buoyancy is the least principal stress, which was shown to be a valid assumption by Beltaos (1995a). Beltaos (1995a, 1995b) noted that some assumptions made by Pariset et al. (1966) are not quite correct; for example, Equation [1.11] is only valid if the streamwise normal stress is a principal stress, which isn't the case as shear is developed within the cover in the xz plane. If it is assumed that t , S_w , B , and

$\bar{\tau}_i$ are constant in the streamwise direction (equilibrium jam condition), Equation [1.8] can be simplified to Equation [1.13] (Beltaos, 1995a).

$$\mu s_i(1 - s_i)\rho g t^2 = B(\bar{\tau}_i + s_i\rho g S_w t) - 2C_i t \quad [1.13]$$

where the strength coefficient, μ , combines all coefficients governing the stress relationships within the ice cover, defined by Equation [1.14].

$$\mu \equiv k_0 k_1 K_x (1 - p_j) \quad [1.14]$$

Uzuner and Kennedy (1976) published their own theoretical model of river ice jams a decade after Pariset et al. (1966), suggesting that the internal strength of a rubble accumulation is governed by two dimensionless coefficients, K_x and C_o , as in Equations [1.15] and [1.16] (Beltaos, 2010).

$$\bar{\sigma}_x = K_x \bar{\sigma}_y \quad [1.15]$$

$$\tau_{xz} = C_o \bar{\sigma}_y + C_i \quad [1.16]$$

where C_o relates the shear stress developed at the banks to the average vertical effective stress on the ice cover. Notice that Equations [1.15] and [1.16] are very similar to Equations [1.9] and [1.10] developed by Pariset et al. (1966), except that K_x is not assumed to be the passive pressure coefficient (therefore does not assume that $\bar{\sigma}_x$ is a principal stress), and the side friction depends on $\bar{\sigma}_y$ rather than $\bar{\sigma}_z$. Beltaos (1983) showed that the coefficients μ and C_o are related through Equation [1.17].

$$\mu = (1 - p_j)C_o \quad [1.17]$$

For the case of a prismatic channel, Equation [1.8] can be combined with an equation describing the hydraulic resistance of the underside of the ice cover to determine

the change in ice thickness with streamwise distance from the head (upstream end) of the ice cover (Beltaos, 1995a; Uzuner and Kennedy, 1976). This results in a jam configuration with three distinct regions: upstream transition, equilibrium reach, and downstream transition, as shown in Figure 1.7 (Beltaos, 2013a).

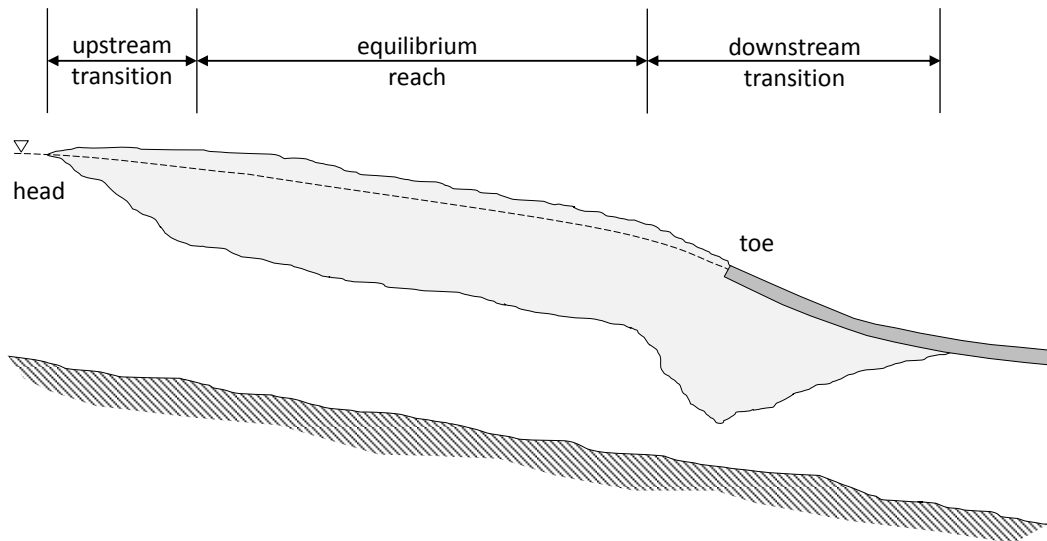


Figure 1.7. Illustration of an ice cover configuration with an equilibrium reach. Adapted from Beltaos (2013a).

In a natural river, an equilibrium reach will eventually form if ice is continuously supplied from upstream. This occurs when the ice cover becomes sufficiently thick such that the gradient in average streamwise stress (first term in Equation [1.8]) approaches zero. In other words, for a given section of the ice cover (as in Figure 1.6), the entirety of the forces directed in the downstream direction are taken up by friction generated at the banks. Distinguishing properties of the equilibrium reach are that the water surface slope and bed slope are equal, and the ice thickness is constant in the streamwise direction. Once formed, additional ice supply from upstream simply lengthens the equilibrium reach without changing the maximum water depth or length of the transition regions (Beltaos, 2013a).

The water depth within the equilibrium reach represents the maximum possible depth that can result from the formation of a wide-channel jam. Therefore, engineers often solve for the equilibrium jam thickness to assess flooding potential rather than calculate the full profile of the ice jam.

Pariset et al. (1966) analyzed field measurements of freeze-up ice jams on the Beauharnois Canal in Quebec and determined the best fit value for the coefficient μ was approximately 1.28. Note that this corresponds to a value of 2.13 for C_o assuming a jam porosity of 0.4. Further, they calculated the product of $C_i t$ to range from 1.1-1.3 kN/m. Beltaos (2013a) demonstrated that this finding implies that the value of C_i would have ranged from 0.3-20 kPa and is inversely related to the cover thickness, which does not seem physically plausible. Beltaos (2013a) also noted several laboratory tests on accumulations of small ice blocks that indicated values of C_i of 0.1 kPa or less (e.g. Keinonen and Nyman, 1978; Merino, 1974; Urroz-Aguirre, 1988). Owing to the difficulties in safely taking measurements of ice jam thickness, Beltaos (1983) developed an indirect method to compute the strength coefficient μ from water level records. In the analysis of 12 breakup jams he found that μ varied between 0.6 and 3.5, but most of the computed values were within the range of 0.9-1.3. It was also determined that the cohesive intercept, C_i , was negligible. In studies where ice thickness could be measured in breakup jams, μ was found to be in the same range of 0.9-1.3 (e.g. Beltaos, 1988, 1984; Neill and Andres, 1984). Because of the apparent negligible cohesion found in breakup jam studies, it is possible that the non-frictional resistance identified by Pariset et al. (1966) was not true cohesion, but was instead the effect of the freezing of a solid ice crust at the surface of the ice jam (Michel, 1978). Freeze-bonding between adjacent ice blocks in contact could also have

this effect (Schaefer and Ettema, 1985). This suggests that, unlike at breakup where the internal strength is produced mainly by the frictional interlocking of ice blocks (which can be represented by an appropriate value of μ), freezing effects occurring at freeze-up could have significant implications on the strength of the ice cover and its ability to resist mechanical thickening (which should not be modeled by a single value of C_i , but rather depends on the air temperature and the cumulative amount of freezing that has occurred).

The strength parameter K_x which relates the vertical effective stress caused by buoyancy to the maximum streamwise effective stress that an ice jam can withstand (see Equation [1.15]) has been found to be in the range of 10-12 through model applications to equilibrium and non-equilibrium breakup jams (Beltaos, 2010). There is no information on the value of K_x for freeze-up jams, but since the coefficient only influences the transitional reaches and not the equilibrium reach, Beltaos (2013a) has suggested using a value in the range of 10-12 as a working hypothesis. The value of k_1 is typically assumed to be 0.33 (Tuthill and Mamone, 1998; White, 1999), but can vary in the range of 0.36-0.55 (Beltaos, 2010). Reported values of the internal friction angle, ϕ , have shown a wide spread. Values range from as low as 21° , calculated by Michel (1980) on data from 10 ice jams, to 76° determined by Timco and Cornett (1999) in laboratory tests (White, 1999). In a review of reported values of ϕ in the literature, Beltaos (2010) suggested that the internal friction angle is typically between 55.8° and 58.6° but can be lower depending on the assumptions made regarding the magnitude of the principal stresses and the side frictional resistance.

White (1999) reviewed ice jam literature and identified a group of variables important in the determining the thickness of an ice cover formed through shoving and

internal collapse, and the effect on channel hydraulics. A selection of these variables and their recommended ranges suggested by White (1999) are shown in Table 1.1. Note the wide range of possible values for each parameter.

Table 1.1. Important parameters affecting ice jams and recommended range of values. Adapted from White (1999).

Property	Case	Range of Values
Ice cover Manning roughness (n_i)	Freeze-up ice cover	0.01-0.06
	Freeze-up ice jam	0.02-0.10
	Breakup (thin ice)	0.02-0.10
	Breakup (thick ice)	0.035-0.15
Cohesion (C)	Freeze-up ice jam	960-1200 Pa
	Breakup ice jam	0-100 Pa
Coefficient of internal strength (μ)	Freeze-up or breakup	1.0-2.0
Angle of internal friction (ϕ)	Frazil	20-45°
	Rubble	40-60°
Porosity (p_j)	Freeze-up ice jam	0.35-0.45
	Breakup ice jam	0.35-0.8

1.2.2 Laboratory Investigations on the Strength of Floating Ice Covers

Several laboratory experiments have been conducted on ice rubble to evaluate its apparent resistance to deformation under a variety of conditions, and to assess the ability of theoretical equations in describing the behaviour of rubble ice. This section will summarize strength tests performed on floating ice rubble in laboratory settings. An emphasis is given to shear and compression tests, as these are the dominant failure modes of river ice under natural stresses.

Prodanovic (1979) used a direct shear test to measure the shear strength of saline ice rubble. The box was suspended in water so that the rubble could float to its natural level, and shear forces were exerted on one half of the box while maintaining a constant normal force. He found that the shear strength showed little dependence on the strain rate,

and that the shear strength increased approximately linearly with confining pressure. For rubble created from parent ice sheets with thicknesses of 19 mm and 38 mm, the friction angles were 47° and 53° , and the cohesive intercepts were 0.26 kPa and 0.58 kPa, respectively. Hellman (1984) also performed direct shear testing, but was unable to maintain a constant normal stress during the shearing. He did, however, identify three distinct modes of shearing of ice rubble. Firstly, the ice pieces will become more densely packed as the strain increases. Second, the shear stress continues to increase and peaks when the test sample dilates. The third mode is the continuous shearing of the rubble at large strains. As pointed out by Ettema and Urroz (1989), it is unclear in most shear box studies whether peak or continuous shear strengths were actually measured, which could contribute to some of the scatter seen in the reported values of strength parameters. Løset and Sayed (1993) noted that the stresses and strains within the sample are not uniform in a direct shear test; results are highly dependent on the apparatus and procedure and could lead to scatter in reported data. Instead, they used a bi-axial chamber with two moveable sidewalls that put the rubble in a state of plane-strain. It was found that a wide range of internal friction angles can be used to describe the failure envelope depending on the strain path.

Urroz and Ettema (1987) performed strength tests on floating ice rubble with a simple shear apparatus that had plan dimensions of 530 mm by 609 mm. They investigated the effects of shear rate, rubble layer thickness, and size of ice fragment on the shear strength of a continuously deforming ice cover with the air temperature set to 0°C . They used three different sizes of ice blocks, as well as polyethylene blocks that were similar in size to the medium size ice blocks. The shear and normal force applied to the moving wall

of the simple shear box were measured during the tests. At low shear rates, distinct peaks could be observed in the force records. At high shear rates the exact point of failure was not as pronounced, and the forces were observed to oscillate around a mean value similar to the failure value. Urroz and Ettema (1987) observed that failure coincided with sudden relative motion of the ice rubble blocks. It was found that the shear and normal stresses at failure both decreased with increasing shear rate, and increased with increasing cover thickness. The polyethylene blocks showed no distinct variation in strength with shear rate, leading the authors to believe that freeze-bonding was causing this phenomenon in the ice block tests. In plotting the shear force and normal force at failure, Urroz and Ettema (1987) found that the cohesive intercept was very close to zero for all block sizes. This finding suggests that a continuously deforming ice cover behaves as a cohesionless material. The average internal friction angles were found to be 36.6° , 51.6° , and 30.5° for the small, medium, and large ice blocks, respectively. The larger friction angle for the medium blocks was attributed to sharper edges at the corners compared to the small and large blocks.

Tatinclaux et al. (1977) performed tests to determine the compressive strength of a fragmented ice cover. They used an insulated tank that was 0.9 m wide and 0.6 m deep, with a floating mass of ice rubble confined in the lateral direction. A plate attached to a variable speed carriage supplied the compressive force, and the force and displacement were logged for each test. Three different sizes of ice floes were used to create the rubble. The total thickness of the rubble accumulation varied from 6.1-18.3 cm, and cover lengths from 0.3-1.8 m were tested. The air temperature during their experiments was 0°C . The authors considered the first maximum recorded force peak as the failure strength, but noted

that significant variation in this failure strength was observed for nominally identical test conditions, with larger variation for tests with larger rubble blocks. Therefore, they performed at least 10 trials for each test condition and reported the average values of the failure strength. In plotting the stress-strain curves, Tatinclaux et al. (1977) found that the compressive strength and yield strain both decreased as the plate velocity increased. They concluded that this was caused by melting and refreezing at the particle interfaces under pressure; lower deformation rates will allow a longer time for this process to occur, resulting in stronger freeze-bonds. They also concluded that the compressive strength was independent of the cover length and the rubble floe shape, although they commented that the latter finding was an unexpected result. They suggested using a wider range of jam thickness and floe sizes to investigate this further.

Cheng and Tatinclaux (1977) conducted experiments in the same facility as Tatinclaux et al. (1977), and followed an identical experimental process. The goal of this set of experiments was to evaluate the effect of preloading on the compressive strength of an ice cover (however some tests with no preload were also performed). They used a single ice block size, consistent with one chosen by Tatinclaux et al. (1977). They also observed considerable variation in the compressive strength between nominally identical test conditions, and ran each test an average of eight times to account for this. The failure of the ice cover was marked by a drop in the force registered by the compression plate, and was found to occur in three different modes. For low plate speeds (less than 0.3 mm/s) the fragmented ice cover did not fracture, but a buckling type of failure was observed due to increased interparticle bonding. In this failure mode, the force on the plate plateaued and slowly decreased. For intermediate plate speeds, cracks were observed in the cover and

the failure was marked by a sudden drop in the recorded force. For high plate speeds (greater than 1.8 mm/s), the cover was observed to fracture near the driving plate. Following the initial drop in recorded force, the force on the plate fluctuated over a small range. Overall, the results indicated that the compressive strength increases as the cover thickness increases. The ratio of compressive strength to cover thickness decreases with increasing deformation rate, but attains a constant value of approximately 1570 N/m^3 for plate speeds larger than 3 mm/s. Unlike Tatinclaux et al. (1977), they found that for a greater cover thickness (15-23 cm), the yield strain increased linearly with increasing plate speed. For thinner covers, the yield strain decreased non-linearly with increasing plate speed. Cheng and Tatinclaux (1977) attributed this to the mode of failure in each case; higher ratios of cover length to cover thickness lead to a buckling type failure whereas lower ratios cause a crushing type failure. This finding suggests that a length to thickness ratio of less than 4.0 should be used to reduce the risk of buckling failure.

In comparing the results of Tatinclaux et al. (1977) and Cheng and Tatinclaux (1977) for similar cover thicknesses, large discrepancies exist (up to 50%), with the former reporting greater strengths than the latter (see Figure 1.8). This inconsistency in the results was attributed to slight deviations in air temperature from 0°C which affected the formation of the interparticle bonds near the surface. If small variations in air temperature truly caused such large changes to the compressive strength of the cover, presumably due to freezing effects, it is clear that this phenomenon should be investigated more closely.

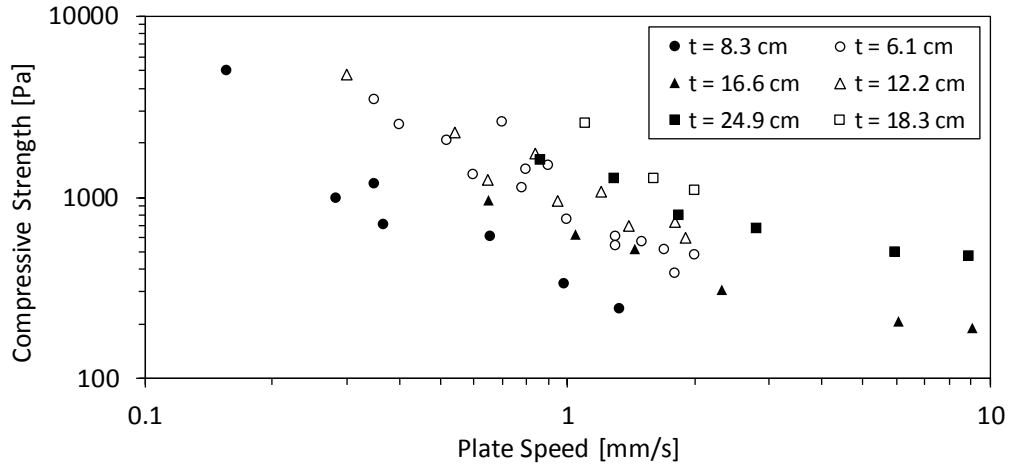


Figure 1.8. Influence of plate speed on compressive strength. Data from Cheng and Tatinclaux (1977) (solid markers) and Tatinclaux et al. (1977) (hollow markers).

Hopkins and Tuhkuri (1999) used a combination of physical experiments and numerical simulations to investigate the compressive strength of floating rubble fields. Ice floes cut into 400 mm diameter discs were floated in a rectangular channel measuring 6 m wide by 20 m long. Initially, the ice discs only covered approximately 70% of the water surface. A pusher plate was moved from one end of the channel to compress the ice floes and record the force required to deform the cover. Pusher speeds of 15 mm/s, 33 mm/s, and 52 mm/s, and ice thicknesses of 28 mm, 38 mm, and 54 mm were tested. A numerical discrete element model (DEM) was calibrated to replicate the results of the experiments, and was subsequently used to simulate various conditions (e.g. channel length and width, floe size, and friction coefficients), to evaluate their effect on the resistance of the rubble ice. Three distinct phases were observed during the movement of the pusher plate. First, the floating ice pieces were herded to one end of the channel and the surface concentration increased. The force on the pusher plate was low during this phase, arising mostly from the inertia of the accelerating floes. Next, the consolidated surface layer deformed by mechanisms of overturning and rafting. Thick or rough disks had a higher tendency to

underturn, while thin or smooth disks were more likely to raft. The force on the pusher plate increased gradually during this phase; the force was caused by the resistance to overturning and rafting, and from frictional contact with the channel edges. Third, when the entire initial surface layer of floes had failed, the jumbled field of floes continued to deform. Forces on the pusher plate increased more rapidly during this phase due to the thickening of the rubble layer. Computer simulations with the DEM showed that reducing the aspect ratio of the floes or increasing the friction coefficient increased the compressive resistance of the rubble layer. Increasing the channel width reduced the compressive resistance, as frictional effects from the channel edges became less important. Modifying the size distribution of floes did not have a significant impact on the resistance of the rubble layer.

In summary, several laboratory investigations have been conducted to test the strength of a floating rubble ice mass. Results generally agree with the Mohr-Coulomb theory, which is likely why it continues to be the accepted method of calculating the resistance of a rubble ice jam, and why satisfactory results can be achieved by applying the theory to real world ice jams. However, as noted by many of the studies, there are strain rate effects on the apparent strength of a rubble accumulation resulting from freeze-bonds that can quickly form at contact points between the ice rubble fragments. Further to this point, it is known that during the formation of an ice cover at freeze-up, the advancing ice cover goes through cycles of frontal progression and collapse. During the time that a portion of the ice cover is stationary, it is exposed to cold ambient air temperatures that freeze a solid crust at the surface of the rubble layer. There has not yet been a detailed laboratory investigation on the effects of a frozen surface crust on the ability of a rubble

ice cover to resist mechanical thickening, despite its known significance. Even in their seminal paper on the dynamics of river ice jams, Pariset et al. (1966) noted the primary importance of the relationship between meteorological conditions and mechanical thickening of an ice cover. They suggested that under extreme cold weather conditions, cohesion (or a non-frictional resistance term similar to cohesion) is at a maximum and allows the formation of thinner and more stable ice covers. Corroborating this notion, Pariset et al. (1966) noted that the most severe ice conditions in their study area in Montreal correspond to freeze-up periods with mild air temperatures.

1.2.3 Investigations on the Freezing of a Solid Ice Layer Through Ice Rubble

Section 1.1.3 provided an overview of theoretical equations to calculate the growth of a solid ice crust through a porous accumulation of ice rubble; this section will provide a review of literature relevant to this process, including laboratory and field studies.

The growth of a frozen layer through an accumulation of ice rubble has received more attention in sea ice literature than in river ice works. This is perhaps due to the fact that sea ice can grow much thicker than river ice, therefore allowing safe access for researchers. When sea ice ridges form, the keel is initially composed of cold broken ice rubble. Initially, the cold reserves in the ice rubble cause freezing throughout the entire volume, creating freeze-bonds at contact surfaces (Marchenko, 2008). This process is also known as sintering. Over time, thermal processes permit the growth of a solid crust layer vertically downward through the rubble. The solid crust layer is synonymous with the term “consolidated layer” in sea ice literature, but only the former will be used in this document for clarity; “consolidation” typically refers to the process of an ice jam shoving and mechanically thickening in river ice literature. Høyland et al. (2001) performed tests in a

model basin at the Hamburg Ship Model Basin and found that sintering during the initial phase was sensitive to the temperature of the level ice from which the rubble was made. The thickness of the solid crust layer and sintering effects are important from a sea ice perspective because they can influence the design loads of off-shore structures and ice breakers. As mentioned earlier, a very similar process occurs during freeze-up of rivers; competent frazil pans comprise a mass of ice rubble, and cold ambient air temperatures can form a crust that solidifies these pans together.

Leppäranta and Hakala (1992) investigated free floating first year ice ridges in the Baltic Sea and found that the solid crust layer within the ridges (average porosity 0.29) was 1-2 times the thickness of the surrounding level ice. This differential growth rate of solid crust layer to level ice has been identified in numerous other field studies on sea ice ridges (e.g. Croasdale, Allyn, and Marcellus (1990), Leppäranta et al. (1995), and Kharitonov (2008)). The growth rate of the solid crust layer can vary spatially over an ice ridge; Shafrova and Høyland (2008) performed mechanical drilling on first year ice ridges in the Barents Sea and Arctic Ocean in 2005 and 2006, and found that the maximum-to-average thickness ratio of the solid crust layer was around 2.1, while the minimum-to-average thickness ratio was around 0.36.

Timco and Goodrich (1988) investigated the rate of solid crust growth in freshwater and saline ice rubble in a laboratory setting. Deep rubble accumulations were created, and allowed to refreeze under ambient air temperatures ranging from -5°C to -18°C. Ice block size, ambient air temperature, and wind speed were varied to determine their effects on the solid crust growth rate. Thermocouple rakes were used to log temperature profiles in rubble ice accumulations to quantitatively estimate the location of the solid crust front. The

solid crust was allowed to grow through the rubble accumulation for durations ranging from 27-120 hours, resulting in solid crusts that were 10-28 cm thick. Visual inspections of the final location of the solid crust layer were compared with the temperature data. The authors found that the macro porosity and orientation of rubble blocks caused variability in the solid crust growth rates, and that the crust depths obtained with the thermocouples were at the lower end of the visually observed ranges of crust depths. The crust depth was found to be linearly related to the square root of the freezing index (product of time and temperature) and that good fits were obtained by using either the air temperature or the surface temperature in the freezing index calculation. In all cases, the surface ice temperature was measured to be warmer than the ambient air temperature. Based on best fit lines using the air temperature freezing index, the growth of the solid crust layer through the rubble was 1.7 times as quick as the growth of an adjacent level ice sheet.

Toyota and Koda (2016) examined the solidification of saline grease ice (sea ice term for frazil ice grown in turbulent conditions). An insulated tank was placed in a cold room with air temperatures ranging from -10°C to -20°C , and a stirrer promoted the formation of grease ice. A second identical tank was placed in the same room, but without a stirrer. Once the non-stirred tank grew a sheet of ice approximately 5 cm thick, the stirrer was stopped and the grease ice floated to the surface to create a layer approximately 10 cm thick. Measurements from a string of thermocouples spaced at 1 cm intervals were used to identify the rate of growth of the solidified layer through the grease ice. Porosity of the grease ice ranged from 0.27-0.44. Results showed that the solidification through the grease ice was rapid; the upper 4-5 cm of grease ice solidified within two hours after the stirrer

was stopped, and by the time the columnar ice grew to 10 cm thick, the solidified portion of the grease ice was also 10 cm thick.

Croasdale, Allyn, and Marcellus (1990) developed a numerical model (ICEGROW) to predict the solid crust growth through rubble ice. They considered the major sources of heat transfer to be sensible heat flux and net longwave radiation, and modified the thermal properties of the water such as latent heat of fusion and thermal conductivity based on the salinity. The air temperature was found to largely influence the growth rate, while wind speed was important up to a speed of 5 m/s (beyond which any change was less influential). The insulating effects of a snow cover were also evident, with a snow cover of 10 cm cutting the crust growth rate in half. The porosity of the ice rubble was also found to significantly affect the crust growth rate as was found by other authors. The numerical model was compared with field observations in the Beaufort Sea, but could not accurately predict the solid crust thickness of the ice rubble. The authors attributed this to a lack of data on the initial conditions such as rubble porosity and depth of snow cover. A more controlled experiment was performed in the Esso open-air ice basin where rubble ice was allowed to refreeze under atmospheric conditions. Using measured values of rubble porosity and sail height, the ICEGROW model simulated the growth of the solid crust layer reasonably well, although the authors did not include their method of measuring the depth of the solid crust layer. Croasdale, Allyn, and Marcellus (1990) concluded that a solid understanding of the rubble accumulation porosity is crucial to predicting its crust growth rate.

1.2.4 Strength of Solid Crust Layer

The one-dimensional resistance provided by an ice cover can be broken down into the resistance from the solid ice crust and the resistance from the porous ice accumulation. Using the equation for maximum internal streamwise stress proposed by Pariset et al. (1966) (Equation [1.11]), the total resistance per unit width can be calculated with Equation [1.18]. The porous layer resistance is expressed as the product of the strength coefficient, K_x , and the average vertical effective stress caused by buoyancy. Note that the porous ice resistance is a function of its thickness (i.e. a greater thickness increases the buoyant force, thus increasing the maximum possible resistance).

$$R = \sigma_{crust} h_{crust} + K_x \bar{\sigma}_y t \quad [1.18]$$

where R is the total resistance force per unit width, σ_{crust} is the compressive strength of the solid crust layer, and h_{crust} is the thickness of the solid crust layer.

In-situ field tests on freshwater ice performed by Vittoratos and Kry (1979) indicated that the uniaxial compressive strength ranged from 1.5-3.0 MPa depending on the temperature of the ice. Using typical values of 12 for K_x and 0.4 for p_j , representative a freeze-up ice jam (Beltaos, 2013a), and lower limit in-situ field measurements of the compressive strength of solid ice, Equation [1.18] reduces to Equation [1.19].

$$R = 1500 h_{crust} + 2.6 t^2 \quad [1.19]$$

where R is in $\left[\frac{\text{kN}}{\text{m}}\right]$ and h_{crust} and t are in [m]. It is evident that even with a substantial porous ice thickness, the solid crust resistance dominates the total resistance provided by the ice cover. Michel (1978) made a similar comparison, and also concluded that the solid crust dominates the total resistance. He noted that a frozen crust can grow and join the ice

pieces over a relatively short time during cold weather; therefore the solid crust resistance is a very important term to consider during freeze-up.

Andres (1999) examined the growth of the solid crust layer as it relates to the progression and collapse of a forming ice cover at freeze-up. He assumed that the internal frictional resistance of the ice pieces and shear resistance at the banks were negligible compared to the strength of the solid crust layer. The rate of growth of the externally applied forces was equated to the growth rate of the resisting forces due to freezing of the solid crust layer to obtain a stability number, β , that differentiated between a juxtaposing or consolidating ice cover, as shown in Equation [1.20].

$$\frac{|T_a|Bk_i}{QS_w\rho_iL_i} \geq \beta \quad [1.20]$$

where Q is the discharge upstream of the progressing ice cover. The stability number, β , is a function of several variables that include bed and ice cover roughness, strength of the solid crust layer, thickness and porosity of the juxtaposed cover and incoming ice floes, the celerity of the advancing ice front, and the heat transfer coefficient at the ice-air interface. Andres (1999) noted that for typical values of the aforementioned variables, β could range from 0.00002-0.001. Using data and observations of juxtaposed and consolidated freeze-up ice covers on the Peace River in northern Alberta, he deduced a value of 0.0003 for β , and back calculated the strength of the solid crust layer to be approximately 1.2 MPa, although the value could plausibly be in the range of 1-2 MPa based on the data provided. Andres (1999) noted that more work is required to investigate the failure mechanism of progressing ice covers at freeze-up, particularly the effect of planform channel characteristics such as meanders that likely limit the longitudinal forces transmitted

downstream and may promote the formation of a juxtaposed cover. Beltaos (2013a) substituted typical values for ice and water properties along with the value of 0.0003 for β to arrive at Equation [1.21].

$$\frac{|T_a|}{qS_w} \geq 0.42 \times 10^5 \quad [1.21]$$

where q is the unit discharge. Substituting in typical values for the Lower Dauphin River of 2 m²/s for q and 0.0016 for S_w shows that the air temperature would need to be impossibly low for a juxtaposed ice cover to develop. The equation can be more useful in determining the water surface slope at which point a juxtaposed cover could develop; at an air temperature of -10°C, S_w would need to be less than 0.00012.

1.2.5 Field Monitoring of Freeze-up Consolidation Events

Field investigations are a useful approach to study and understand the ice processes of a particular region; data collected over several years can be used to predict the peak ice elevations or estimate return periods of ice-related flooding events. Datasets collected through field monitoring programs can also be used to validate current theory on ice dynamics and effects on channel hydraulics. Detailed field measurements provide calibration and validation datasets for numerical models, increasing their accuracy and capabilities. Unfortunately, planning and executing a field monitoring program has many challenges. Northern rivers often run through remote locations, making access difficult. Cold weather is often unpleasant to work in, and adequate housing and shelter is required for multi-day monitoring. Instrumentation such as pressure transducers to measure changing water levels are sometimes challenging to install in the river during open water season. Further, dynamic ice processes can damage these instruments or move them from

their deployed location. Lastly, and perhaps most importantly, several sought after parameters are often unsafe or impossible to accurately measure. Ice jams are inherently unstable, and navigating on their surface to perform drilling for thickness or discharge measurement is dangerous. Field monitoring programs must be carefully planned out and executed, and even still, the data collected is often incomplete. Still, measurements of real-world ice jam properties is worth the effort, and can provide full scale information that is simply not reproducible in a laboratory setting.

Several field studies have been conducted to investigate breakup ice processes and dynamics (e.g. Beltaos et al., 1996; Beltaos and Carter, 2009; Beltaos et al., 2011; Beltaos et al., 2012). Field datasets most commonly include water surface elevations, either as surveyed instantaneous water surface profiles or as time series collected with installed pressure transducers, likely due to the relative ease of this measurement. Ice thicknesses are often inferred from water level rises or indirectly measured from shear wall remnants that remain attached to the banks after an ice jam has cleared. In general, breakup observations are consistent with traditional Mohr-Coulomb theory of ice jam dynamics, as evidenced by the relatively small range of μ found by Beltaos (1983).

Less attention has been given to freeze-up ice monitoring, perhaps because breakup ice jams can occur more rapidly and pose higher risk of flooding. Michel (1984) reviewed five datasets, collected on the St. Lawrence River (1947-1948, 1949-1950), Beauharnois Canal (1951-1952, 1953-1954), and La Grande River (1977-1978). Each dataset included ice thickness profiles and observations of the mode of ice cover formation. On the St. Lawrence River, it was found that shoves and “heavy shoves” corresponded with periods of warm weather which the author suggested caused thermal weakening of the cover and

subsequent collapse. Regions where shove events occurred had much greater ice thicknesses (in excess of 3 m thick in some areas) than regions where surface juxtaposition occurred (thickness of approximately 0.5 m). In the Beauharnois Canal, the ice cover was observed to form in 1951-1952 by frontal progression with minimal packing. In 1953-1954, some shoves and packing were observed to occur typically in the morning after a discharge increase from a nearby powerplant operation. In the La Grande River, the ice cover formation occurred during a period of continuously cold weather, and no shoves were detected. Michel (1984) concluded that, in general, shoves were prone to occur during a thawing period or an artificial rise in water level. He compared the shoving process to a breakup event, as the rise in temperature and/or rising water levels tend to weaken or destroy the solid crust layer and lead to collapse and mechanical thickening.

Andres (1995) summarized a variety of freeze-up data collected on the Peace River from 1970-1988 including water surface profiles, ice thickness measurements, an under-ice velocity profile, and descriptions of ice front celerities and characteristics. The average winter discharge increased to approximately four times the natural flows since regulation of the Peace River in 1972. The higher flows caused the shoving process to dominate ice cover progression, leading to higher freeze-up water levels and thicker, rougher ice accumulations. A typical water surface gauge record was presented, with several characteristics that are indicative of the ice cover formation sequence. First, the backwater effect of an ice cover progressing via surface juxtaposition and packing reaches a site and the level begins to rise. When the ice front passes the site, the water level plateaus at a temporarily stable level. A sudden increase in stage can occur if the ice cover collapses and shoves due to the thickening of the ice cover and the attendant surge of water released

from storage. The water level tends to decrease gradually from its peak level as the underside of the ice cover is smoothed and because a portion of the flow is being abstracted as the ice front continues to move upstream. Using measured water surface elevations, the internal strength coefficient, μ , was estimated for a range of flow conditions. It was found that at the lower range of discharges (less than 1200 m³/s), the value of μ ranged from 1.5-2.0 or higher. Andres (1995) suggested that the lower flows allowed the downward freezing of the solid crust layer to add strength to the ice cover, reducing the likelihood of shoves and increasing the apparent values of μ . At higher flows (greater than 1500 m³/s), the value of μ was in the range of 0.8-1.5 as the shoving process dominated ice cover formation. The author recommended that values of μ in the range of 0.8-1.0 should be used if the shoving process dominates. Additionally, he suggested that the quasi-steady winter discharge should be increased by 25% when employing the theoretical equations to account for the surges associated with dynamic ice events that can temporarily increase flow and shear forces on the forming ice cover.

Andres et al. (2003) noted that the triggering of consolidation events is not well understood, but three conditions that appear to promote their occurrence are (i) rapid advancement of the leading edge which increases stresses without attendant thermal growth of a surface crust, (ii) a sudden increase in discharge, and (iii) rapid air temperature fluctuations that can quickly weaken the solid crust and compromise the resistance of the ice cover. The authors further differentiated between “primary” consolidation events, which occur early in the ice cover formation before significant interstitial freezing develops, and “secondary” consolidation events, which are the collapse of an existing consolidated or juxtaposed ice cover and affect a larger reach of the river. Andres et al.

(2003) reviewed over 20 years of freeze-up data on the Peace River which included ice front celerities, air temperatures, discharges, and occurrence of secondary consolidation events. The celerity of the ice front generally increased with colder air temperatures, and was nearly zero when the air temperature was warmer than -10°C . They found that consolidation events occurred over the entire range of measured ice front celerities, and could be caused by any of the three conditions previously listed. The authors analyzed water level changes during consolidation events and found that the water released from storage during an ice cover collapse created unsteady flow at downstream gauges that temporarily increased discharge and shear stress. Additional details of this study are included in Section 2.3.2. Andres et al. (2003) concluded that consolidation events can significantly affect the flows in a river, and that numerical ice models are not currently capable of predicting the transient hydraulic properties caused by secondary consolidations.

In general, field observations during freeze-up have shown that shoving events can lead to very thick ice accumulations that can severely constrict flow and lead to staging that can cause flooding. Furthermore, dynamic ice formation processes have been shown to cause unsteady flow conditions that may apply greater stresses to a forming ice cover than may be expected from the (nearly) steady winter discharge. While past research efforts have certainly been fruitful and have resulted in a solid understanding of these processes, additional field datasets are required to fill in the knowledge gaps. In particular, a detailed field monitoring program with measurements of water levels and descriptions of ice cover characteristics at a high spatial and temporal resolution on a reach that is prone to consolidation events would be useful. First-hand observations of the progression of a

surface jam or the sequence of events occurring during a consolidation event would help to confirm and/or advance the ability to predict ice-affected water levels and flooding potential.

1.2.6 Numerical Modeling of River Ice

In the 1970s, the American Society of Civil Engineers published a paper highlighting the research needs regarding the study of river ice (ASCE Task Committee on Hydromechanics of Ice, 1974). Since then, great progress has been made in several areas of river ice research, furthering the state of knowledge on the subject. The concurrent advance in computing power has led to the development of several numerical models that can be used to simulate complex river ice phenomena and the effects on channel hydraulics. Numerical models are the most economical and efficient method to assess the impacts of river ice; the modeler can specify a suite of expected conditions and relatively quickly get a result from the comfort of the office. The properties and features of specific numerical river ice models are driven by the needs of the modeler; therefore, models vary in terms of complexity. The most basic models are steady and one-dimensional, and simply compute the effect of an ice cover on the water surface profile in a channel. Conversely, the most advanced models can simulate the thermal cooling of water, production of ice, unsteady dynamic interactions between ice pieces, transient effects on river hydraulics, and other relevant processes. All models have underlying assumptions and are limited in one way or another by the current state of understanding of river ice processes. The following paragraphs are intended to present a non-exhaustive selection of river ice models to provide context on the common traits of these models and areas where they differ.

The U.S. Army Corps of Engineers Hydrologic Engineering Center River Analysis System (HEC-RAS) model is a one-dimensional model that can perform both steady and unsteady open water computations (U.S. Army Corps of Engineers, 2002). The model employs the standard step method to solve the energy equation to determine water surface profiles and velocities along a reach. Channel roughness is defined with a Manning number that can vary along the reach or within the same cross section in the main channel and overbank areas. HEC-RAS can also account for energy losses associated with common hydraulic structures such as bridge piers and culverts. The model is publicly available for download, and includes a graphical user interface to facilitate model construction, calibration, and interpretation of results. The effects of river ice can be simulated in two ways. First, HEC-RAS can simulate the hydraulics of fixed ice covered reaches with user-prescribed ice roughness and thickness. Alternatively, the program can simulate the thickness of a wide-channel jam under steady flow by solving the force balance equation with the jam stability criteria developed by Pariset et al. (1966) and Uzuner and Kennedy (1976). The user can specify material properties of the ice jam such as porosity and internal friction angle at each cross section. The HEC-RAS model does not incorporate processes related to the thermal growth and decay of ice, or the transport and unsteady dynamics of jamming ice.

The SIMGLACE model (Rousseau, Sauve, Warren, Inc. and Hydro-Quebec, 1983), originally conceived in 1977, is a one-dimensional steady state model that simulates the processes of water cooling, ice generation, ice cover formation, and ice cover breakup. At each cross section, four stability conditions are used to define the ice thickness (Petryk, 1995): (i) frontal progression (narrow jam), (ii) shoving (wide jam), (iii) border ice

stability, and (iv) erosion of the ice cover. The user inputs bathymetry, bed roughness, and ice properties such as porosity. Forcing data includes discharge, meteorological conditions, and water temperature at the upstream boundary of the model. SIMGLACE calculates heat loss through consideration of major heat sources and sinks, or through a linearized heat transfer approach. When the water temperature reaches 0°C, the volume of frazil ice that forms is calculated using the latent heat of fusion of ice. The model can produce outputs such as mean water velocities, water levels, and ice cover thickness.

The River1D hydrodynamic model was developed at the University of Alberta (Hicks and Steffler, 1992). The model solves the mass and momentum equations with the Characteristic-Dissipative-Galerkin (CDG) finite element method. The model includes thermal and dynamic ice process modules. A steady state ice jam profile component, based upon stability equations that follow the theory of Pariset et al. (1966) and Uzuner and Kennedy (1976), is used calculate ice jam profiles and associated water surface profiles (She and Hicks, 2006). Andrishak and Hicks (2008) describe the thermal ice components which consider water cooling, frazil ice formation and floatation, surface ice transport, and thermal ice growth and melt. She et al. (2009) describe another River1D model component which simulates the dynamics of ice jam formation using a purely Eulerian reference frame. Mass and momentum conservative equations for both water and ice are solved using the CDG finite element method in an uncoupled sequence. A new constitutive model was implemented by She et al. (2009) to determine the internal resistance within an ice jam based on the strain rate of the deforming ice. The River1D model can be used to simulate the configuration of an ice jam and corresponding water surface profile for unsteady flow conditions, such as those experienced during hydro-peaking operations (She et al., 2012).

The RICE model was developed by Hung Tao Shen and his collaborators at Clarkson University in the 1980s. The river hydraulics component of the model solves the one-dimensional unsteady flow equations (Lal and Shen, 2007) using a four-point implicit finite difference scheme (Petryk, 1995). The RICE model includes additional components to simulate water temperature and ice concentration, formation of border, skim, and frazil ice, undercover transport and accumulation (based on critical velocity criteria), and thermal growth and decay of an ice cover. A unique feature of the RICE model is that the transport of ice is considered to occur in two layers: a surface layer and a suspended layer. The model considers the surface juxtaposition of an ice cover using the critical Froude number criterion; when the critical Froude number is exceeded, the cover progresses in the hydraulic thickening (narrow jam) mode as defined by Pariset et al. (1966). The shoving mode for wide-channel jam formation is also considered by solving the force balance equation based on the theory of Pariset et al. (1966) and Uzuner and Kennedy (1976). The model also includes a stability criteria for ice covers that includes contributions of the rubble (granular) ice, frazil ice, and solid crust layers that can be used to simulate the onset of mechanical breakup. However, this component has not been tested in field applications. An advanced version of the model, RICEN, was based upon the RICE model. The major improvements include (i) the ability to simulate the supercooling phenomenon and anchor ice formation, (ii) computation of undercover transport based on a transport-capacity formulation, and (iii) inclusion of wind effects, artificial ice breaking, and flow resistance due to the moving ice (Shen et al., 1995).

The ICESIM computer model was developed in 1973 by Acres International Limited to study river ice problems, and has been under continual development since then.

It has been applied to hydroelectric design problems on the Nelson River in northern Manitoba, as well as to problems involving ice jams on several other Canadian Rivers (Carson and Groeneveld, 1997). The ICESIM model is one-dimensional and can simulate steady state flow conditions and the effects of river ice on backwater surface profiles. The model computes the daily volume of ice entering the domain using heat transfer theory and mean daily temperatures, or the user can simply input the ice volume (Petryk, 1995). Border ice is calculated using the empirical formula proposed by Newbury (1968) or by a user-defined open water fraction. Ice cover progression through surface juxtaposition and hydraulic thickening are considered with a critical Froude number criterion. Mechanical thickening through shoving is handled in the model by numerically integrating the external forces and internal stressing along the cover, and allowing the ice cover at a given section to thicken such that it can just withstand the applied stress (Petryk, 1995). The computation is performed from upstream to downstream, and the total volume of ice required to thicken all unstable sections is calculated. This volume of ice is taken from the upstream portion of the ice cover, resulting in the recession of the leading edge. Another model, called ICEPRO, is very similar to ICESIM as Rick Carson was the principal author of both programs (Carson et al., 2011). ICEPRO includes some minor improvements to the shoving process over ICESIM, as well as the incorporation of a Manning roughness of an ice jam that is a function of its thickness. As an extension to the ICESIM model, a dynamic version called ICEDYN was developed which considers the transient effects of hydropower peaking operations and other varying river flows on the dynamics of river ice (Carson and Groeneveld, 1997). ICEDYN solves the St. Venant equations for hydrodynamics through a weighted four-point implicit finite difference scheme. The ice

mechanics modules are similar in form to ICESIM, and are loosely coupled with the hydrodynamics module.

Development of the RIVICE model began in 1988 through a collaborative effort between Environment Canada and several other government and private agencies (Lindenschmidt, 2017). RIVICE is a one-dimensional, comprehensive numerical model that can simulate dynamic river ice processes under a set of hydraulic, meteorological, and bathymetric data (Martinson et al., 1993). The model uses an implicit finite difference scheme and accounts for several processes related to river ice including water cooling, ice generation, transport, cover formation, shoving, thickening, erosion, melting, and breakup initiation. The RIVICE model can simulate the generation of skim ice or frazil ice in open water. Border ice growth can be calculated using a variety of methods, including a user-defined relationship between cumulative degree-days of freezing or the Matousek (1984) method. Ice cover evolution and breakup modules are implemented on regions with a stable ice cover, depending on the application. The fundamental premise of RIVICE is that the calculations of ice generation and evolution can be separated from the calculation of hydrodynamics if they are performed with a small enough time step (Environment Canada, 2013). This “loosely coupled” method is employed by several other numerical models to simplify the computational procedure.

The ICEJAM model was developed to compute the ice thickness and water surface profile over the entire length of a wide-channel jam with a floating toe (Flato and Gerard, 1986). The model uses the theory of Pariset et al. (1966) and Uzunur and Kennedy (1976) for calculating the thickness of a cohesionless, granular mass of ice. The program solves for the thickness profile of the ice jam using an approach similar to the one employed by

HEC-RAS. First, an ice thickness is assumed at the head of the jam and an estimated thickness profile is obtained using a forward difference scheme (calculations move from the head of the jam to the toe). The standard step method is then used to compute the depth at each cross section (calculations move from the toe to the head). Using the newly computed depth profile, the thickness profile is recalculated. This calculation procedure repeats until the solution converges. Considerations are also made for ice erosion using a critical velocity criterion. Seepage through the interstitial voids within the ice cover is neglected.

RIVJAM is a one-dimensional steady state model developed by Dr. Spyros Beltaos. It is applicable primarily to breakup ice jams, where thickness is governed by the balance between internal resistance and external loads (Carson et al., 2011). The thickness of the wide-channel ice jam is calculated based on the theory of Pariset et al. (1966) and Uzuner and Kennedy (1976). The thickness relationship is coupled with the hydraulic computations for water depth, resulting in a system of two differential equations with two unknowns which are solved using a Runge-Kutta solution technique (Healy, 1997). An important feature of RIVJAM is that it considers seepage flow of water through the voids of an ice jam (Beltaos, 1993). This is an important consideration near the toe of an ice jam where the thickness and water surface slope are high (Beltaos, 1999).

River2D is a two-dimensional, depth-averaged hydrodynamic model developed at the University of Alberta (Steffler and Blackburn, 2002). It is available in the public domain and is under continual development. River2D is a finite element model and uses the conservative Petrov-Galerkin upwinding formulation to compute river hydraulics. The model is able to simulate flow under a floating ice cover with a user-defined thickness and

roughness. The current version of the model does not simulate ice dynamics or the generation of ice via thermal processes, however these processes are to be incorporated in River2D in the future (Wojtowicz, 2010).

MIKE-ICE was developed as an add-in module to the MIKE11 model of the Danish Hydraulic Institute to study the effects of a hydroelectric project by Hydro-Quebec (Theriault et al., 2010). The MIKE11 model applies the six point Abbott-Ionescu implicit finite difference scheme to solve the one-dimensional St. Venant equations. The MIKE-ICE model enables unsteady one-dimensional simulations that incorporate several thermal and dynamic processes related to river ice. These include: supercooling of water and frazil generation, formation of surface ice pans, progression of the leading edge via surface juxtaposition, transport and deposition of ice under a stable ice cover, border ice growth, static ice cover growth and melt, and the development of open water areas in the presence of warm (greater than 0°C) water and/or during warm weather events (Theriault et al., 2010). An ice jam module can also be activated, which calculates the ice thickness profile through a force balance method, following the theory of Pariset et al. (1966) and Uzuner and Kennedy (1976) (Carson et al., 2011).

The DynaRICE model is a two-dimensional numerical model focussed on simulating the dynamic transport and jamming of surface ice. It is unique in that it employs two reference frames for treating hydrodynamics and ice dynamics. Hydrodynamics are modeled using the depth-averaged Navier Stokes equations solved using the streamline upwind Petrov-Galerkin approach on an Eulerian finite element mesh. Ice dynamics are calculated using the Discrete Parcel Method (DPM) and Smoothed Particle Hydrodynamics (SPH) on a Lagrangian reference frame. The major advantage of this

method is that it avoids the numerical diffusion associated with an Eulerian finite difference method (Shen et al., 2000b). The hydrodynamic and ice dynamic components are loosely coupled; ice properties such as thickness and velocity are transferred to the finite element nodes at each coupling interval. DynaRICE includes seepage discharge through the porous ice layer in the hydrodynamic equations allowing for more accurate simulation of grounded ice jams. The DynaRICE model does not account for any thermal processes. The two-dimensional river ice model, CRISSP2D, was built upon DynaRICE to incorporate thermal processes. CRISSP2D is one of the most comprehensive river ice numerical models in the world, and is able to simulate most processes related to river hydrodynamics, ice dynamics, and breakup. Its capabilities include: (i) simulation of unsteady flow conditions including supercritical and subcritical flows, and dry-wet bed conditions, (ii) calculation of heat transfer using linearized equations or a complete energy budget based on weather data, (iii) water temperature calculations and supercooling events, (iv) simulation of freeze-up processes including generation of frazil ice, anchor ice, border ice, and occurrence of surface ice runs, (v) simulation of undercover transport and deposition of ice, (vi) simulation of dynamic transport of surface ice and ice jam evolution, (vii) simulation of thermal growth and decay of ice covers, as well as mechanical breakup conditions (Liu et al., 2006). The CRISSP2D model is employed and developed in this research, with the specific goal of simulating the processes of ice cover consolidation and thermal strengthening during freeze-up. A more detailed description of the CRISSP2D model is provided in Chapter 4.

Although several numerical river ice models exist, the majority of them are based on classical theory of rubble ice cover strength by Pariset et al. (1966) and Uzuner and

Kennedy (1976). Very few distinguish between breakup and freeze-up scenarios, and those that do typically simply apply a cohesion value to the rubble strength to account for thermal processes that provide additional non-frictional resistance. The growth of a surface crust on an ice cover, which depends on a variety of factors including ice cover porosity, motion and deformation of the ice cover, and surface heat transfer rate, is an important process that affects the formation of a river ice cover at freeze-up. Incorporating this process into a numerical model would increase model robustness, and enhance its ability to predict the rate of ice cover progression, likelihood of consolidation events, final ice cover thickness, ice-affected water levels, and potential toe locations of freeze-up ice jams.

1.3 Research Objectives

As evidenced in the previous sections, river ice has several implications on humanity. Ice-affected staging can lead to flooding and result in damage to infrastructure, emergency evacuation of communities, and even threat to human safety. Ice conditions on fast flowing rivers during freeze-up can be highly dynamic due to the shoving and mechanical thickening of a forming ice cover. Consolidation events cause rapid changes in water level that can cause flash flooding without warning. Factors influencing the timing and severity of consolidation events are still not completely understood, partially due to the lack of detailed field data and observations of the consolidation process. Numerical modelling capabilities have advanced over the last several decades, however the link between thermal strengthening of an ice cover and the consolidation process is an area that requires further study and development. The objectives of this research are as follows:

1. Obtain a better understanding of the processes associated with river ice cover formation on fast-moving rivers through field research. A multi-year field program will be conducted to monitor freeze-up river ice processes on a reach prone to consolidation events. Detailed quantitative datasets including water levels, top-of-ice elevations, and ice cover thickness will be collected, in addition to meteorological data. Qualitative observations of the shoving process will also be documented. These data will be useful in advancing the understanding of the interdependencies of thermal and ice dynamic processes, and the associated impact on river hydraulics during the freeze-up period. Additionally, collected datasets will be useful in assessing the ability of state-of-the-art numerical models in simulating the freeze-up regime of the selected river.
2. Investigate the effect of surface freezing of interstitial water on the consolidation resistance of a floating rubble ice cover through a series of controlled laboratory experiments. The test series will assess the effect of freezing index (degree-minutes of freezing) on the ability of an ice cover to resist mechanical thickening. Results will be used in conjunction with field observations to inform numerical model development.
3. Advance the capabilities of a numerical river ice model by incorporating the process of ice cover strengthening due to thermal processes. Subroutines will be added to the CRISSP2D model to simulate the link between thermal and dynamic processes as they relate to mechanical thickening of ice covers during the freeze-up period. The new CRISSP2D formulations will be tested on an idealized channel to assess their functionality at a conceptual level.

4. Evaluate the applicability of the revised numerical model to the river selected for the field monitoring program. Simulations will be run for multiple freeze-up seasons, and the model will be evaluated on its ability to capture processes related to the progression, collapse, and thickening of the ice cover as observed in the field. Model results will be compared to field measurements of water levels, ice elevations, and ice thicknesses.

1.4 Organization of Thesis and Contributions of the Author

Chapter 2 of this thesis presents work related to the field monitoring program that was undertaken to further the understanding of dynamic ice processes at freeze-up. Two papers are included in this chapter. The first, titled “Field monitoring of secondary consolidation events and ice cover progression during freeze-up on the Lower Dauphin River, Manitoba”, is an extended version of a paper published in the journal *Cold Regions Science and Technology* in 2018. The paper is supplemented with data from the 2017-2018 monitoring season, which occurred after the original publication. Additional historical data is also included to provide context. The author was part of the group of three, including Dr. Shawn Clark (program co-advisor) and Alexander Wall (lab technician), that deployed the instrumentation and visited the study site on several occasions over the three monitoring seasons. The author contributed to the installation of equipment and selection of monitoring techniques and sites. The author processed and analyzed the collected data independently, with guidance and suggestions from Dr. Clark. The paper was written solely by the author of this thesis, again with comments and revisions suggested by Dr. Clark. The paper was accepted for publication in January 2018.

The second paper of Chapter 2, titled “Effects of freeze-up consolidation event surges on river hydraulics and ice dynamics on the Lower Dauphin River” was also written solely by the author of this thesis, with reviews from Dr. Clark and Dr. Jarrod Malenchak (program co-advisor). The author independently analyzed water level data collected during the 2016-2017 field monitoring season, which made up the primary content of the paper. The paper was accepted for publication in September 2018.

Chapter 3 presents laboratory experiments that were performed to understand the impacts of thermal processes on the ability of an ice cover to resist mechanical thickening. The apparatus and experimental methods were designed primarily by the author, with guidance from the program co-advisors. The author independently conducted the experiments and analyzed the results. The paper, titled “Laboratory investigation of the consolidation resistance of a rubble river ice cover with a thermally grown solid crust”, was written solely by the author, with revisions and suggestions provided by the program co-advisors. The paper was accepted for publication in October 2018.

Chapter 4 presents work on the incorporation of the processes observed in the preceding chapters into a two-dimensional numerical river ice model, CRISSP2D. An overview of the capabilities of CRISSP2D is provided, along with the current model formulations for ice dynamics. A revised formulation is proposed, which enables the simulation of thermal strengthening of a forming ice cover at freeze-up. The functionality of the revised formulations is demonstrated through idealized simulations on a rectangular, straight channel. The base and revised models are then applied to a case study of the Lower Dauphin River and results are compared with field measurements.

Chapter 5 summarizes the major findings and contributions of this research. Suggested topics for future work related to field monitoring, laboratory experiments, and numerical model development are also presented.

Chapter 2 Field Monitoring of River Ice Cover Formation at Freeze-up

2.1 Introduction

A primary objective of this research was to design and execute a detailed field monitoring program that centred around data collection related to the dynamic formation of an ice cover at freeze-up and the occurrence of consolidation events. This chapter includes two papers related to the field program.

The first paper, presented in Section 2.2, outlines the methodology for the field program, including the selected study site of the Lower Dauphin River and instrumentation and monitoring techniques employed. Data from two monitoring seasons (2015-2016 and 2016-2017) were included in the original publication. Data from the 2017-2018 monitoring season was added in Section 2.2 to provide an additional field dataset for comparison purposes. The paper is focussed on the effects of dynamic consolidation events on water level records and ice cover characteristics through the freeze-up season. Detailed first-hand observations of the progression and collapse of the ice cover were documented. The spatial and temporal resolution of the data and observations constitute one of the most detailed and complete studies of freeze-up processes in the literature to date. The collected datasets advance the state of knowledge on consolidation events and their impact on the final configuration of a river ice cover. Furthermore, the detail of the monitoring program

provides an excellent test case for evaluation of state-of-the-art numerical models, which could highlight deficiencies in these models and areas for future development. The paper was accepted in January 2018 and published in the peer-reviewed journal Cold Regions Science and Technology (doi:10.1016/j.coldregions.2018.01.014).

The second paper, presented in Section 2.3, examines the effect of ice consolidation events on river hydraulics. Specifically, the paper uses an analysis method previously published in the literature to determine the hydraulic properties at various monitoring sites based on the measured waveform of surges originating from the collapse of a forming ice cover. This work builds off the success of the field monitoring program, and is a novel study in estimating the unsteady hydraulic properties of the flow in a reach prone to consolidation events. The work highlights the importance of dynamic ice events on the final ice configuration and peak water levels, and suggests that using the steady state winter discharge and a simplified calculation of the equilibrium jam thickness may not always provide accurate predictions. The paper was accepted to the journal Cold Regions Science and Technology in September 2018 (doi:10.1016/j.coldregions.2018.09.003).

2.2 Paper 1: Field monitoring of secondary consolidation events and ice cover progression during freeze-up on the Lower Dauphin River, Manitoba

2.2.1 Abstract

Dynamic freeze-up processes on fast-moving rivers determine the ultimate thickness of the ice cover and the associated ice-affected water levels. Secondary

consolidation events, the shoving and mechanical thickening of a progressing ice cover at freeze-up, can cause rapid water level rises that can flood land unexpectedly. Additionally, they can lead to over-thickening of an ice cover beyond that calculated with a steady discharge. The lack of field data and observations of secondary consolidations in the literature results in a poor understanding of the conditions that affect the timing and severity of these events. In this study, data pertinent to further the understanding of secondary consolidation events were collected over three freeze-up monitoring seasons on the Lower Dauphin River. The high spatial resolution of pressure transducers and trail cameras, along with first-hand observations of consolidation events, provide a dataset that was used to analyze the extent of several secondary consolidation events and their effect on water levels throughout the 12 km reach. The most significant event caused a water level increase of 2.5 m over a span of 8 minutes when approximately 7.5 km of ice cover consolidated to a length of 4.7 km. An unmanned aerial vehicle was used to obtain aerial imagery of the ice front, which advanced through a series of localized shoves as incoming ice packed against the stationary cover. Other data collected includes top-of-ice elevations, ice thickness estimations, and meteorological data. Collectively, the dataset provides a relatively complete account of three freeze-up seasons on the Lower Dauphin River, including the processes of ice cover advancement and consolidation that play an important role in the peak ice-affected water levels and flooding potential. The dataset can be used by ice researchers to assess the ability of current state-of-the-art numerical models in simulating dynamic freeze-up processes, and highlight areas for future research and model development.

2.2.2 Introduction

Freeze-up ice jams in fast-moving rivers can cause water levels to rise far beyond open water elevations, which can lead to flooding and socioeconomic concerns. The thickness of a rubble ice jam has historically been modeled using theory derived from soil mechanics; the ice is thought to behave as a granular material with strength resulting largely from frictional interlocking of the ice blocks. The ability of an ice cover to resist mechanical thickening under external forces of gravity and water shear is proportional to the confining pressure due to buoyancy (Beltaos, 1995a). Although they are often modeled the same way, freeze-up ice jams differ from breakup jams in that the air temperature is below 0°C, causing continual heat loss from the river (Michel, 1991), which adds strength as interstitial water freezes near the surface. As an ice cover progresses upstream during freeze-up, a portion of it may unexpectedly shove and consolidate, creating a thicker accumulation with a greater backwater effect. Hereafter, the term “shove” will refer to the mobilization of a stationary ice cover that is in the process of collapsing, and “consolidation” will refer to the mechanical thickening of the collapsing cover. These two processes often occur simultaneously. Factors influencing the magnitude, frequency, and extent of consolidation events are poorly understood.

Several field studies have been conducted to investigate breakup ice processes and dynamics (e.g. Beltaos et al., 1996; Beltaos and Carter, 2009; Beltaos et al., 2011; Beltaos et al., 2012). However, the number of field studies during freeze-up is more limited (e.g. Michel, 1984; Andres, 1999; Robichaud and Hicks, 2001; Andres et al., 2003; Sui et al., 2005). It has been observed in the field that ambient air temperatures during freeze-up are linked to ice dynamics, and can control whether an ice cover is more prone to frontal

progression or consolidation events (Andres, 1999; Michel, 1984). Michel (1991) suggested that shoving is much less likely during freeze-up due to freezing effects, unless the stationary front progresses very quickly. Andres et al. (2003) observed ice consolidation events at freeze-up on the Peace River. They used the term “primary consolidation” to refer to the small, frequent shoves that occur very early in the formation period, before the freezing of interstitial water. They used the term “secondary consolidation” to refer to the collapse and mechanical thickening of an existing consolidated or juxtaposed ice cover. Secondary consolidations were found to produce the highest freeze-up levels due to the effects of the surge sent downstream, resulting in over-thickening of the ice cover. The conditions that are thought to trigger secondary consolidation events are (i) rapid advancement of the stationary front, (ii) sudden increases in discharge, and (iii) dramatic changes in atmospheric conditions (e.g. air temperature and solar radiation) that can deteriorate or hinder the growth of the solid crust at the surface of the ice cover (Andres et al., 2003).

The Dauphin River in central Manitoba has been observed to undergo secondary consolidation events during freeze-up in the past that have caused ice-related staging of up to 4-5 m or more (Clark and Wall, 2016). Lindenschmidt et al. (2012) applied the numerical model RIVICE to simulate ice thicknesses and ice-affected water levels for the 2010-2011 freeze-up season on the Dauphin River, but the quantity and completeness of field data with which to compare numerical simulation results was limited. Further, only final ice elevations at the end of freeze-up were used to calibrate model parameters and the ability of the model to simulate time-dependent variations in water levels that are greatly influenced by secondary consolidation events was not shown. More comprehensive field

datasets on the timing and magnitude of secondary consolidation events are required to advance the current understanding of freeze-up ice dynamics and the factors that trigger these events.

The objective of this paper is to present data collected during three freeze-up monitoring seasons on the Dauphin River in central Manitoba, with a specific focus on the processes of ice cover advancement and secondary consolidation events in the steeper, lower reach. Novel to this study is the completeness, quantity, and variety of collected data; the spatial and temporal resolution of measurements captured dynamic conditions of ice cover progression and consolidation events to an unprecedented degree. This dataset sheds light on processes that lead to very thick, consolidated ice jams in fast-moving rivers, and implores further study of how atmospheric conditions affect timing and severity of consolidation events at freeze-up. Visual observations of ice conditions before, during, and after secondary consolidation events, paired with quantitative data collected from deployed instrumentation (including water levels and meteorological variables), provide a unique dataset that advances the current state of published freeze-up field data. Collectively, the data also provides other ice researchers with a well-documented case study with which to evaluate the ability of state-of-the-art numerical models in simulating secondary consolidation events and their effects on channel hydraulics.

2.2.3 Methodology

2.2.3.1 Study Site and Historical Data

The Dauphin River is located in central Manitoba, approximately 250 km north of the city of Winnipeg, and is the final river in the water network connecting Lake Manitoba

to Lake Winnipeg (see Figure 2.1). The outflow of Lake Manitoba travels along the Fairford River to Lake Pineimuta, then to Lake St. Martin. The natural outlet to Lake St. Martin is the Dauphin River, which is approximately 52 km long and drains to Lake Winnipeg. Water levels on Lake Manitoba are regulated by the Fairford River Water Control Structure (FRWCS) which has been in operation since 1961. The stoplog control structure is currently operated to maintain the water level of Lake Manitoba between 810.5-812.5 ft (240.0-247.7 m), and to avoid sudden changes in water levels on Lake Pineimuta and Lake St. Martin. The range was lowered to 810.0-812.0 ft (246.9-247.5 m) for a period of five years beginning in 2013 in response to a recommendation in the 2013 report of the Lake Manitoba/Lake St. Martin Regulation Review Committee (Manitoba Infrastructure, 2015).

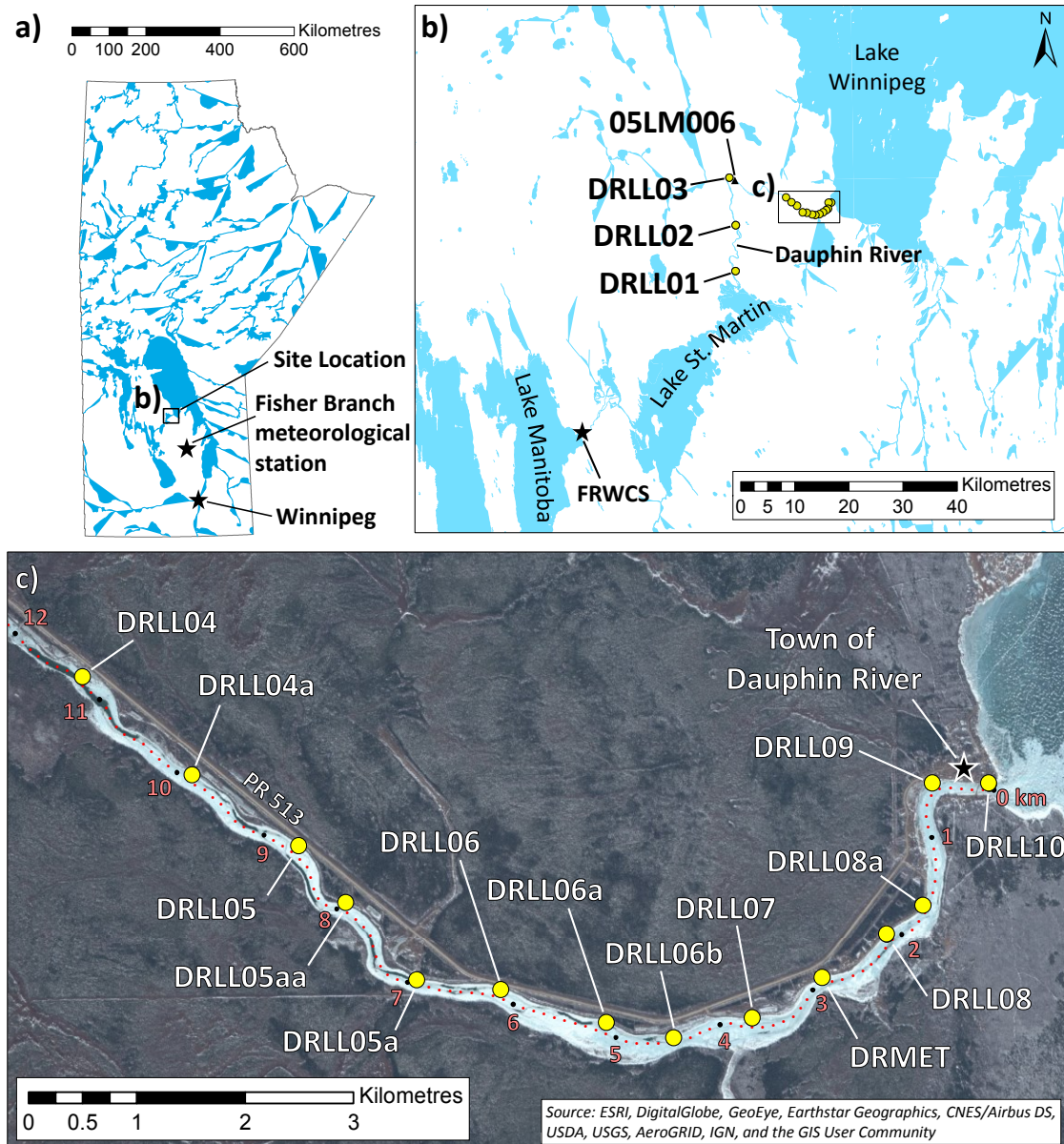


Figure 2.1. Map of Dauphin River study area. Monitoring locations are shown with shaded circles. The Fisher Branch meteorological station, Environment Canada gauge 05LM006, and the Fairford River Water Control Structure (FRWCS) are shown for reference.

An interesting property of the Dauphin River is the abrupt channel slope change that occurs approximately 11.2 km upstream of the outlet to Lake Winnipeg. The upper reach has a slope of approximately 0.029% over its 40 km length, while the lower reach has a slope of 0.16%; roughly 5.5 times steeper than the upper reach (Clark and Wall,

2016). The upper reach is characterized by numerous channel meanders and a wider top width with shallow banks; conversely, the lower reach is much straighter and has steeper banks. The width of the lower reach typically varies from about 110-160 m. The steeper slope of the lower reach causes the open water velocity in certain areas to exceed 1.5 m/s. The turbulent, fast-moving water can generate significant volumes of frazil ice during freeze-up. Historically, this has led to the formation of a hanging dam at the outlet to Lake Winnipeg as the entrained frazil ice is deposited under the thermal lake ice cover (KGS Group and North/South Consultants, 2014). Thick ice jams composed of frazil ice pans form in the steeper reach as the cover advances upstream from Lake Winnipeg; ice-affected water levels in this reach can rise 4-5 m or more above open water conditions (Clark and Wall, 2016).

Environment Canada operates a discharge and water level gauge on the Dauphin River (gauge 05LM006) that is located about 25 km upstream of the outlet (see Figure 2.1). A percentile analysis of the recorded flows during the ice-affected season from 1977-2017 is presented in Figure 2.2, showing that the flows from 2015 to 2018 were generally higher than average. The average discharges in 2015-2016, 2016-2017, and 2017-2018 during the period of ice cover advancement up the lower reach in each freeze-up season were approximately 130 m³/s, 210 m³/s, and 220 m³/s, respectively.

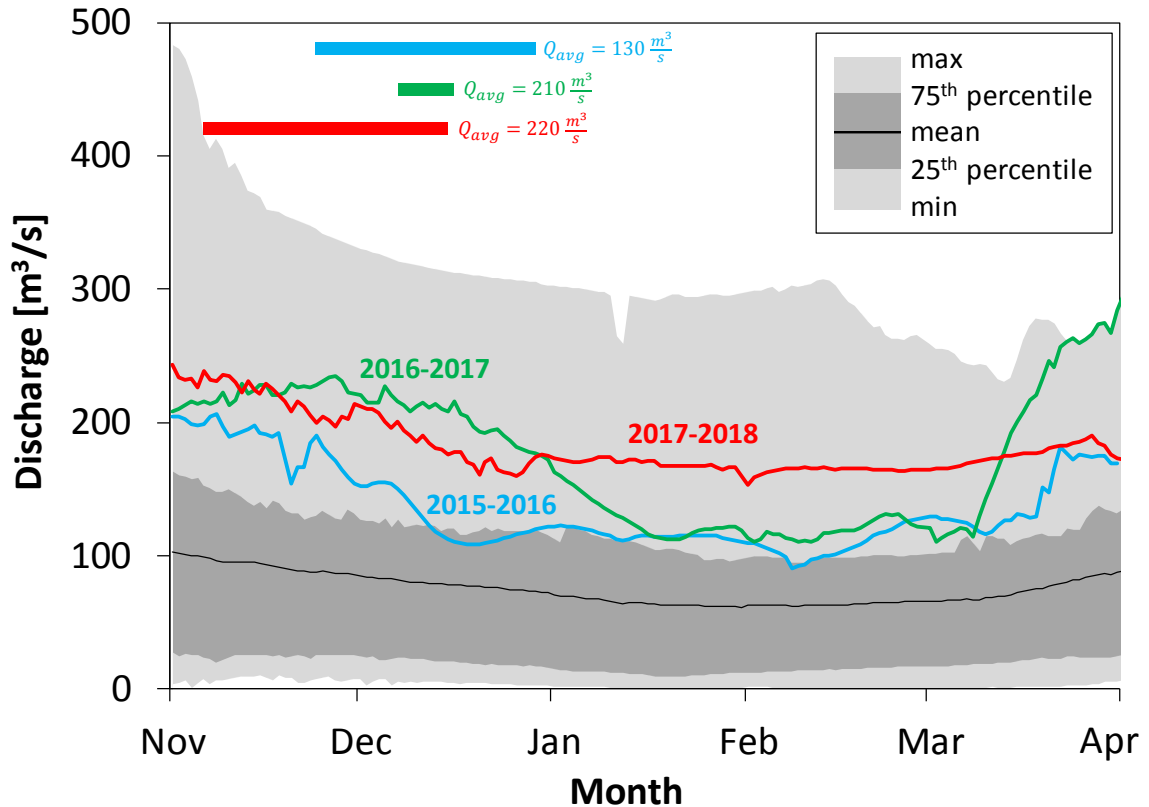


Figure 2.2. Historical flows during ice-affected period on the Dauphin River from 1977-2018. Data from Environment Canada gauge 05LM006. Bars indicate period of ice cover progression up the Lower Dauphin River in each monitoring season.

Figure 2.3 shows the historical variation in average Dauphin River flow over the months of November and December, and the water level on Lake Manitoba. As shown, levels on Lake Manitoba have been higher than average in the past decade, and the average flow on the Dauphin River during the time of ice cover formation has increased from approximately $43 \text{ m}^3/\text{s}$ prior to 2005, to $186 \text{ m}^3/\text{s}$ after 2005.

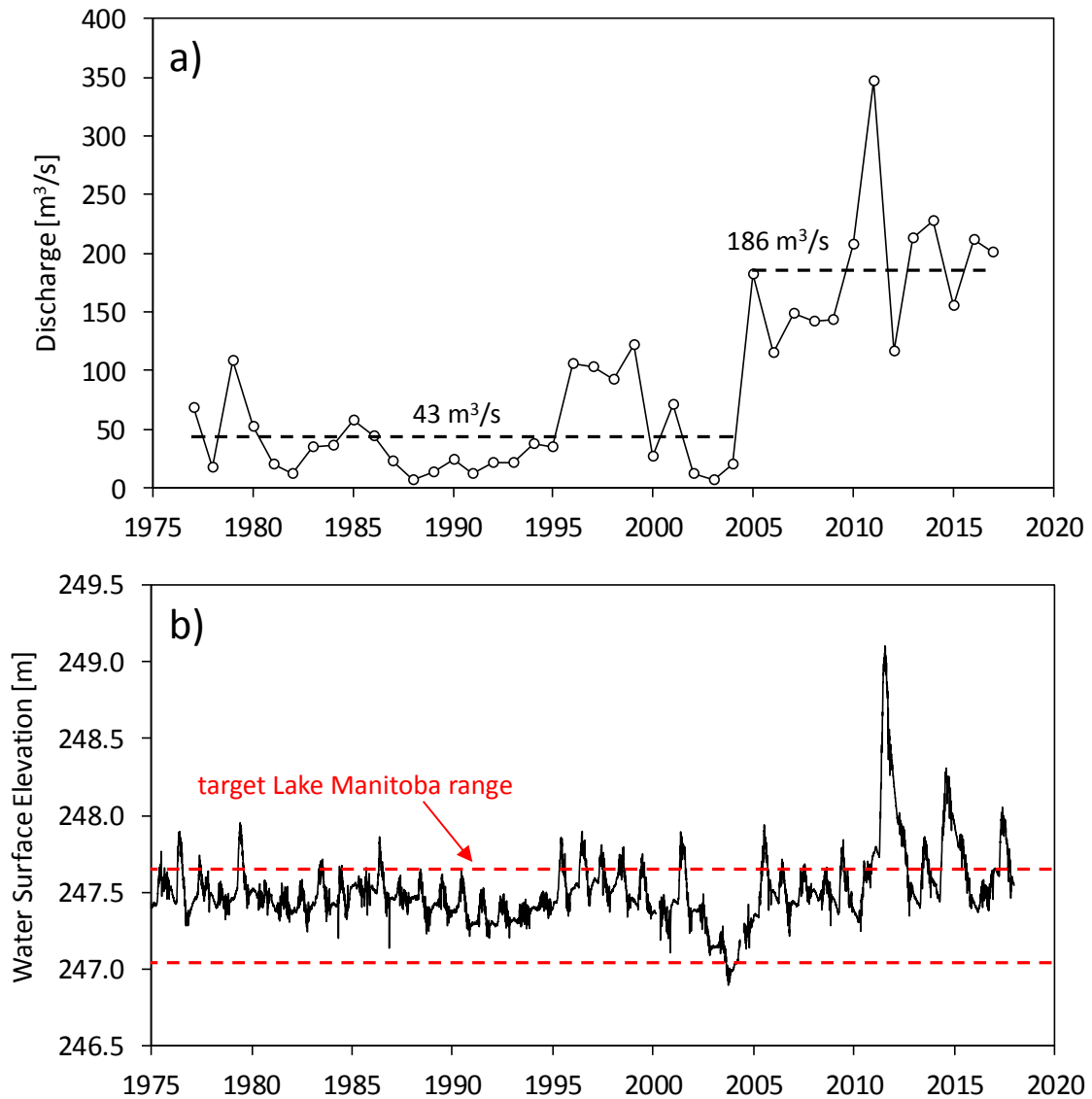


Figure 2.3. Historical variation of a) average Dauphin River discharge during months of November and December, and b) Lake Manitoba water levels.

Water levels at gauge 05LM006 have been recorded since 2002. Figure 2.4 shows the average air temperature during the freeze-up period and the peak water level at 05LM006 each year back to 2002. The “freeze-up period” was assumed to span the time over which the degree-days of freezing (DDOF) increased from 50 to 400. As shown, the peak water levels at gauge 05LM006 increased in 2005, corresponding to the time at which

the average flow on the Dauphin River increased. However, a direct relationship between peak water level and average flow does not exist. For example, the flow was highest in 2011, but the peak water level at 05LM006 was not the highest on record. In the years 2016 and 2017, the average discharge was similar, but the peak stage at 05LM006 in 2017 was the highest on record, and over one metre higher than in 2016. The variability is caused by the effects of freeze-up ice accumulations on flow conveyance, which differ in severity from year to year. The average air temperature from the Fisher Branch meteorological station is also shown in Figure 2.4; the average temperatures during the period of ice cover formation from 2002-2017 varied from -18°C to -7°C . The severity of ice accumulations at a specific location along the river likely depend on complex relationships between flow and meteorological conditions at the time of cover formation. Thus, prediction of peak staging requires sophisticated numerical models rather than a simple relationship between flow and stage. These models rely on detailed field observations for development and calibration.

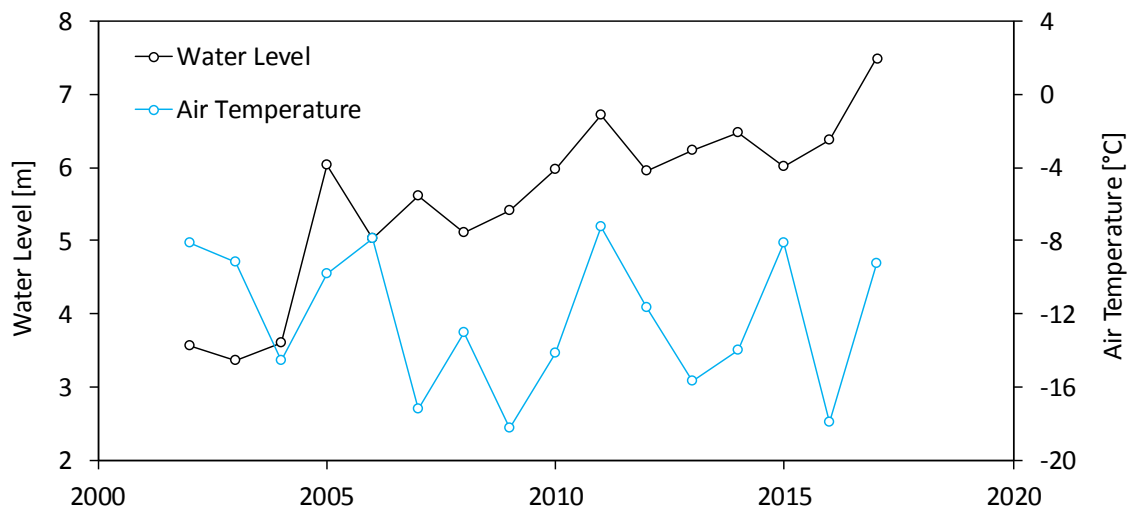


Figure 2.4. Peak stage at gauge 05LM006 and average air temperature during freeze-up periods from 2002-2017.

2.2.3.2 Overview of Monitoring Efforts

A summary of instrument types and deployment locations for the 2015-2016, 2016-2017, and 2017-2018 seasons is shown in Table 2.1. The number of monitoring sites was increased in the second monitoring season due to the apparent success of the 2015-2016 season and need for higher resolution data. Monitoring sites were originally named sequentially as DRLL01 (where “DRLL” indicates “Dauphin River Level Logger”), DRLL02, etc. from Lake St. Martin to Lake Winnipeg (see Figure 2.1). Sites added in subsequent years were appended with a letter to denote their position between original sites.

Table 2.1. Instrumentation deployed and retrieved in the 2015-2016, 2016-2017, and 2017-2018 monitoring seasons.

	DRLL01	DRLL02	DRLL03	DRLL04	DRLL04a	DRLL05	DRLL05aa	DRLL05a	DRLL06	DRLL06a	DRLL06b	DRLL07	DRMET	DRLL08	DRLL08a	DRLL09	DRLL10
2015-2016																	
Water Level	○	●	●	●		●			○			●		●		●	●
Trail Camera	●	●		●		●			●			●		●			●
Barometric Pressure									●				●				
Meteorological Data													●				
2016-2017																	
Water Level		●	●	●	●	●		●	●	●	●	●	●	●	●	●	●
Trail Camera		●	●	○		●		○	●			●	●	○	●		●
Barometric Pressure									●				●				
Meteorological Data													●				
2017-2018																	
Water Level		●	●	●	●	●	●	○	●	●	●	○	●	●	●	●	●
Trail Camera			●		○	●	○	●	●	●	●	●	●	●	●		○
Barometric Pressure									●				●				
Meteorological Data													●				
Note: ● retrieved ○ lost/malfunctioned																	

Two site visits were conducted on December 9, 2016 and December 12-14, 2016 during the 2016-2017 season to coincide with ice cover progression up the Lower Dauphin River. Similar site visits were conducted from November 20-23, 2017 and from December 13-14, 2017 for the 2017-2018 monitoring season. Visual observations of shoving events during these visits aided in comprehending the data collected from the loggers and trail cameras. Surveys of the near-shore top-of-ice profiles were conducted on March 23-24, 2016, February 21-22, 2017, and February 21-23, 2018 for each respective monitoring season.

For the 2016-2017 season, near-shore transect surveys of ice elevations and ground elevations were conducted during site visits on March 15-16, 2017 and May 15-16, 2017 to quantify the thickness of the ice grounded along the channel bank. Similar transect surveys were performed on October 24-26, 2017 (ground) and February 21-23, 2018 (ice) for the 2017-2018 season. A schematic illustration of the evolution of the ice cover in the spring is shown in Figure 2.5. Warm air temperatures and increased flows caused the ice cover to decay in the middle of the channel. The water level dropped with the increased conveyance and caused the ice cover to rest along the bank. One set of transect surveys were conducted during this time to obtain near-shore top-of-ice elevations. The second set of transect surveys were conducted when the ice cover had fully cleared to obtain ground elevations at the same locations. Shear cracks were often observed near the bank during the ice transect survey, but they typically did not extend to the ground and therefore did not provide a good estimate of the ice thickness.

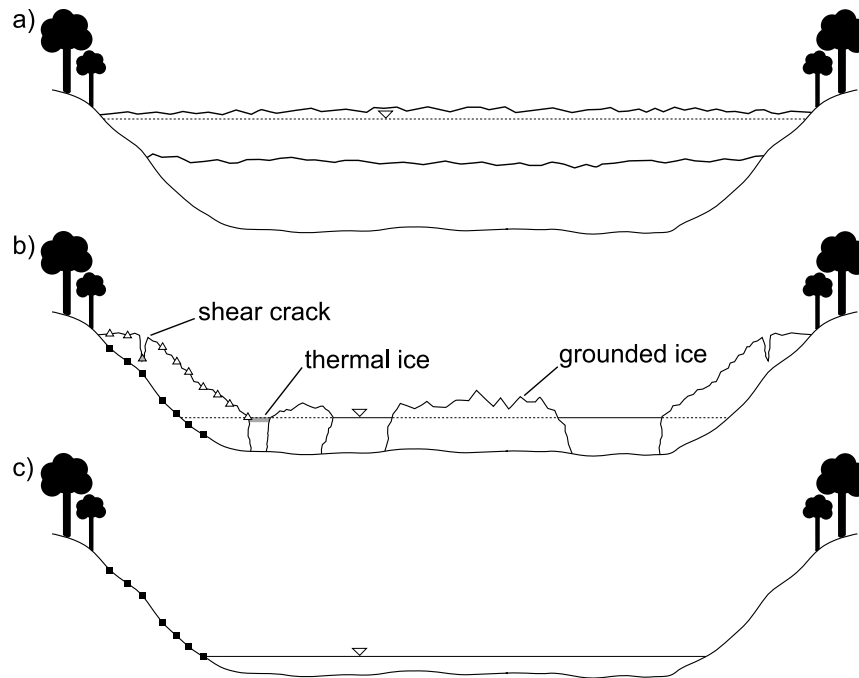


Figure 2.5. Schematic of the river transition at breakup based upon visual observations at DRLL05a; a) full ice covered condition, b) ice grounded at banks at onset of breakup, and c) open water condition. Surveys were timed to obtain top-of-ice elevations (hollow triangles) as the ice cover was resting on the channel bank and ground elevations (solid squares) when the ice had fully cleared.

2.2.3.3 Equipment Specifications and Deployment Methods

Water levels were recorded using pressure sensors (Levellogger Model 3001, Solinst, USA). These instruments are capable of measuring water level with an accuracy of ± 0.003 m. The loggers were fastened to $75 \times 75 \times 315$ mm iron angles and the assembly was secured to the bed of the river using rebar. Each logger had a cloth membrane wrapped around the pressure sensor for protection from fine sediments. At certain locations, the iron angles were attached to 1,000 kg tensile strength steel slings which were trenched and anchored to large trees or bolted to bedrock at the channel banks. One barometric pressure sensor (Barologger Model 3001, Solinst, USA) was installed on a tree at site DRLL06 to allow compensation of the water level loggers for atmospheric pressure fluctuations. A different pressure sensor was installed at DRMET for the 2016-2017 and 2017-2018

seasons (HOBO MX2001, Onset Computer Corps., USA), which had the added benefit of Bluetooth capabilities and its own barometric pressure logger for self-compensation. Typically the water level loggers were installed within 5 m of the channel bank. In 2017-2018, additional Solinst loggers were installed near sites DRLL06 and DRLL07. These additional loggers were programmed to sample at a higher frequency (sampling interval of 1 minute versus 8-10 minute sampling interval of typical installations) in an attempt to capture finer details of water level fluctuations.

Time-lapse photographs of ice conditions were taken hourly with Moultrie M-1100i trail cameras mounted to mature trees. During site visits, aerial imagery was taken using a UAV (unmanned aerial vehicle, Phantom 2 Vision+ and Phantom 3 Professional, DJI, China) to validate trail camera images and to provide scale and perspective of sites. All UAV flights were flown in accordance with the exemptions outlined in the Canadian Aviation Regulations (SOR/96-433) or under approved special flight operators certificates (SFOC) approved by Transport Canada to be compliant with updated Canadian Aviation Regulations.

The meteorological station at DRMET consisted of a 3 m high tripod (UT10, Campbell Scientific, Canada) mounted with an air temperature and relative humidity sensor (HC2-S3-L, Rotronic, Canada; temperature accuracy $\pm 0.1^{\circ}\text{C}$), wind monitor (05103-10, RM Young, USA), barometric pressure sensor (CS106, Vaisala, USA), net radiometer sensor (NRLITE2, Kipp & Zonen, USA), and data logger (CR1000-XT, Campbell Scientific, Canada). A second tripod was set up with a high resolution outdoor camera (CC5MPX, Campbell Scientific, Canada) and water temperature probe (RBRvirtuoso T, RBR, USA; accuracy $\pm 0.002^{\circ}\text{C}$) linked to a satellite modem (9552B

Iridium, Campbell Scientific, Canada) to relay daily pictures and water temperature to a website accessible from the office. The daily updates were used to time site visits during freeze-up.

Surveys were carried out using a complete L1 GPS post process kinematic (PPK) system (R3 GPS L1, Trimble, USA) in 2015-2016, and a GNSS real time kinematic (RTK) system (VIVA GS 14 / CS20, Leica, USA) in 2016-2017 and 2017-2018. Each unit is capable of achieving sub-centimetre elevation accuracy depending on satellite orientation throughout the day and proximity to obstructions. All surveyed elevations were recorded in metres above sea level using the Canadian Geodetic Vertical Datum of 1928 (CGVD1928).

2.2.4 Results

2.2.4.1 Meteorological Data

Meteorological data were collected from the meteorological station at DRMET beginning on December 8, 2015. Prior to this, meteorological data from the Environment Canada station at Fisher Branch were used, which is approximately 100 km from the Lower Dauphin River (see Figure 2.1). In general, the air temperature recorded at Fisher Branch matched well with the measurements at DRMET. Air temperatures during the 2015-2016, 2016-2017, and 2017-2018 freeze-up periods are shown in Figure 2.6. The average air temperatures during ice cover advancement up the Lower Dauphin River (i.e. from Lake Winnipeg to DRLL04) in 2015-2016, 2016-2017, and 2017-2018 were -9°C, -21°C, and -9°C, respectively. Average wind speeds recorded while the ice cover was progressing up the Lower Dauphin River for both the 2015-2016 and 2016-2017 seasons were

approximately 1.6 m/s, while for 2017-2018 the average wind speed was approximately 2.0 m/s. The average net radiation (including both longwave and shortwave) recorded while the cover was progressing up the Lower Dauphin River in 2015-2016, 2016-2017, and 2017-2018 was -21 W/m^2 , -45 W/m^2 , and -27 W/m^2 respectively.

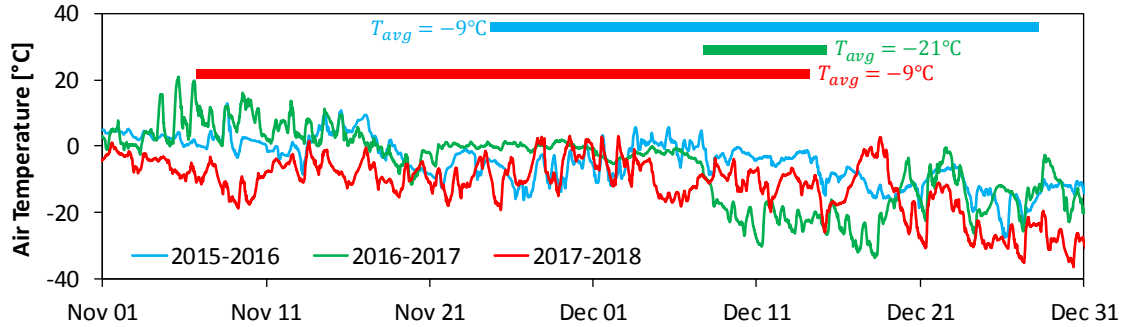


Figure 2.6. Air temperatures during freeze-up 2015-2016, 2016-2017, and 2017-2018. Bars indicate period of ice cover advancement up the Lower Dauphin River in each season.

Figure 2.7 shows the DDOF of the three monitoring seasons compared to historical data from the Fisher Branch weather station. For years back to 2002, the timing of the peak water level at Environment Canada gauge 05LM006 are also shown. On average, the peak water level at 05LM006 corresponds to 321 °C-day. In 2015-2016, 2016-2017, and 2017-2018, the peak at 05LM006 corresponded to 317 °C-day, 219 °C-day, and 358 °C-day, respectively.

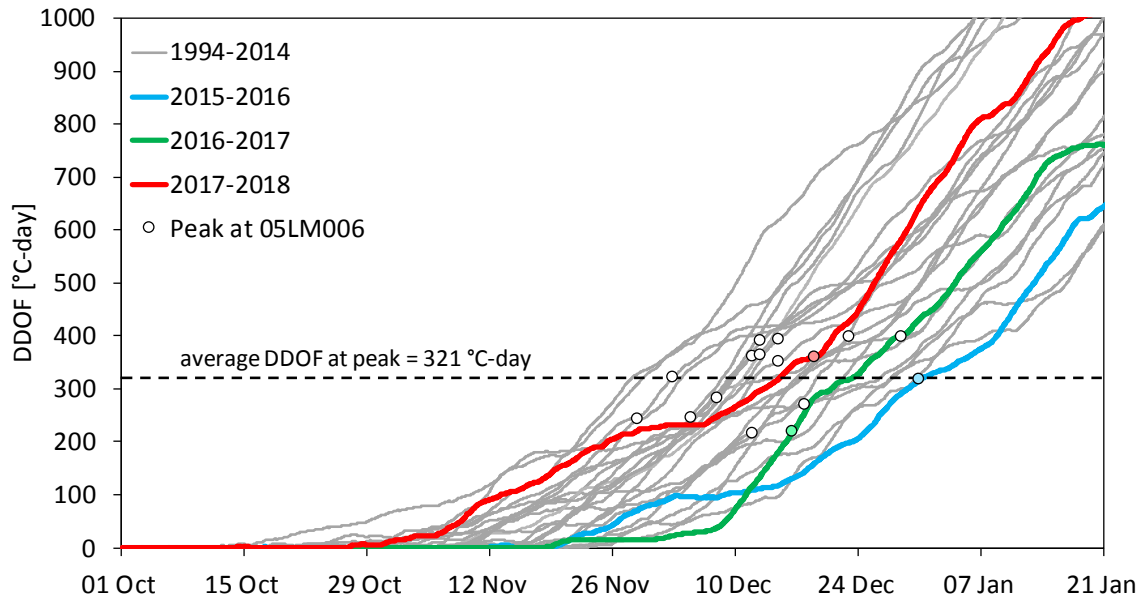


Figure 2.7. Comparison of DDOF of monitored seasons to historical data from Fisher Branch weather station.

2.2.4.2 Water Level Data

The water surface elevations recorded as the ice cover progressed up the Lower Dauphin River during the 2015-2016, 2016-2017, and 2017-2018 seasons are shown in Figure 2.8, Figure 2.9, and Figure 2.10, respectively. The corresponding DDOF for each season are also shown. Detailed plots of specific consolidation events are presented in Section 2.2.5.

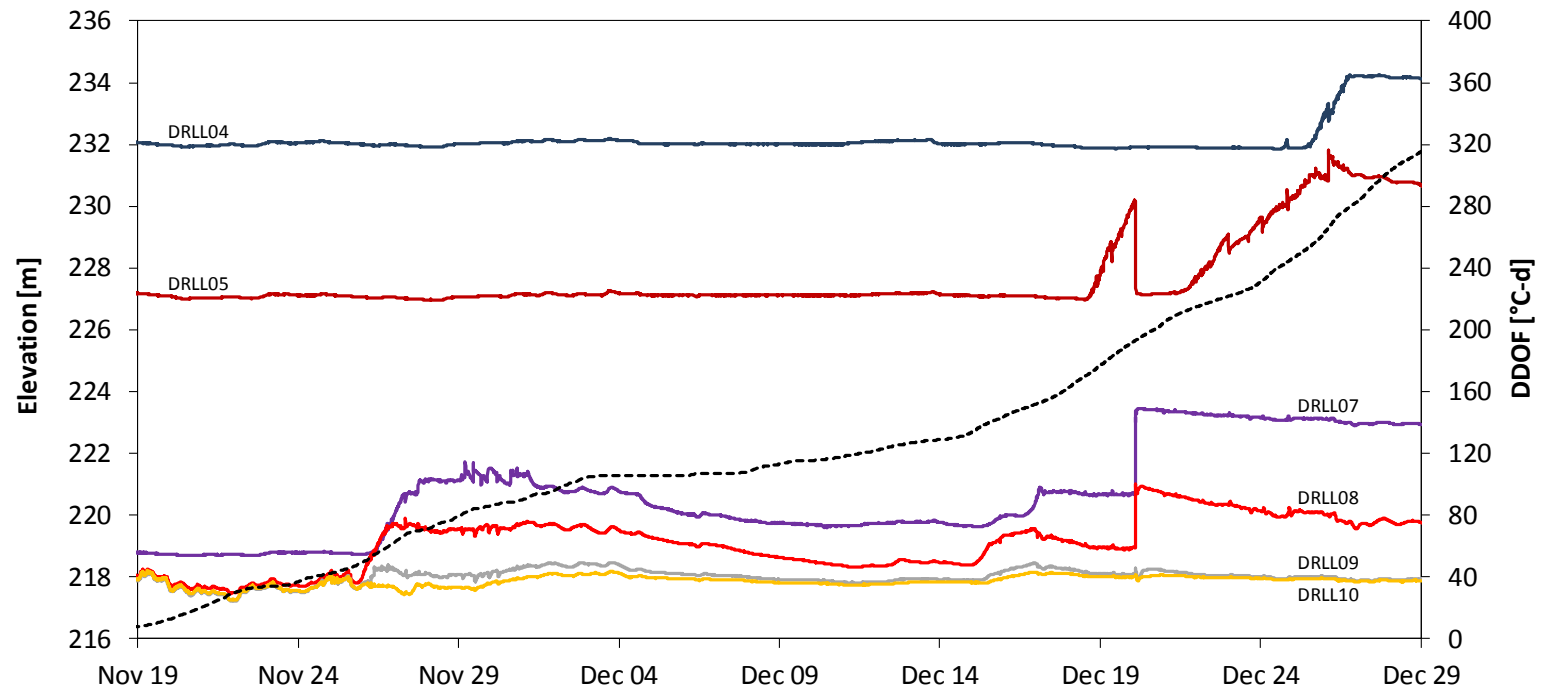


Figure 2.8. Water surface elevations during freeze-up 2015-2016.

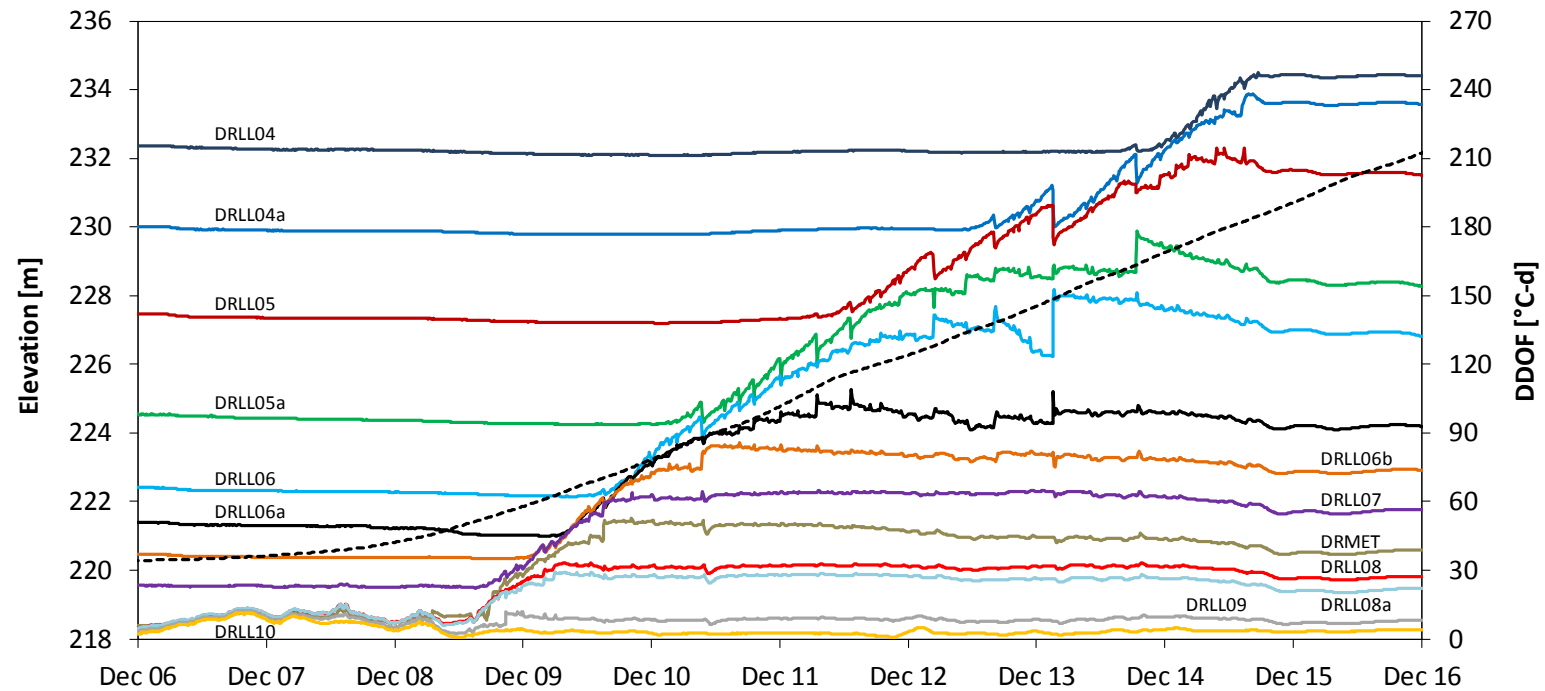


Figure 2.9. Water surface elevations during freeze-up 2016-2017.

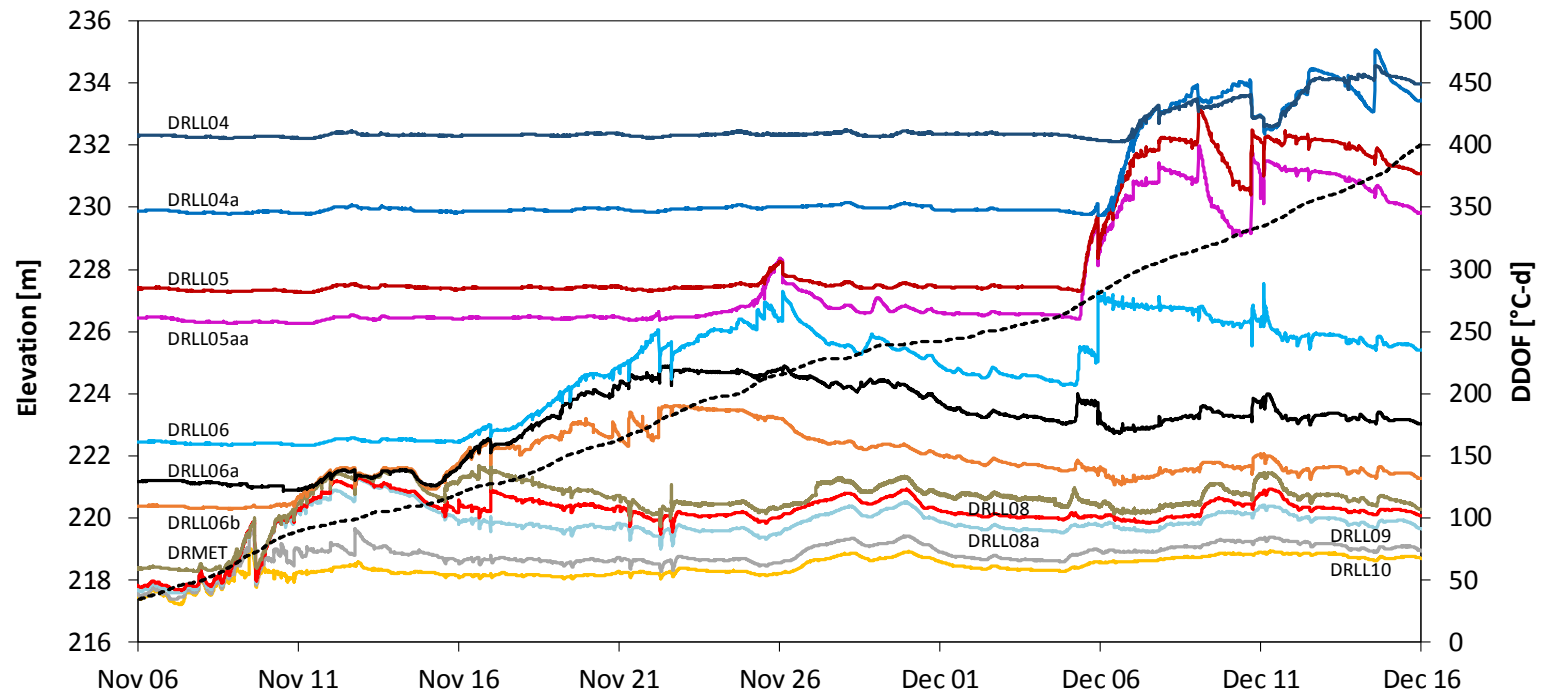


Figure 2.10. Water surface elevations during freeze-up 2017-2018.

2.2.4.3 Top-of-Ice Profiles

The surveyed top-of-ice elevations along the Lower Dauphin River for all monitoring seasons are shown in Figure 2.11. Peak water surface elevations recorded by deployed loggers are also shown. In most cases, the peak water levels tend to be located near the surveyed top-of-ice elevations. This suggests that the profiles surveyed in the early spring of each monitoring season provide a reasonable representation of the peak ice elevations experienced during the freeze-up period. The open water surface profile was calculated using a HEC-RAS model of the Lower Dauphin River with a Lake Winnipeg elevation of 218.5 m and a discharge of 200 m³/s.

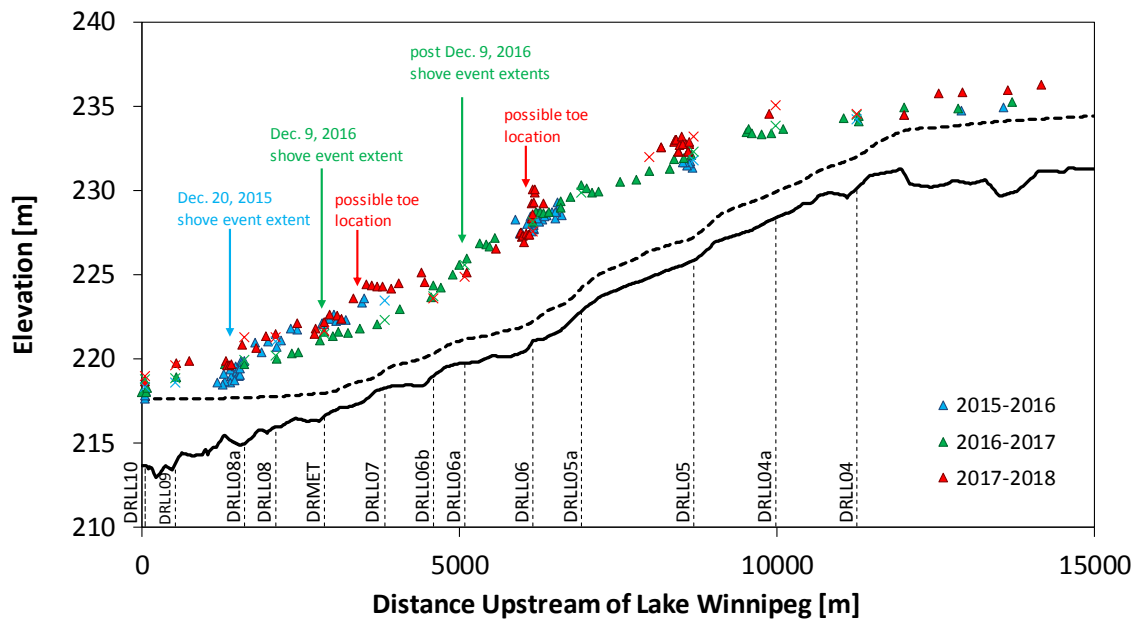


Figure 2.11. Surveyed top-of-ice profiles and peak water surface elevations recorded by loggers along the Lower Dauphin River in 2015-2016 (blue), 2016-2017 (green), and 2017-2018 (red). X markers indicate peak water surface elevations as recorded with the installed pressure sensors.

2.2.4.4 Trail Camera Imagery

Qualitative descriptions of ice conditions using hourly photographs from trail cameras at sites along the Lower Dauphin River in 2015-2016 and 2016-2017 are summarized in Figure 2.12. Trail cameras deployed in 2017-2018 were set to capture images at higher frequencies (e.g. one picture every five minutes) to allow for better observations of the highly dynamic processes during shove events. The cameras did not have the battery capabilities to last the entire season, so an evaluation of the ice conditions at each site over the entire season was not possible. Thus, this season is omitted from Figure 2.12. The ice conditions were broadly classified into categories based on visual interpretation of the images. The “open water” category refers to a condition where the majority of the channel was open and no frazil slush or pans were observed at the surface. The categories of various percentages of surface ice concentrations refer to cases where frazil slush and pans were present at the surface, and were moving freely downstream. Due to the oblique angle of the trail camera photographs, there is error associated with these estimations. The “actively consolidating cover” category refers to an ice cover that has 100% surface concentration (or nearly so), estimated to be not more than a few pans thick, that is going through frequent cycles of progression and small shoves similar to “primary consolidations” as described by Andres et al. (2003). In this category, the ice cover had a noticeably different appearance in consecutive hourly photographs. The “stationary consolidated cover” category refers to a consolidated ice cover that would move up and down with water level fluctuations, but had the same appearance between consecutive photographs (i.e. no change in orientation of ice pieces or movement of the cover). Another category was defined for a cover with significant open water leads, which were present

during the warm spell in 2015-2016. Finally, if an ice cover was observed to hold the same appearance for several consecutive hours (thus categorized as a stationary cover) then suddenly shove, a “secondary consolidation” event was marked. Ice conditions during the night were estimated based on conditions the previous evening and next morning. If it was suspected that a secondary consolidation event occurred overnight (by the changed appearance of a stationary cover), the water level records were used to mark the location of the event.

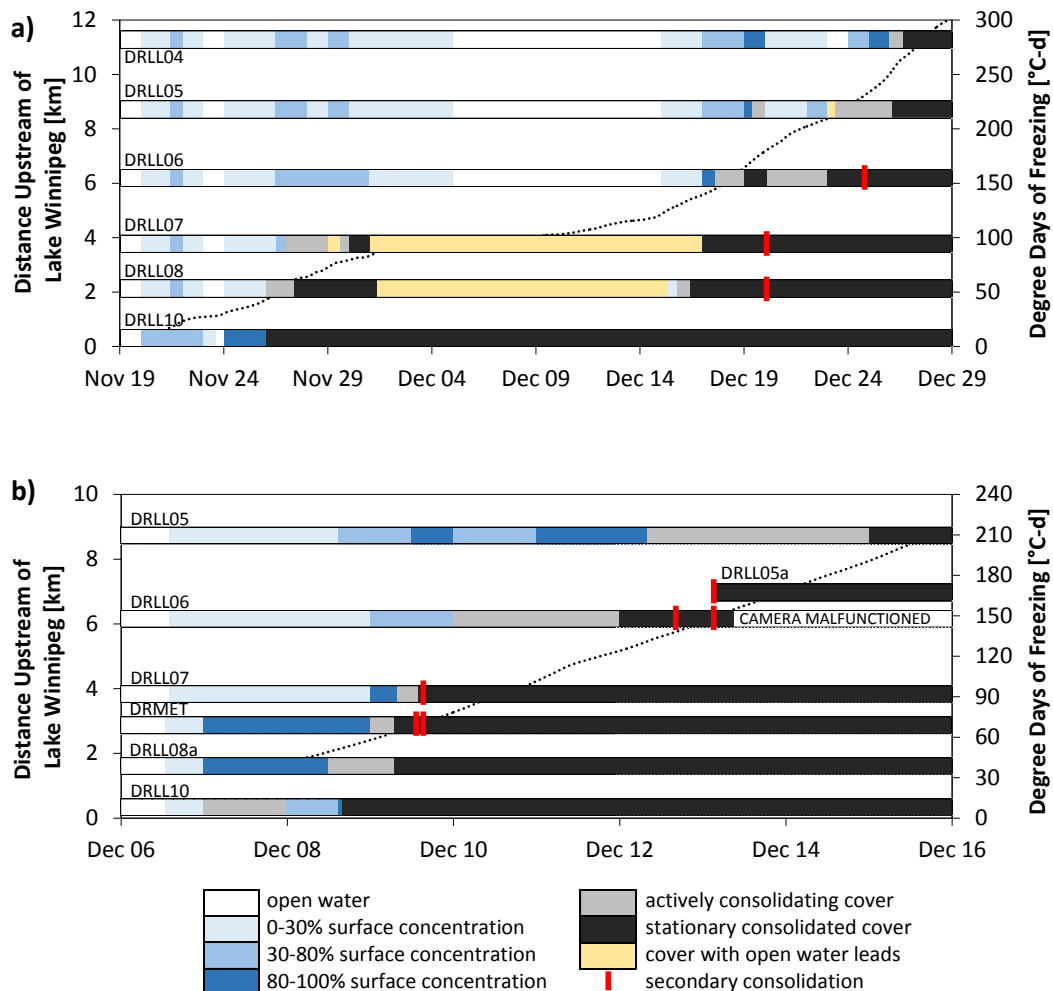


Figure 2.12. Qualitative ice conditions from trail camera photographs during freeze-up in a) 2015-2016 and b) 2016-2017. The degree-days of freezing (DDOF) for each freeze-up period are also shown.

2.2.4.5 Ice Thickness Estimations

The results of transect surveys in 2016-2017 and 2017-2018 are shown in Figure 2.13 and Figure 2.14, respectively. The water level at each site at the time of the ice survey is shown as well. At points where both an ice and ground elevation were surveyed, more than 8-10% of the ice is above the water surface which indicates that the bottom of the ice is resting on the bank. The fact that the ice elevations are approximately parallel to the ground elevations also supports this assumption. The differences between these points were used to estimate the total ice thickness.

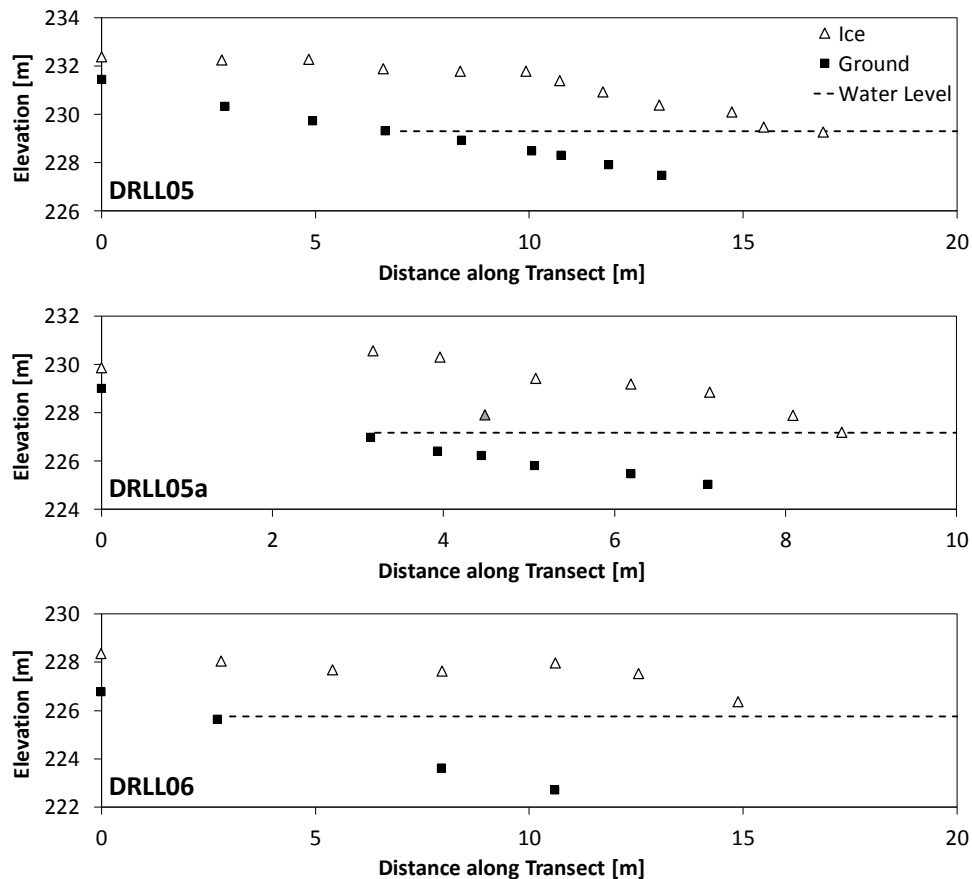


Figure 2.13. RTK transect surveys at sites DRLL05 (top), DRLL05a (middle), and DRLL06 (bottom) performed in spring of the 2016-2017 season. Water levels at the time of the ice transect surveys are shown. The shaded marker at DRLL05a indicates the elevation at the bottom of a shear crack.

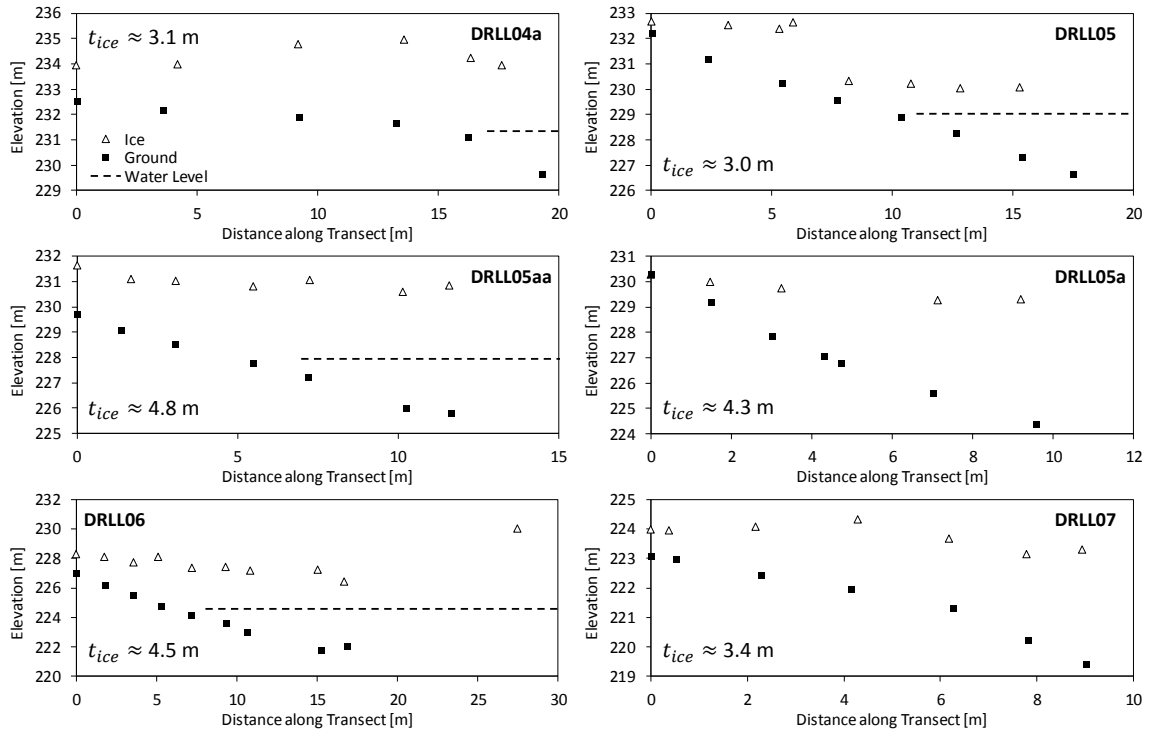


Figure 2.14. RTK transect surveys at performed in spring of the 2017-2018 season. Water levels at the time of the ice transect surveys are shown.

2.2.5 Analysis and Discussion

2.2.5.1 Observations of Ice Cover Advancement and Secondary Consolidation Events

Site visits on December 9, 2016 and December 12-14, 2016 were timed to coincide with ice cover progression up the Lower Dauphin River. Qualitative observations made from shore and with UAV imagery were vital in supplementing the quantitative data from the deployed instruments. A simplified, conceptual schematic of the observed ice cover progression is shown in Figure 2.15. In general, at a given time, the ice cover appeared thicker in the downstream direction. This is simplified dramatically in Figure 2.15 by a number of stable ice regions, each with their own average thickness (t_i denotes the thickness in Region i). Observations indicated that a well-defined leading edge did not

exist at the upstream end of the stable cover. Instead, there was a zone where incoming ice transitioned from free-moving pans to a thin, stationary cover (upstream of Region 1 in Figure 2.15).

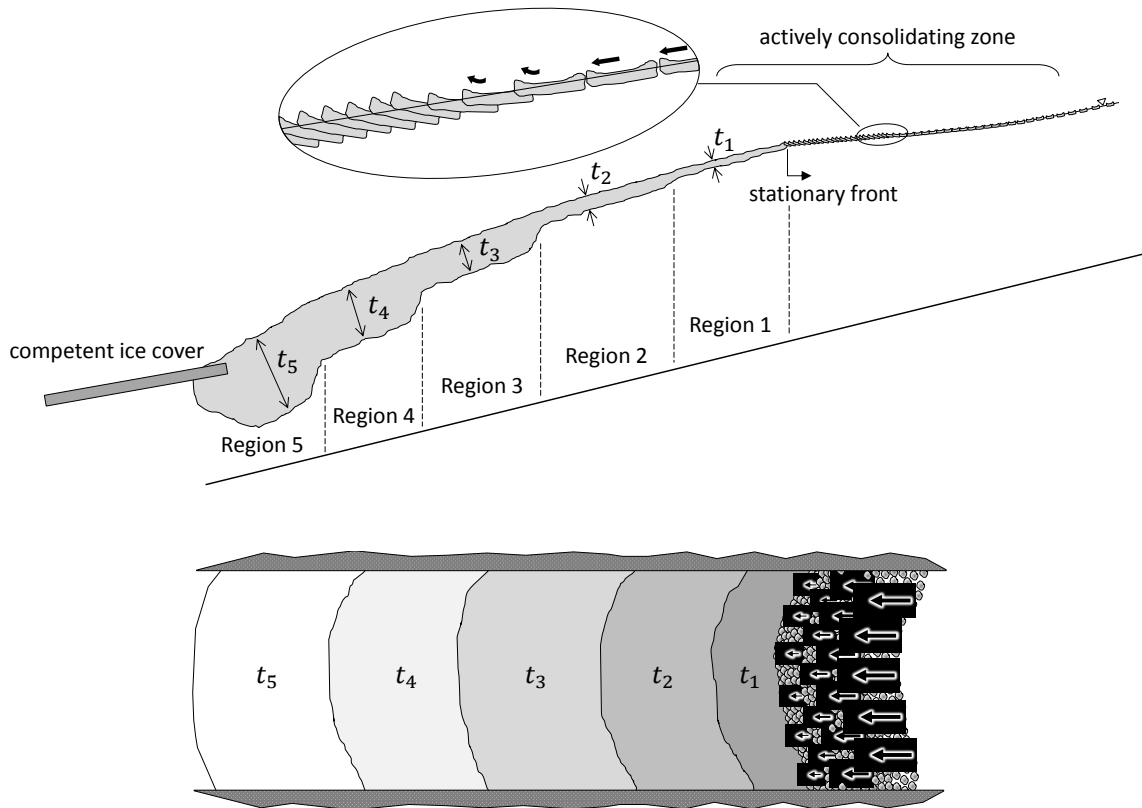


Figure 2.15. Conceptual schematic of observed ice cover progression on the Lower Dauphin River. Profile view (top) and plan view (bottom).

The incoming ice had a surface concentration of nearly 100% and took the form of relatively small pans, estimated to be on the order of 5-7 m² in surface area. The pans had upturned edges, sometimes referred to as “pancake ice” in the literature (Prowse, 1995), and were partially frozen together. As the pans moved nearer to the stationary front, they began to decelerate. Faster moving pans from upstream would ride up onto the slower moving pans downstream, thereby creating a moving cover that consisted of partially

stacked pans. This moving ice would continue downstream until it contacted the stationary front, which was often parabolic in shape as shown in Figure 2.16.

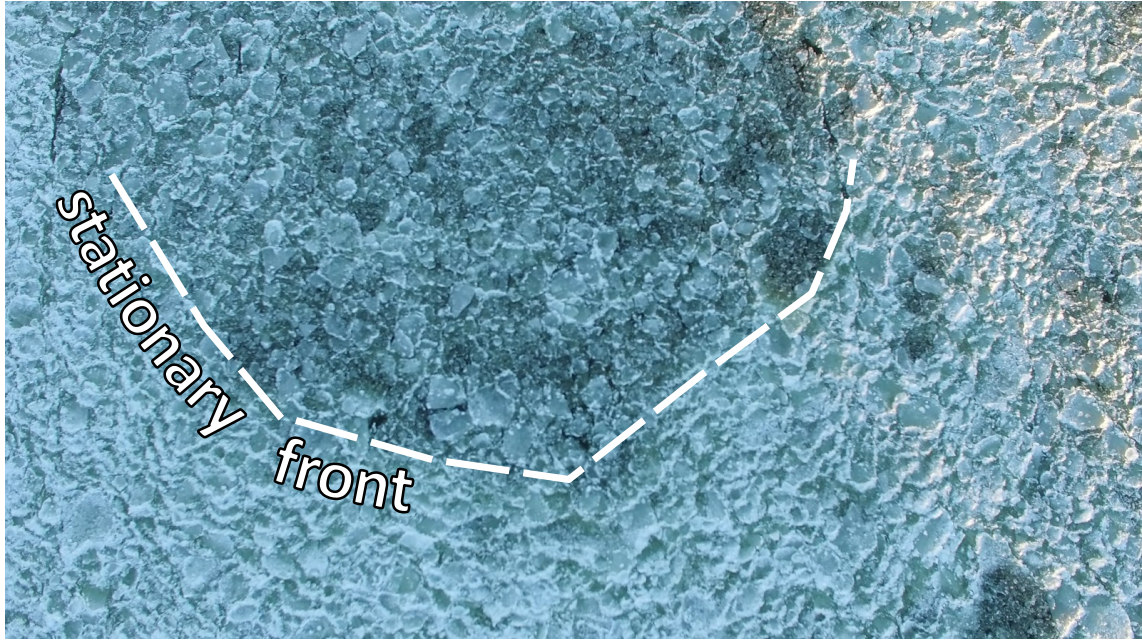


Figure 2.16. Aerial view of the actively consolidating zone taken with UAV. Flow is from top to bottom. The stationary front can be seen in the middle of the image, taking on a parabolic shape.

As incoming pans contacted the stationary front, they ridged and packed together, forming a hummocky ice cover shown in the bottom portion of Figure 2.16. Pans were never observed to overturn or sink to be transported beneath the stationary cover. The stationary front would move upstream while remaining thin (estimated to be not more than a few pans thick) at its most upstream edge, thereby lengthening Region 1. Suddenly, the thin stationary cover would mobilize, and the stationary front would relocate downstream where the ice was thicker (between Region 1 and Region 2). The newly mobilized cover (Region 1) would then pack against the newly established stationary front, again allowing it to move upstream (effectively lengthening Region 2). The most upstream regions of the stationary cover would lengthen via cycles of cover advancement and small, frequent shove

events which occurred approximately every 20 minutes or less. This region of the ice cover was called the “actively consolidating zone” and is similar to the description of “primary consolidations” by Andres et al. (2003) and the explanation of local packing that can cause the stationary front to move back and forth in large rivers by Michel (1984). If a shove event in the actively consolidating zone caused a region downstream (which had been observed to be stable for several hours) to become unstable, either from the added weight of the newly collapsed cover or from the destabilizing effects of the ice momentum and water surge, a secondary consolidation event would occur. The consolidating front moved in the downstream direction as portions of previously consolidated cover mechanically thickened even further. Secondary consolidation events were typically associated with the release of a large surge of water downstream which could be detected in loggers several kilometres away. Close inspection of the meteorological data collected, specifically air temperature and net radiation, did not reveal a clear correlation between atmospheric conditions and the timing of the consolidation events. Therefore, the shoves were evidently triggered by mechanical factors that depend on a complex relationship between the rate of cover lengthening and rate of strengthening from freezing, and not only strength reduction from thermal decay.

2.2.5.2 2015-2016 Freeze-up: Summary of Ice Cover Progression and Major Secondary Consolidation Events

The freeze-up period in 2015-2016 began on November 19 when air temperatures began to dip below 0°C (see Figure 2.6). Trail camera images showed that a stable ice cover was attained at DRLL10 late on November 25 and reached DRLL08 about one day later. The presence of an ice cover had little effect on the water levels at DRLL10 and

DRLL09 because of the backwater effects of Lake Winnipeg. The water levels at DRLL08 and DRLL07 rose approximately 6.3 cm/hr and 8.2 cm/hr, respectively, as the ice cover progressed past each site. Open leads developed approximately on the morning of December 1 and remained for about 15 days due to warm weather. The development of open water leads are signified in the water level records at DRLL07 and DRLL08 (Figure 2.8) by the transition from jagged lines with rapid fluctuations to smooth lines with more gradual changes (the fluctuations caused by the actively consolidating zone were absent during the warm spell). On December 15, air temperatures dropped again and ice began to appear in the trail camera photos at the upstream sites of DRLL04, DRLL05, and DRLL06. A stationary ice cover formed at DRLL08 at about 12:00 pm on December 16 and overnight from December 16-17 at DRLL07. A stationary ice cover formed at DRLL06 overnight from December 18-19. The cover at DRLL06 remained in the same configuration for the full day on December 19 while the backwater effects began to increase water levels at DRLL05. The saw-toothed pattern of the water levels at DRLL05 and fluctuations at downstream loggers is indicative of the actively consolidating zone undergoing small shove events, as shown in Figure 2.8.

A major secondary consolidation event occurred in the morning of December 20, 2015. A zoomed in view of water levels at this time is shown in Figure 2.17. Trail camera photographs taken on the evening of December 19 and the morning of December 20 at DRLL05 and DRLL08 are shown in Figure 2.18. The water level at DRLL05 decreased 2.9 m over approximately 40 minutes beginning at 2:04 am. The ice condition at this site changed from a thin, moving ice cover with nearly 100% surface concentration to a mostly open channel with ice attached at the banks. Trail camera images of DRLL06 showed that

the cover transitioned from a relatively thick, stable, consolidated cover to a thin, moving cover. This indicated that the consolidated cover completely mobilized at DRLL06 and moved downstream. The water level at DRLL07 showed a sudden increase at 2:20 am, when it jumped 2.5 m over the span of eight minutes. At DRLL08, the water level rose 2.1 m over a span of 16 minutes. The water levels at DRLL07 and DRLL08 remained elevated following the shove, indicating that the cover mechanically thickened in these regions. The trail camera images of DRLL08 in Figure 2.18 corroborate this; the cover became visually rougher and elevated. At DRLL09, the water level rose 0.3 m, but then returned to its pre-shove level. A similar water level response was observed at DRLL10. This indicates that no significant mechanical thickening occurred at these regions, but the surge from the shove event temporarily increased flow and water levels.

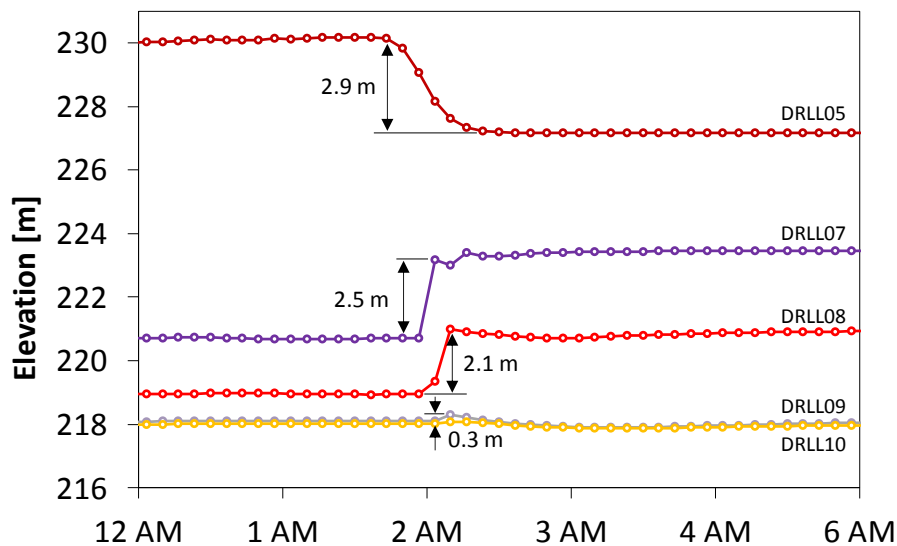


Figure 2.17. Water levels during shove event on December 20, 2015. Measurements taken at eight minute intervals.

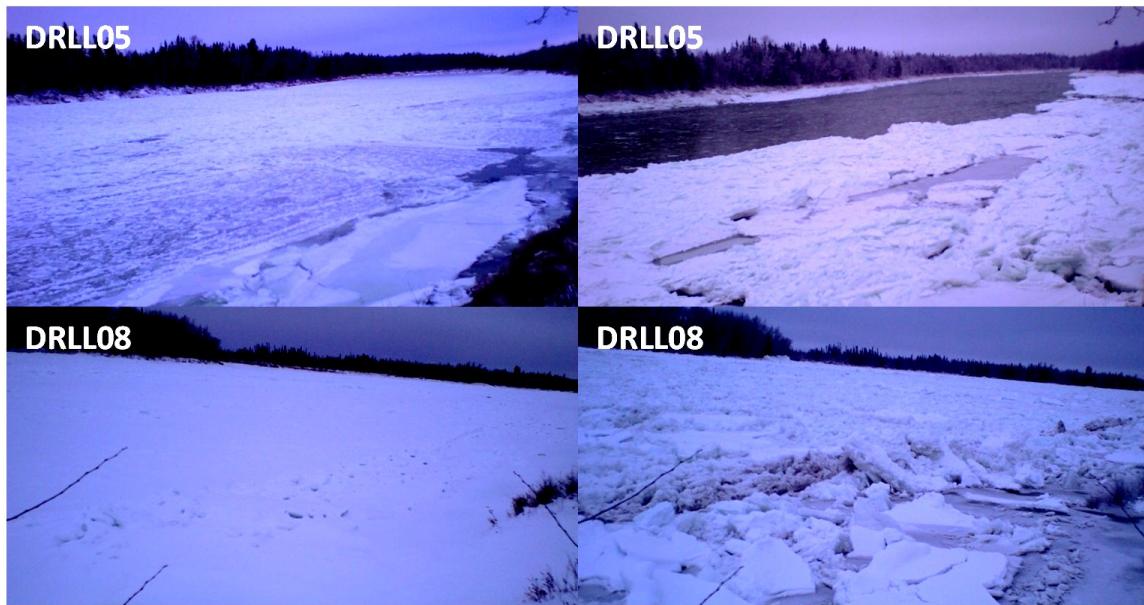


Figure 2.18. Trail camera photographs taken at DRLL05 and DRLL08 before (left) and after (right) the secondary consolidation event on the morning of December 20, 2015.

Immediately after the event, conditions at DRLL06 returned to those characteristic of the actively consolidating zone, while the majority of the channel at DRLL05 was open and transporting surface frazil ice. The ice cover then began advancing again, and changed from actively consolidating to a stationary cover at DRLL06 overnight from December 22-23. In the evening of December 24, trail camera images showed a secondary consolidation event occurred at DRLL06. Unfortunately the water level logger at DRLL06 was not retrieved to corroborate the relative magnitude of this event, but the water level at DRLL05 shows a sharp decrease of 0.6 m over 16 minutes at 11:48 pm on December 24. The cover at DRLL05 became stationary overnight from December 25-26, while a stationary cover reached DRLL04 at about 5:00 pm on December 26.

2.2.5.3 2016-2017 Freeze-up: Summary of Ice Cover Progression and Major Secondary Consolidation Events

The freeze-up period in 2016-2017 was more condensed than in 2015-2016 because air temperatures remained well below 0°C for the duration of freeze-up. The ice cover began advancing up the Lower Dauphin River on the afternoon of December 8, when a stationary cover was established at DRLL10. The stationary cover reached DRLL08a and DRLL08 on the morning of December 9. By early afternoon, the cover had attained a stable configuration at DRMET and DRLL07.

A secondary consolidation event occurred at approximately 2:50 pm on December 9, 2016 while the authors were conducting a site visit. The water levels at selected sites during this event are shown in Figure 2.19. At 2:50 pm, the water level at DRLL06b began to decrease; at about the same time levels at DRLL07 and DRMET increased abruptly. A burst of three photographs at a 30 second interval with the field camera at DRMET indicated that the consolidating front progressed from upstream to downstream. The water level at DRMET attained a peak elevation of 221.47 m, but then receded to 221.36 m. This brief peak was likely caused by the temporary increase in discharge caused by the surge of water during the consolidation event. A similar response is shown at DRLL07. At DRLL08 and DRLL08a, the water levels indicate that the surge of water from the shove passed these sites without thickening the cover.

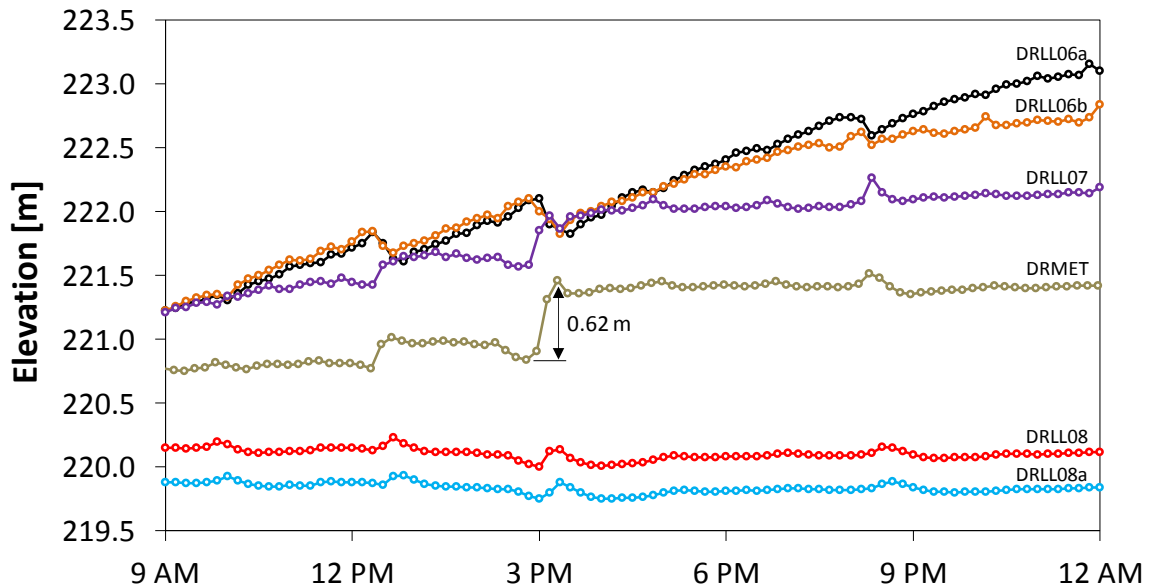


Figure 2.19. Water levels near DRMET during secondary consolidation event on December 9, 2016. Measurements taken at 10 minute intervals.

A UAV photograph taken at 3:30 pm after the event is shown in Figure 2.20. The distinct change in the roughness and apparent thickness of the cover supports the notion that the downstream extent of the shove was between DRMET and DRLL08. A more detailed account of this event, including visual observations from shore, can be found in Wazney et al. (2017).



Figure 2.20. UAV photograph taken at 3:30 pm on December 9, 2016. View is looking downstream from site DRMET. The ridge in the ice cover indicates the downstream extent of the shove event.

Following this event, the stationary front of the ice cover again began advancing upstream from DRLL07. Several more shove events occurred as the ice cover advanced up the river, as evidenced by the saw-toothed patterns of the water elevations in Figure 2.9. However, the water levels indicate that no additional mechanical thickening occurred at DRLL07, suggesting that these events occurred upstream of this site (also supported by the ice profiles in Figure 2.11). Figure 2.21 highlights two major secondary consolidation events that took place during the site visit from December 12-14, 2016, allowing detailed visual accounts of the ice conditions before and after these events.

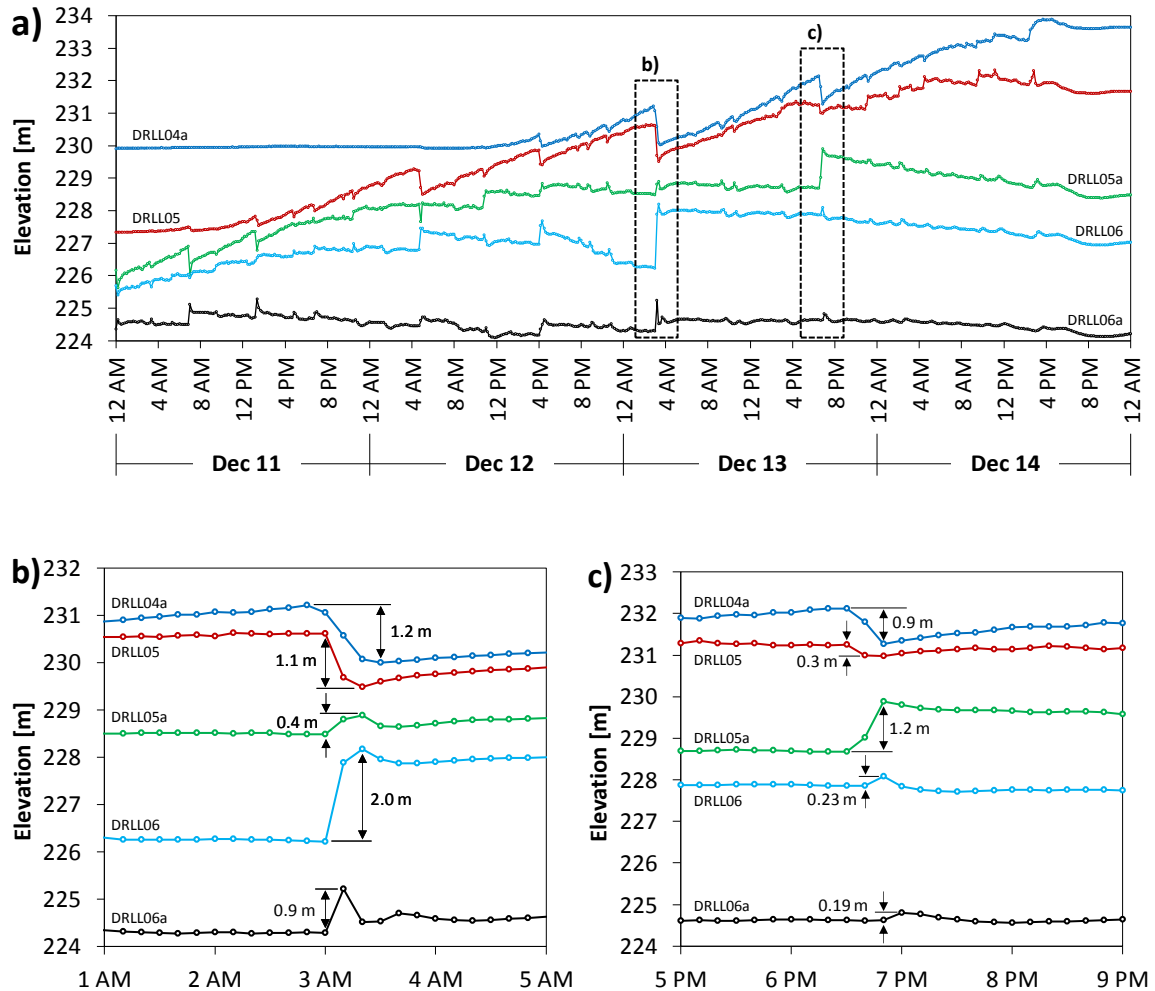


Figure 2.21. Water levels near DRLL05a; a) from December 11-14, 2016, b) during secondary consolidation event on morning of December 13, and c) during secondary consolidation event on evening of December 13. Measurements taken at 10 minute intervals.

The event on the morning of December 13 (Figure 2.21b) was likely initiated from upstream near DRLL04a and DRLL05. The water level at DRLL04a first began to decline at 2:50 am. The responses in the water level loggers downstream were delayed, not showing signs of the shove until 3:00 am; a smaller shove near the upstream end of the stationary ice cover may have caused a sequence of shoves that propagated downstream. The water level at DRLL06 increased 2.0 m over 20 minutes, indicating that the cover mechanically thickened near this site (see Figure 2.22). Over the same time period, the

water level at DRLL05 decreased 1.1 m. The water level at DRLL06a showed a temporary increase of 0.9 m as the surge passed and remained about 0.2 m higher than the pre-shove level, indicating some roughening or thickening of the ice cover in this area.



Figure 2.22. Consolidated ice cover at DRLL06 on December 12, 2016 (left) and the morning of December 13, 2016 (right). The cover became visibly rougher due to the shove event on the morning of December 13.

The secondary consolidation event on the evening of December 13 (Figure 2.21c) also appears to have been initiated from upstream. Water levels at DRLL04a and DRLL05 decreased by 0.9 m and 0.3 m, respectively. Major thickening during this event occurred at DRLL05a; water levels increased by 1.2 m over 20 minutes at this site. The ice cover at DRLL06 and DRLL06a had sufficient strength to resist mechanical thickening during this event. Figure 2.23 shows photographs of ice conditions before and after the shove event at selected sites.

The stationary ice cover reached sites DRLL05 and DRLL04a on the morning and afternoon of December 14, respectively. Interestingly, the water levels show a distinct pattern change at about 6:00 pm on December 14, when they show less rapid fluctuations and instead vary more gradually (Figure 2.9). This is because the ice cover progression up the upper reach of the Dauphin River is less dynamic and does not include an actively consolidating zone or secondary consolidation events.

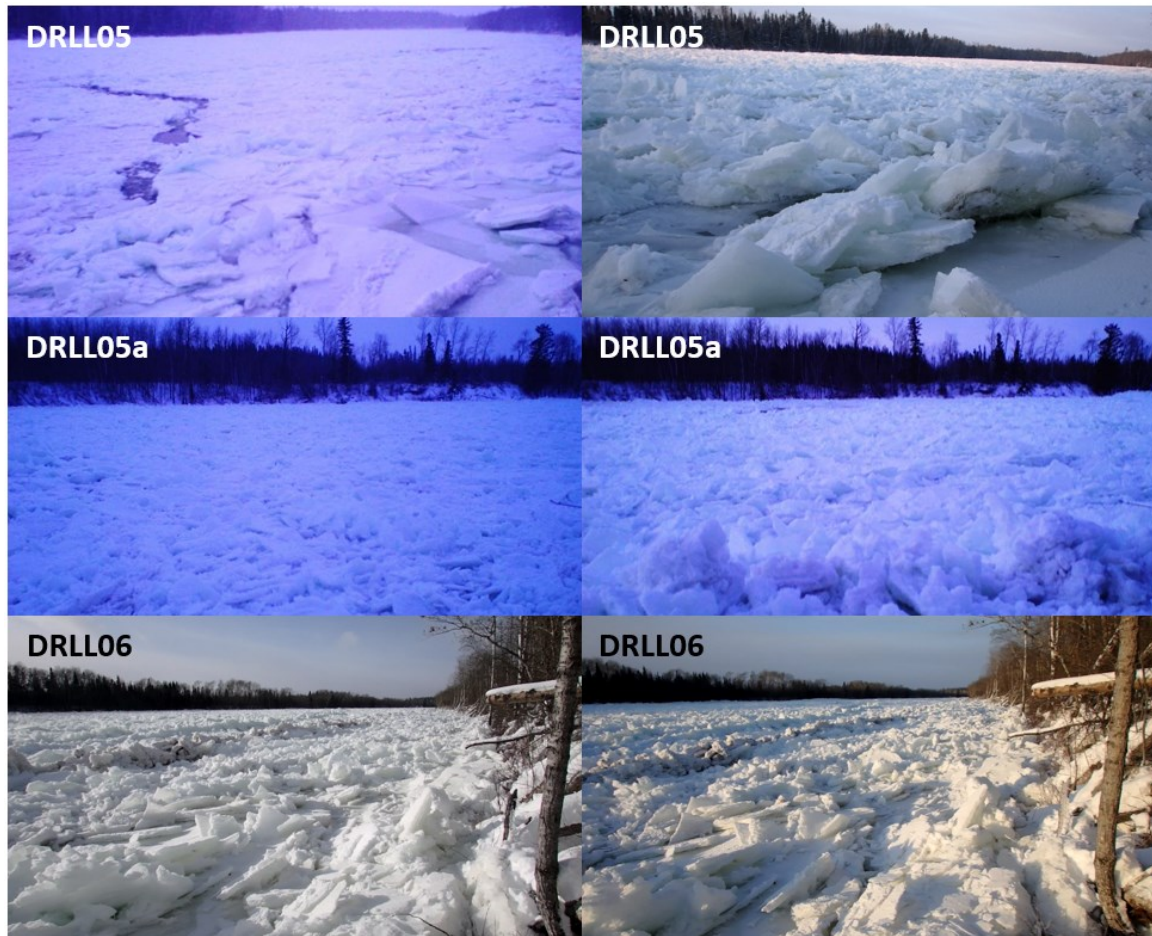


Figure 2.23. Ice conditions at DRLL05 (top), DRLL05a (middle), and DRLL06 (bottom) before and after the secondary consolidation event on the evening of December 13, 2016. Note the ice appearance changes at sites DRLL05 and DRLL05a, while at DRLL06 it remains the same.

2.2.5.4 2017-2018 Freeze-up: Summary of Ice Cover Progression and Major Secondary Consolidation Events

The ice cover began progressing upstream from Lake Winnipeg on November 8 in the 2017-2018 season. The timing corresponds to just over 50 DDOF, much like the other two monitoring seasons. The actively consolidating zone quickly reached DRLL08 on the morning of November 9. A shove event in the afternoon of November 9 caused recession of the ice cover downstream and the corresponding decrease in water levels at sites

downstream of DRMET. The cover progressed from Lake Winnipeg to site DRLL05 (just under 9 km) from November 9-26, going through several cycles of frontal progression and consolidation as witnessed in the previous monitoring seasons. A warm spell from November 26 to December 5 slowed the rate of advancement of the cover; water levels at sites DRLL06, DRLL06a, and DRLL06b decreased slowly during this time, due to some smoothening of the underside of the ice cover and development of open water leads. Air temperatures dropped again on December 5 and the ice front continued to progress upstream. A major consolidation event occurred late on December 5, causing the water level to increase by 2.3 m in approximately 16 minutes at DRLL06 as shown in Figure 2.24. Two additional water level loggers, placed approximately 200 m and 400 m downstream of site DRLL06, were configured to take measurements every minute from December 1-28. Water levels at these logger locations were not surveyed during the sampling period, thus the water surface elevations are assumed for plotting purposes. The higher frequency measurements show that the total increase in stage was caused by two separate upward steps, and that the staging progressed from downstream to upstream.

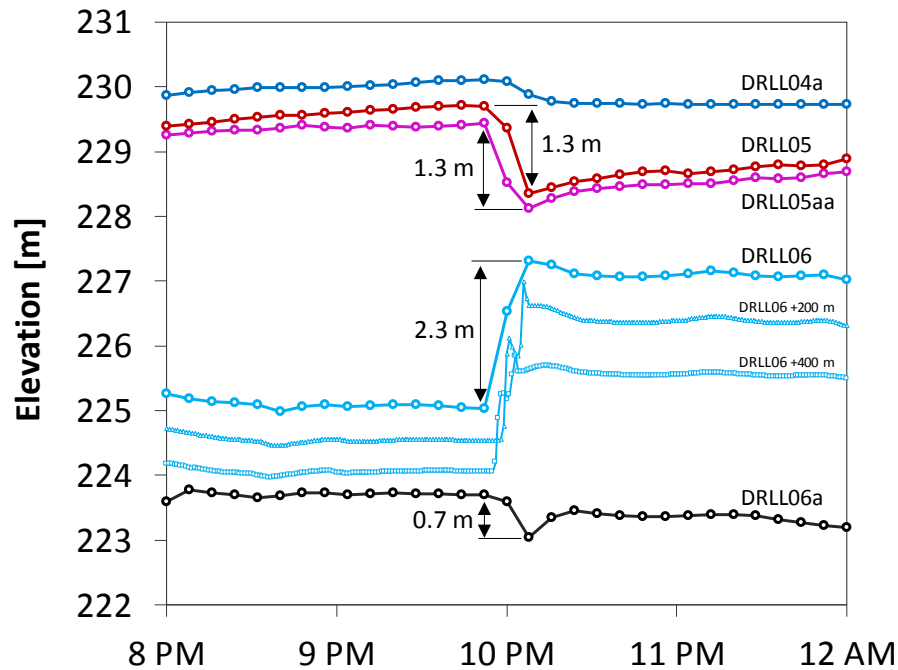


Figure 2.24. Water levels during shove event on December 5, 2017. Measurements taken at eight minute intervals. Two additional loggers recorded water levels at a one minute interval downstream of site DRLL06.

Following the consolidation event on December 5, the high sampling frequency loggers near sites DRLL06 and DRLL07 recorded several waves caused by upstream consolidations, which were observed in trail camera images at DRLL05a; Figure 2.25 shows water levels from December 6-8 at these locations. The larger waves correspond to more significant shoves as captured in the trail camera images at DRLL05a. These events appear to occur approximately every 6-8 hours. Based on the timing of the peaks of each wave near DRLL06 and DRLL07, the average time for the waves to travel 2.4 km was 31 minutes. This equates to an average wave peak celerity of 1.3 m/s. The smaller fluctuations are likely caused by the smaller shoves localized at the leading edge of the ice cover which occur more frequently.

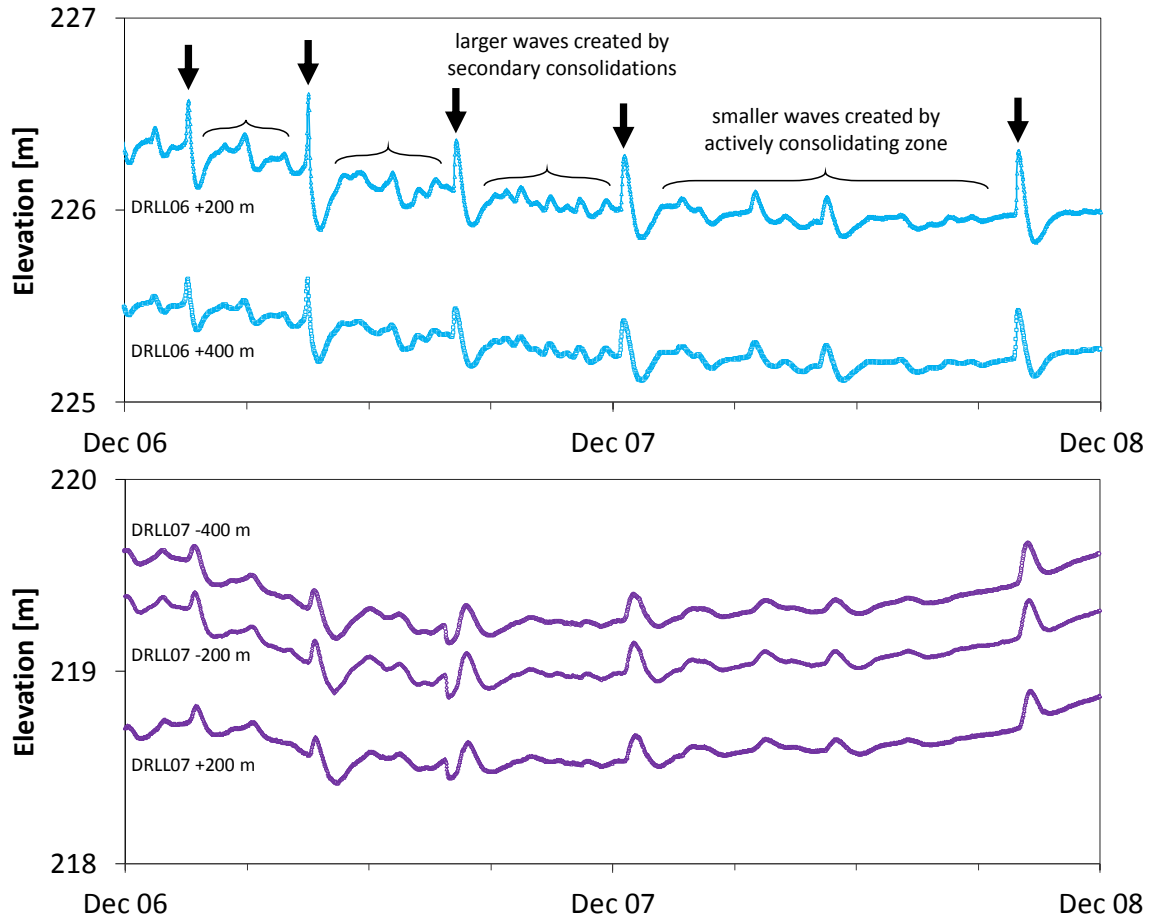


Figure 2.25. Water levels from high sampling frequency loggers at DRLL06 and DRLL07 showing effects of consolidation events and propagation of waves downstream.

Another consolidation event occurred overnight from December 8-9, causing water levels at sites DRLL05aa and DRLL05a to increase by 1.1 m and 1.0 m, respectively. The most drastic step in water levels from a consolidation event during the 2017-2018 freeze-up period occurred at DRLL05aa on December 10. As shown in Figure 2.26, the water levels at sites DRLL05aa and DRLL05 rose by 2.6 m and 1.8 m, respectively. The water level at DRLL06 decreased temporarily for approximately 40 minutes as water entered storage upstream of the thickened reach. The lower peak of this storage effect occurred simultaneously for the other gauges sampling at one minute intervals near DRLL06.

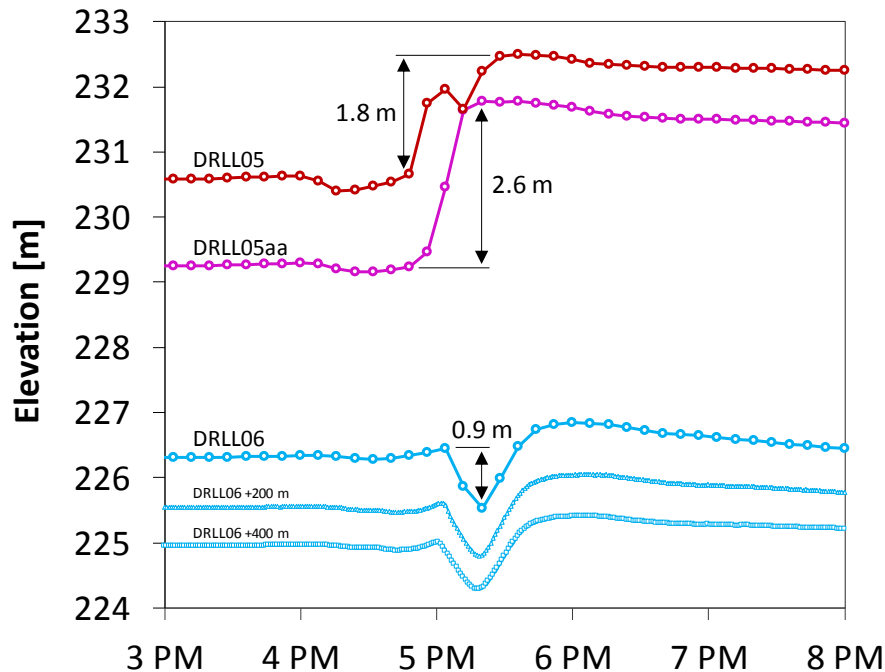


Figure 2.26. Water levels during shove event on December 10, 2017. Measurements taken at eight minute intervals. Two additional loggers recorded water levels at a one minute interval downstream of site DRLL06.

A dynamic ice event in the early morning of December 11 caused a temporary decrease in water levels at sites DRLL05aa and DRLL05, while the water level at DRLL06 spiked up by 1.4 m over 8 minutes, as shown in Figure 2.27. Based on the water level records, a plausible sequence of events is as follows. First, a section of the ice cover between DRLL05aa and DRLL06 released. The sudden removal of the resistance to flow caused a negative wave to travel upstream as water came out of storage. This is why the water level at DRLL05aa decreases before the level at DRLL05. The mobilized ice and surge of water moved in the downstream direction. The arrival of the leading edge of this surge wave can be seen in the loggers with measurements at one minute intervals. The surge wave had a destabilizing effect on the ice cover; the added shear stress and sudden increase in water level which lifted the cover away from the banks made it more prone to

collapse. As portions of the ice cover released near DRLL06, the water levels at these gauges decreased. The water levels at DRLL06 increased again as the flow continued to increase due to the initial release of the ice cover upstream. After decreasing during the trailing edge of the surge wave, water levels near DRLL06 leveled out to elevations slightly higher than the pre-event elevations. Water levels at DRLL05aa and DRLL05 also increased after the event as the ice cover continued to build up and cause staging.

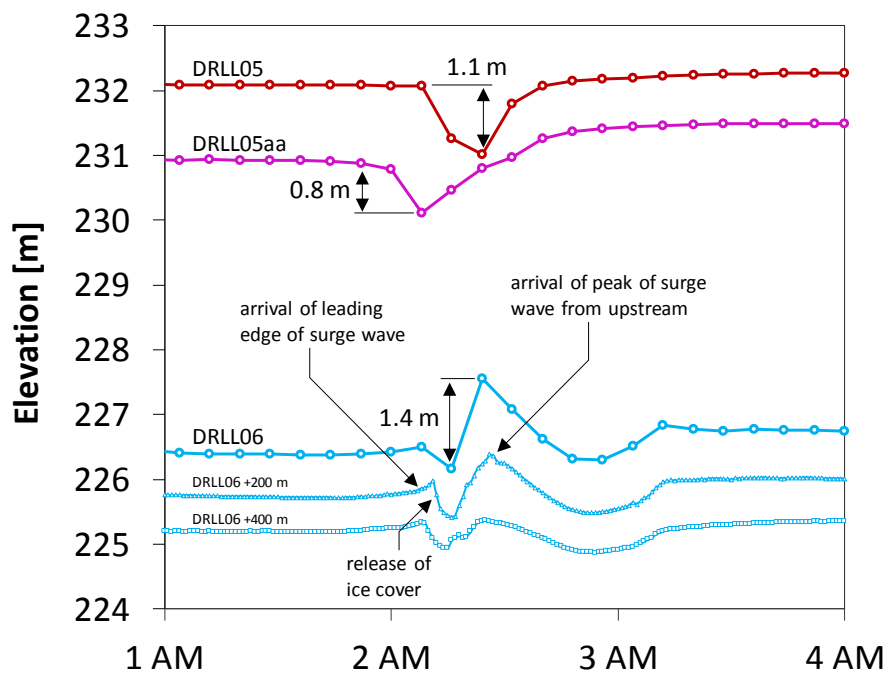


Figure 2.27. Water levels during shove event on December 11, 2017. Measurements taken at eight minute intervals. Two additional loggers recorded water levels at a one minute interval downstream of site DRLL06.

The ice at DRLL06 was observed to be piled extremely high along the banks after this event and was likely grounded in many areas. An open water section, which was present in the middle portion of the channel prior to the event, was wider following the event likely from the release of the ice cover in this area. The open portion likely conveyed the majority of the flow at this time. Surveyed points indicate that the peak of the pile of

ice rubble was approximately 9 m above the channel bed. Photographs of these observations are shown in Figure 2.28 and Figure 2.29.



Figure 2.28. Trail camera images of the ice cover at DRLL06 before (left) and after (right) the event on the morning of December 11, 2017.



Figure 2.29. Ice cover appearance at site DRLL06 on December 13, 2017. Left photograph was taken with the UAV looking upstream. Note the ice ridge on the right side of the image. Right photograph is taken from the bank, also looking upstream.

Continued dynamic formation of the ice cover led to flooding of PR 513 upstream of DRLL04 on December 14, as shown in Figure 2.30. The water overtopped the road in several locations in the area between DRLL04 and DRLL03 as the ice cover advanced through this reach. The town of Dauphin River was evacuated for several weeks until the water receded and the ice was cleared from the roadway to allow access to the community.



Figure 2.30. Overtopping of PR 513 on December 14, 2017 caused by ice-affected staging.

The ice cover progressed beyond DRLL04 on December 15, corresponding to approximately 380 DDOF. This value was much larger than in 2016-2017 (approximately 200 DDOF) and in 2015-2016 (approximately 300 DDOF), suggesting that the volume of ice comprising the ice cover in 2017-2018 was greater than in the previous two seasons.

2.2.5.5 Comparison of Ice Cover Advancement and Effects on Channel Hydraulics in 2015-2016, 2016-2017, and 2017-2018

The general mode of ice cover advancement up the Lower Dauphin River was similar for the 2015-2016, 2016-2017, and 2017-2018 freeze-up seasons. Typically, the water level at a given site would remain relatively constant until the backwater effects of the advancing ice cover reached that location. The water level would then rise with intermittent downward steps; this was indicative of the presence of the actively consolidating zone. Once a stable, stationary cover developed at the site, the water level

stopped increasing and flattened out. If the cover mechanically thickened thereafter, the water level showed an upward step and the water would remain at the elevated level.

Relatively speaking, 2015-2016 was a warmer (-9°C), lower flow ($130\text{ m}^3/\text{s}$) freeze-up, while 2016-2017 was a colder (-21°C), higher flow ($210\text{ m}^3/\text{s}$) freeze-up. The 2017-2018 season had an average air temperature similar to 2015-2016 (-9°C), while the average flow was similar to 2016-2017 ($220\text{ m}^3/\text{s}$). Net staging due to ice effects in all monitoring seasons are shown in

Table 2.2. The pre-freeze-up levels correspond to the water elevations at the approximate time DRLL10 developed a stationary ice cover. The pre-freeze-up levels at most gauge locations were higher in 2016-2017 and 2017-2018 than in 2015-2016 due to the higher flow. The low level of Lake Winnipeg in 2017-2018 caused the pre-freeze-up levels downstream of DRMET to be lower than in 2016-2017.

Net staging downstream of DRMET was greatest in the 2017-2018 season. This could be due to the higher water surface gradient in this area due to the low lake level and the lower average air temperature during freeze-up which limited the ability of the forming ice cover to gain non-frictional strength. Staging at DRLL08 and DRLL07 was higher in 2015-2016 than in 2016-2017 due to the large secondary consolidation event that occurred later in the season, on December 20, 2015. At sites DRLL06b, DRLL06a, and DRLL06, staging in 2016-2017 was comparable, but slightly higher, than in 2017-2018. The top-of-ice profiles (Figure 2.11) indicate that the ice rubble was piled up higher in 2017-2018 than in 2016-2017 at these locations, despite the lower peak water levels. It is possible that the sampling interval of the water level loggers missed the peaks of the events that caused the

piled up rubble in 2017-2018. Alternatively, the rubble could have been piled up along the banks by the flow during shove events without the water level achieving the same peak elevations. The highest recorded staging in 2016-2017 occurred at DRLL06, where the ice covered water surface reached 5.65 m higher than the pre-freeze-up level. Top-of-ice elevations (Figure 2.11) show that the water elevation likely peaked at approximately the same level in 2015-2016 at this site (unfortunately the logger was lost in this season so staging could not be calculated). Water level records indicate that this site was subjected to a large secondary consolidation event on December 13, 2016, as well as several smaller shove events during freeze-up. Staging at DRLL05 was only slightly higher in 2016-2017 than in 2015-2016, even though flows were nearly double. This could be due to the nature of consolidation events at this site in each year; 2015-2016 included a large shove event on December 26, 2015, which probably over-thickened the cover beyond that from the steady state discharge. Less severe shove events occurred at DRLL05 in 2016-2017, possibly due to the lower air temperature allowing for more rapid thermal strengthening of the forming ice cover. Net staging at DRLL05 in 2017-2018 was the highest on record for the three seasons; the peak water surface elevation was 5.73 m above the pre-freeze-up level. Once again, it is likely that the mild air temperature and high flow of this season made the forming cover more susceptible to shoving and over-thickening. At site DRLL04a, staging in 2017-2018 was over a metre higher than in 2016-2017, while similar staging was recorded at DRLL04 for the two seasons. At DRLL03, staging of 1.42 m, 1.72 m, and 2.46 m were recorded in 2015-2016, 2016-2017, and 2017-2018, respectively. The elevation of PR 513 in this area is approximately 239.35 m.

The largest secondary consolidation event in 2015-2016 occurred on December 20, 2015. The downstream extent of this event was between DRLL08 and DRLL09. At the time of the consolidation, the stationary front was approximately at DRLL05, meaning that about 7.5 km of ice cover consolidated to about 4.7 km (the stationary front receded to just downstream of DRLL06). The largest secondary consolidation event in 2016-2017 occurred on the morning of December 13, 2016. The downstream extent of this event was likely between DRLL06 and DRLL06a. The stationary front was between DRLL05a and DRLL05 at the time of the event, and did not recede downstream of DRLL05a during the event. This indicates that a cover length of 3.2 km (or less) consolidated to 1.5 km (or more). In either case, the maximum shove-affected reach length of a single secondary consolidation event was greater in 2015-2016 than in 2016-2017.

Table 2.2. Peak water surface elevations (WSE) during the 2015-2016, 2016-2017, and 2017-2018 freeze-up seasons. Net staging was calculated by subtracting the pre-freeze-up water level from the peak water level.

	2015-2016			2016-2017			2017-2018		
	(Q = 130 m ³ /s; T _{air} = -9°C)			(Q = 210 m ³ /s; T _{air} = -21°C)			(Q = 220 m ³ /s; T _{air} = -9°C)		
	Pre-Freeze-up WSE [m] (Nov. 26 12:00 am)	Peak WSE* [m]	Net Staging [m]	Pre-Freeze-up WSE [m] (Dec. 8 12:00 pm)	Peak WSE* [m]	Net Staging [m]	Pre-Freeze-up WSE [m] (Nov. 5 12:00 am)	Peak WSE* [m]	Net Staging [m]
DRLL09	217.79	218.24	0.45	218.17	218.72	0.55	217.57	219.65	2.08
DRLL08a				218.48	219.90	1.42	217.73	221.26	3.53
DRLL08	218.01	221.01	3.00	218.50	220.18	1.68	217.86	221.30	3.44
DRMET				218.66	221.36	2.70	218.44	221.70	3.26
DRLL07	218.74	223.39	4.65	219.52	222.09	2.57			
DRLL06b				220.37	223.62	3.25	220.42	223.62	3.20
DRLL06a				221.10	224.85	3.75	221.23	224.90	3.67
DRLL06				222.23	227.88	5.65	222.51	227.55	5.03
DRLL05a				224.32	229.69	5.37			
DRLL05aa							226.51	231.97	5.46
DRLL05	227.06	231.81	4.75	227.30	232.13	4.83	227.47	233.20	5.73
DRLL04a				229.85	233.87	4.02	229.95	235.07	5.12
DRLL04	232.00	234.24	2.24	232.18	234.51	2.33	232.37	234.57	2.21
DRLL03	237.37	238.79	1.42	237.60	239.32	1.72	237.78	240.24	2.46

*passing of surges not considered

2.2.5.6 Top-of-Ice Profiles and Thickness Measurements

Referring to Figure 2.11, the top-of-ice profiles showed some differences in the three monitoring seasons. The profile measured in 2015-2016 is shallow from Lake Winnipeg to a location just downstream of DRLL08a, where it suddenly becomes very steep. This is the location of the downstream extent of the major secondary consolidation event that occurred on December 20, 2015 (see Figure 2.17). The 2016-2017 profile shows a more gradual increase in ice elevation from Lake Winnipeg to DRLL08. Between DRLL08 and DRMET, a steeper ice profile indicates the downstream extent of the secondary consolidation event on December 9, 2016. From DRLL08a to DRLL06, the ice elevations in 2015-2016 were greater than 2016-2017. This is likely due to the one large secondary consolidation event on December 20, 2015 that caused the cover in this region to mechanically thicken. In 2016-2017, a number of smaller shove events resulted in a thinner, consolidated ice cover in this region. Referring to Figure 2.9, the shove events after December 9, 2016 appeared to have a small effect on water levels at DRLL07; the downstream extents of these events were upstream of DRLL07, resulting in a steep profile near DRLL06b and DRLL06a. Ice elevations at DRLL06 were approximately the same in both years. Upstream of DRLL06, the profile in 2016-2017 is higher than 2015-2016, likely due to the higher flows.

The top-of-ice profile in 2017-2018 is noticeably higher than the other two seasons' profiles at several locations along the Lower Dauphin River. It is clear that the cover near Lake Winnipeg thickened the most in this season. The 2017-2018 profile was also higher near DRLL07, and the steep ice profile just downstream of DRLL07 indicates that a consolidation toe may have been located nearby. Ice elevations near DRLL06a were

slightly lower in 2017-2018 than in 2016-2017. Another potential toe location of consolidation events in 2017-2018 is near DRLL06 where the ice profile is very steep. Ice elevations are clearly highest in 2017-2018 in the area between DRLL06 and DRLL04a. The top-of-ice profile in 2017-2018 was extended upstream beyond DRLL03, and in several locations the ice elevation was higher than the elevation of PR 513, indicating flooding.

Surveyed top-of-ice and ground elevations in 2016-2017 in Figure 2.13 provide an estimate of the ice thickness. Results show that the thickness ranged from 2.5-3.2 m at DRLL05, 3.6-3.9 m at DRLL05a, and 4.0-5.3 m at DRLL06. The relative thicknesses agree with the nature of the consolidation events identified in the water level records at these sites; referring to Figure 2.9, the water level record at DRLL06 shows several distinct steps, indicating mechanical thickening of the cover during multiple secondary consolidation events. Fewer steps are shown in the record at DRLL05a, and even fewer at DRLL05. This corroborates the notion that secondary consolidation events cause over-thickening of the ice due to increased shear stress during the accompanying surge and the momentum of the mobilized ice cover.

Surveyed ice and ground transects in 2017-2018 in Figure 2.14 indicate that the ice cover was thickest in the region between DRLL05aa and DRLL06 (approximately 4.3-4.8 m thick). The cover was approximately 3.4 m thick near DRLL07, where the ice profile appeared steeper which could indicate a toe/grounding location of the ice cover. At DRLL05 and DRLL04a the ice cover was estimated to be approximately 3.0 m and 3.1 m thick, respectively.

2.2.6 Conclusions

Collected data and visual observations during the 2015-2016, 2016-2017, and 2017-2018 freeze-up seasons indicate that the ice cover advanced up the Lower Dauphin River through a series of consolidation events in all three years. The leading edge of the ice cover was typically not well defined; a distinct boundary between ice cover and open water did not exist, except for periods of warm weather and reduced ice production. Instead, the area upstream of the stationary cover was a transition zone where free-drifting ice floes from upstream decelerated as they approached the stationary front, converging to create a moving ice cover that had nearly 100% surface concentration. As the pans contacted the stationary front, they usually packed together by partially stacking and remained at the surface. A region called the actively consolidating zone was identified near the stationary front. In this zone, the ice cover went through frequent cycles of frontal progression and small, localized shoves.

Secondary consolidation events were also observed, during which a previously stationary section of the ice cover would collapse and mobilize, causing further thickening and roughening of the cover. Several kilometres of ice cover were observed to mobilize during a secondary consolidation event, which sent a surge of water downstream. The resulting combination of the ice momentum and temporary increase in discharge led to increased thickening of the ice cover in the shoved reach. This suggests that simply knowing the steady state discharge during freeze-up will not guarantee an accurate estimate of ice thickness, but the factors influencing the timing and severity of shove events play an important role. Rapid thickening at the downstream extent of the consolidated reach caused water levels to rise very quickly; the most dramatic event recorded measured a water level

rise of 2.5 m over a span of 8 minutes (rise of approximately 0.3 m/min), with several other comparable events in the three seasons of collected data. The surge of water released during a secondary consolidation event was detectable in water level loggers several kilometres downstream of the shoved reach. Further analysis on surge measurements during secondary consolidation events will provide insight on the magnitude and duration of flow increase at each site, the corresponding increase in shear stresses and/or uplift forces acting to destabilize the ice cover, and the required strength of an ice cover to resist collapse under these forces. The three monitored seasons indicate that high discharge coupled with mild air temperatures during the period of ice cover formation lead to thicker ice accumulations and higher ice-affected water levels. This is presumably linked to the ability of the forming ice cover to gain strength via thermal freezing of interstitial water, which increases its ability to resist further consolidation.

The completeness of available and collected data (i.e. bathymetry, discharge, water levels, and meteorological data) facilitates numerical modelling of freeze-up on the Lower Dauphin River. Current numerical ice models can be evaluated on their ability to simulate water levels and ice conditions that were observed at each monitoring site over the three freeze-up seasons. Specifically, the ability of numerical models to capture the shoving events that affect final ice thicknesses can be assessed. Estimated ice thicknesses and surveyed top-of-ice profiles can also be used for further calibration and validation of the models. The data and observations may serve to highlight deficiencies in the capability of state-of-the-art ice models in simulating dynamic freeze-up processes, and rationalize further model development to better capture these processes.

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2.3 Paper 2: Effects of freeze-up consolidation event surges on river hydraulics and ice dynamics on the Lower Dauphin River

2.3.1 Abstract

Ice cover formation in fast-moving rivers during freeze-up involves progression of the leading edge with intermittent collapses that cause rapid consolidation (mechanical thickening) of the ice cover and withdrawal of the leading edge downstream. The release of water from storage during these consolidation events causes a surge wave to travel downstream, resulting in an unsteady flow condition that briefly increases water levels and other concomitant hydraulic parameters that are difficult to measure directly such as discharge, velocity, and shear stress. In this study, the previously published Rising Limb Analysis Method (RLAM) was used to deduce the transient hydraulic properties at several

monitoring sites on the Lower Dauphin River during the release of surge waves caused by freeze-up consolidation events in 2016-2017. It was found that parameters were most greatly amplified near the toe of the consolidated reach, and decayed in the downstream direction. On one occasion, computed discharge was found to be nearly three times the steady state discharge during the passage of a surge wave that caused water levels to rise by 0.93 m in less than 10 minutes at one gauge location. The calculated consolidation stress applied to the ice cover during this event increased by 99 Pa; the peak stress was approximately 2.5 times the unperturbed value. Results of the RLAM application highlight the interdependent relationship between ice dynamics and unsteady flow hydraulics during freeze-up, and the importance of considering the effects of consolidation events when predicting the final ice thickness in fast-moving rivers.

2.3.2 Introduction

The formation of a stable ice cover on fast-moving rivers is a dynamic process that involves cycles of ice front progression and collapse. When an ice cover collapses under externally applied forces, it consolidates and the leading edge of the ice cover retreats downstream. The river conveyance near the upstream end of the consolidated reach suddenly increases with the absence of an ice cover, releasing water from storage. The term “surge” is often used in river ice literature and will be adopted herein to describe the wave of water sent downstream, although the slope of the rising limb is far less than that associated with a classical dam-break type surge (Beltaos, 2013b). The surge moves downstream at a celerity related to the magnitude and rapidity of the consolidation, and temporarily increases discharge and water velocity. As the surge moves into a stable ice-covered reach downstream, it increases the stresses attempting to collapse the ice cover;

the increased water velocity increases the shear stresses acting on the underside of the intact cover while the steeper water surface slope during the rising limb of the surge wave increases the downstream component of the weight of the ice cover. Additionally, the increased water level during the passing of the surge can lift the ice cover away from the river banks and reduce the frictional forces keeping the cover stable. If the heightened driving forces exceed the strength of the ice cover, including frictional interlocking of ice floes and non-frictional resistance provided by the solid ice crust at the surface of the cover, it will collapse and consolidate; an upstream consolidation event can therefore trigger consolidation events farther downstream.

Published data on surge waves created by ice events at freeze-up or breakup are relatively scarce, due to the unknown locations and timing of jam formation and release and cost to deploy instrumentation in remote locations. However, breakup observations are more common, and measurements that have been conducted demonstrate the dramatic effects of ice release events on water levels. Kowalczyk and Hicks (2003) and Kowalczyk Hutchison and Hicks (2007) documented several breakup ice jam release waves (also called “javes” in the literature) on the Athabasca River from 2001-2003, most notably an event in 2002 that caused a rise in water level of 4.3 m in only 15 minutes at one gauge site. The leading edge and peak celerities of the wave were calculated as 4.3 m/s and 3.2 m/s, respectively (Beltaos and Burrell, 2005a). Beltaos and Burrell (2005a) documented breakup jam release waves on the Restigouche River in 2000 and Saint John River in 2002. On the Restigouche River, one event caused a water level rise of 2.85 m in 6 minutes. On the Saint John River, one event caused a surge that increased water levels by 1.2 m over 70 minutes with leading edge and peak wave celerities as high as 5.5 m/s and 2.5 m/s,

respectively. Other studies involving field observations of breakup jam release waves include Beltaos and Krishnappan (1982), Jasek (2003), Beltaos and Carter (2009), and Beltaos et al. (2011). In general, observations show that the surge wave is most severe near the point of release (toe of jam), causing the greatest and most rapid rise in water level, and amplifying concomitant hydraulic properties such as discharge, velocity, and shear stress. Another common finding is that the celerity of the leading edge of the surge wave is greater than that of the peak.

Very few studies have investigated surge waves produced by ice cover consolidation events during freeze-up. Andres (1995) presented a case study of freeze-up processes on the Peace River, using data from 1970-1988. It was concluded that the occurrence of consolidation events and attendant surges thickened the ice cover beyond that predicted by the steady winter discharge. The author recommended that a nominal increase of 25% to the steady winter discharge should be used in conjunction with a reduced internal friction coefficient of the ice cover to 0.8 from typical values in the range of 1.2-1.3 (Beltaos, 1995a) to account for these effects. Andres et al. (2003) presented water level data from three gauges that captured the passing of a surge wave in an ice covered reach during a freeze-up consolidation event on the Peace River in 2001. The surge caused a water level rise of approximately 1.31 m over a period of three hours at the gauge nearest to the toe of the consolidated reach. They attempted to recreate the event using a numerical model and estimated that the peak discharge at the downstream extent of the consolidated reach was approximately 140% of the steady discharge. Shear forces on the underside of the ice cover were estimated to have increased to approximately 160% of the pre-consolidation value, but did not destabilize the intact cover. A separate freeze-

up consolidation event on the Peace River in 2003-2004 was well documented by Andres et al. (2005). During this event, 73 km of ice cover consolidated to 40 km, releasing a surge of water that increased the water level at a downstream gauge by approximately 2.1 m over one hour. The authors suspected that the consolidation was triggered by a period of warm weather that closely followed a period of extreme cold weather that allowed the ice front to progress rapidly. Andres et al. (2005) took several measurements near the toe region of the consolidated reach, including ice thicknesses, bathymetry, and flow strength beneath the ice cover. They concluded that extensive grounding near the toe region helped to arrest the consolidating ice front, while preferential flow areas allowed the passage of the surge of water downstream. It is suspected that ice covers that form via telescoping (i.e. intermittent cycles of frontal progression and collapse) can over-thicken due to the heightened forces acting on the ice cover during a consolidation event (Andres et al., 2003). More field data is required to understand the characteristics of surges due to consolidation events at freeze-up, and the effects on channel hydraulics that can affect ice dynamics downstream. Thus, the first objective of this study was to obtain an extensive dataset of water levels that could be used to investigate the properties of surge waves originating from consolidation events during freeze-up.

One major obstacle to the study of surge waves caused by ice events is the difficulty in directly measuring water velocity and discharge; ice conditions during freeze-up and breakup pose safety concerns that prohibit manual measurement techniques, and deployed instruments are at risk of being damaged or removed by moving ice. The Rising Limb Analysis Method (RLAM), developed by Beltaos and Burrell (2005b) is an analytical method that uses measured waveforms to determine hydrodynamics of surge waves that

are difficult to measure in the field. The method has been successfully applied to several datasets of measured javes, finding that the discharge can increase by a factor as high as 1.2-10.3, while shear stress can increase by a factor as high as 1.1-6.6 (Beltaos, 2013b). Beltaos (2013b) also verified the RLAM with a numerical modeling exercise, finding that the two methods produced similar results during the rising limb, except for near the peak where the RLAM over predicted discharge by approximately 10%. Application of the RLAM to surges caused by consolidation events at freeze-up is much more limited. Beltaos and Andres (2005) applied the RLAM to the consolidation surge on the Peace River documented in Andres et al. (2003), finding that the results compared well to the numerical modeling results of the original publication. It was found that peak shear stresses just downstream of the consolidation toe were nearly double the unperturbed flow value, yet the ice cover remained intact due to strength gained by the freezing of a solid crust. Andres et al. (2005) also applied the RLAM to the 2003-2004 Peace River event, finding that the peak discharge was nearly three times the unperturbed discharge. Wazney et al. (2018) documented multiple freeze-up seasons on the Lower Dauphin River, noting that the ice cover mechanically thickened most severely during discrete secondary consolidation events that occurred intermittently throughout the freeze-up period. Water levels and ice conditions changed rapidly during these events. The data and observations published in the literature suggest that the thickening of an ice cover at freeze-up depends on more than just the hydraulics associated with the steady state winter discharge, yet very little is known about the effects of consolidation events on surge hydraulics that are linked to ice dynamics. The second objective of this study was to apply the RLAM to a detailed dataset of freeze-up water levels to (a) further verify the applicability of the method to

consolidation surges, and (b) to enhance the understanding of the effects of consolidation events on river hydraulics and, subsequently, ice dynamics during freeze-up.

2.3.3 Methodology

2.3.3.1 Background

Consolidation events are highly dynamic, and result in rapid changes to ice cover characteristics and water levels. A simplified illustration of a consolidation event is shown in Figure 2.31. In the upper panel of Figure 2.31 (before the consolidation), point D is located in a consolidated ice cover, points C and B are located in a thin, juxtaposed section of the progressing ice cover, and point A is located far upstream of the ice cover where surface ice floes are still free-drifting. During the consolidation event, the thin ice cover shoves downstream and thickens against the previously consolidated portion of the ice cover. The toe of the consolidation event is defined as the most downstream location where the ice cover thickens during the event. Shown in the bottom panel of Figure 2.31 (after the consolidation event), as the leading edge of the ice cover recedes downstream, water is released from storage (shown by the red shaded area) and a positive wave moves in the downstream direction. At nearly the same time, the ice cover mechanically thickens within the consolidated reach, which increases the resistance to flow. This causes an upward step in water level to move in the upstream direction as flow is abstracted by the thicker and rougher ice cover (shown by the blue shaded area). The abstraction of flow causes the discharge downstream of the toe of the consolidated reach to decrease, creating a negative wave that moves in the downstream direction. As a result of the consolidation event, the water level increases within the portion of the cover that mechanically thickened (point C). Upstream of the consolidated reach, the water level decreases abruptly due to the removal

of the thin ice cover (point B). The water level far upstream of the consolidation event is unaffected (point A). The water level at locations downstream of the toe (e.g. point D) will respond to the passage of the positive and negative waves caused by the release and abstraction of water upstream. It is in this region that the unsteady flow conditions caused by the consolidation event upstream may temporarily heighten the driving forces attempting to collapse the ice cover.

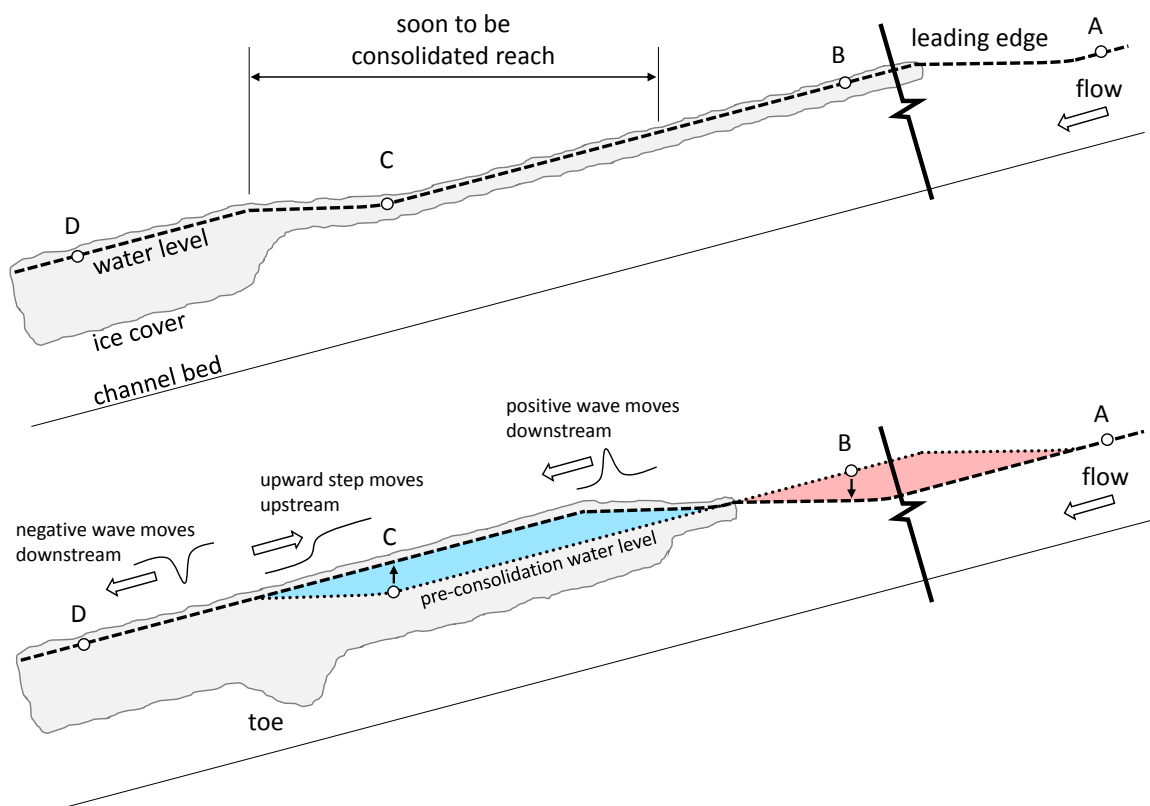


Figure 2.31. Illustration of ice cover characteristics and water levels before (top panel) and after (bottom panel) a consolidation event. Circular markers indicate the pre-event water levels at each location.

2.3.3.2 Study Reach

The Dauphin River is located in central Manitoba and is the last component of the water network that carries water from Lake Manitoba to Lake Winnipeg. An interesting

property of the 52 km long Dauphin River is the abrupt channel slope change that occurs approximately 11.2 km upstream of Lake Winnipeg; the upper reach has a slope of approximately 0.029% and is characterized by numerous channel meanders, low water velocities, and shallow banks. The lower reach is much steeper with a slope of approximately 0.16%. The hydraulics of the Lower Dauphin River differ vastly from the upper reach; the average open water depth is approximately 1.1 m, and the open water velocity and Froude number range from 0.5-2.0 m/s and 0.1-0.6, respectively. Each freeze-up, the ice cover undergoes several consolidation events as it progresses upstream from Lake Winnipeg, resulting in thick accumulations of ice that cause ice-affected water levels to reach 4-5 m or more above open water conditions (Clark and Wall, 2016). In December 2017, flooding of PR 513 due to freeze-up ice accumulations caused the emergency evacuation of the town of Dauphin River First Nation, which is situated at the outlet to Lake Winnipeg.

The outflow of Lake Manitoba has been regulated by the Fairford River Water Control Structure since 1961. Environment Canada operates a discharge and water level gauge on the Dauphin River (gauge 05LM006) that is located about 25 km upstream of Lake Winnipeg, in the more gently sloped Upper Dauphin River. The 25th percentile, mean, and 75th percentile freeze-up flows (over months of November and December) recorded at gauge 05LM006 from 1977 – 2017 are 22 m³/s, 86 m³/s, and 131 m³/s, respectively. The mean discharge during ice cover advancement up the Lower Dauphin River in 2016-2017 (a period of seven days) was approximately 210 m³/s.

2.3.3.3 Instrumentation and Data Collection

A map of the study reach is shown in Figure 2.32. Monitoring sites were established on the Lower Dauphin River at a relatively high spatial resolution to capture the passing of surge waves generated by freeze-up consolidation events. Monitoring sites were originally named sequentially as DRLL01, DRLL02, etc. from Lake St. Martin to Lake Winnipeg (where “DRLL” refers to “Dauphin River Level Logger”). Sites added in subsequent years were appended with a letter to denote their position between original monitoring sites. In 2016-2017, 13 monitoring sites were active in a stretch of 11.2 km of the Lower Dauphin River.

Water levels were recorded using pressure transducers. Solinst Model 3001 M5 Leveloggers (accuracy of ± 0.003 m) were fastened to $75 \times 75 \times 315$ mm iron angles and the assembly was anchored to the bed of the river with rebar. The loggers were programmed and installed during the open water period with a constant sampling interval of 10 minutes. The water surface at each monitoring site was surveyed at the time of logger installation; recorded data was in water surface elevation format and allowed the computation of water surface gradients along the study reach. The water level data was corrected for barometric pressure fluctuations during post processing. At each monitoring site, trail cameras were mounted to trees and programmed to take photographs at set intervals (typically ranging from 15 – 60 minutes) to obtain a qualitative log of ice cover characteristics throughout the freeze-up season.

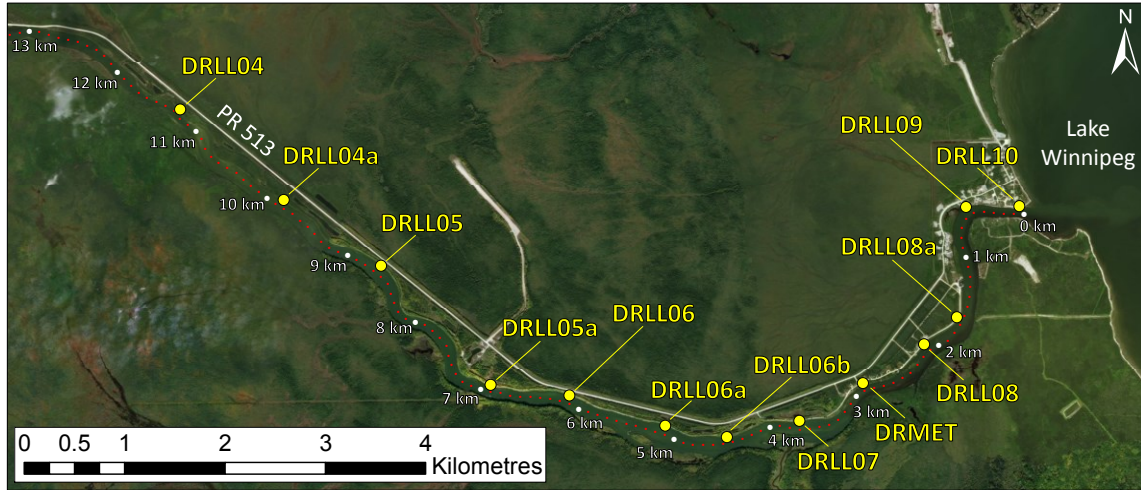


Figure 2.32. Map of study reach and locations of monitoring sites along the Lower Dauphin River.

2.3.3.4 Calculation of Consolidation Surge Hydraulics

The Rising Limb Analysis Method (RLAM), proposed by Beltaos and Burrell (2005b), was used to determine the hydraulic characteristics during the passage of surge waves caused by consolidation events. The brief description of the method here follows that of Beltaos (2013b). The RLAM is based upon the one-dimensional equations of motion of river flow and applies the frozen wave approximation (Henderson, 1966) to develop the following equation for the rising limb of the wave:

$$S_f - S_o \approx \frac{1}{C} \frac{\partial y}{\partial t} \left[1 - a \frac{U^2}{gh} + \frac{C^2}{gh} \left(\frac{U}{C} - 1 + a \frac{q}{Ch} \right) \right] \quad [2.1]$$

where S_f is the friction slope, S_o is the water surface slope prior to the arrival of the surge wave, C is the celerity of the surge wave, y is the vertical coordinate of the water surface, t is time, a is a dimensionless coefficient relating the average water depth to the water surface elevation above a known datum, U is the average streamwise velocity, g is

gravitational acceleration, h is the average water depth, and q is the discharge per unit width.

Cross sectional elevations were only available from bathymetric surveys in the main portion of the channel. The side slopes (h:v) in the Lower Dauphin River can be as steep as 1.6:1 and are typically less than 3.0:1. As such, a rectangular channel approximation ($a = 1$) was made to characterize the geometry at each monitoring site.

The surge wave celerity is not constant along the waveform; the celerity of the leading edge is greater than the celerity of the peak. For a surge wave passing underneath an ice cover that remains intact, the wave attenuates as it moves downstream. Beltaos and Burrell (2005b) proposed the following equation for the variation of celerity along the rising limb of the surge wave:

$$C(y) = C_p + (C_l - C_p) \exp \left[-m \left(\frac{y - y_o}{y_p - y} \right) \right] \quad [2.2]$$

where C_p and C_l are the celerities of the peak and leading edge, respectively, m is a dimensionless coefficient that modifies how the celerity changes along the waveform, y_o is the water surface elevation prior to the arrival of the surge wave, and y_p is the peak water surface elevation during the passage of the surge wave. The average streamwise velocity can be calculated using the continuity equation during the passage of the surge:

$$U = \frac{q_o + \int_{y_o}^y C dy}{h} \quad [2.3]$$

where q_o is the unit discharge of the unperturbed flow. Equation [2.1] can be rearranged to solve for $\frac{\partial t}{\partial y}$ (Beltaos, 2005), and setting $a = 1$ gives the equation:

$$\frac{\partial t}{\partial y} = f(y) = \frac{1}{C(S_f - S_o)} \left[1 - \frac{U^2}{gh} + \frac{C^2}{gh} \left(\frac{2U}{C} - 1 \right) \right] \quad [2.4]$$

A Runge Kutta algorithm can be used to numerically integrate Equation [2.4] to obtain (Beltaos, 2005):

$$t_{n+1} = t_n + \frac{\Delta y_n}{6} (k_1 + 2k_2 + 2k_3 + k_4) \quad [2.5]$$

where Δy_n is a small increment in water surface elevation, $k_1 = f(y_n)$, $k_2 = f\left(y_n + \frac{\Delta y_n}{2}\right) \approx \frac{f(y_n) + f(y_{n+1})}{2}$, $k_3 = k_2$, and $k_4 = f(y_{n+1})$.

The RLAM procedure involves modifying the parameters C_p , C_l , and m such that the computed rising limb waveform matches the observed data. An approximate range for C_p and C_l can be determined if a surge wave is recorded at several gauges along the same reach. However, as shown by Beltaos and Burrell (2005b), even a “blind” calibration of the three parameters results in uncertainties of less than 10% for computed hydraulics during the surge.

Applying the RLAM to consolidation surges recorded on the Lower Dauphin River proceeded as follows. First, a consolidation surge wave was identified in the water level record. Only gauges that showed a distinct rising and falling limb were used; gauges that showed an upward step followed by a constant elevated water level were not used, as these locations were within the consolidated reach where the ice cover mechanically thickened significantly during the event (e.g. point C in Figure 2.31). The rising limb encompassed the first rise in water level to the peak water level. For each gauge, the initial and peak water surface elevations, y_o and y_p , were identified. The water surface slope prior to the passage of the surge, S_o , was computed for each monitoring site using the water surface

elevations and streamwise distance to the adjacent downstream site. The ice thickness at each site was determined using a trial-and-error approach, such that the friction slope (from Manning equation) was equal to the measured water surface slope in the unperturbed flow:

$$S_f = S_o = \left(\frac{Un_c}{R^{\frac{2}{3}}} \right)^2 \quad [2.6]$$

where n_c is the composite Manning roughness of the ice-covered section and R is the total hydraulic radius of the section. The composite Manning roughness was calculated using the Sabaneev equation (Ashton, 1986):

$$n_c = \left(\frac{n_i^{\frac{3}{2}} + n_b^{\frac{3}{2}}}{2} \right)^{\frac{2}{3}} \quad [2.7]$$

where n_i is the Manning roughness of the underside of the ice cover and n_b is the Manning roughness of the channel bed (assumed to be 0.03 based on experience with open water model calibration of the reach). The ice roughness was expressed as a function of ice thickness by fitting a logarithmic relationship to the data published by Nezhikhovskiy (1964) for an accumulation of ice floes at freeze-up, as shown in Figure 2.33. Note the high values of roughness at greater ice thicknesses caused by the irregular shaped ice floes comprising the ice cover. In consolidated covers, the floes are arranged in a disorderly fashion and protrude out into the flow at the underside of the ice cover. Smoothing of the underside of the ice cover over time was not considered due to the short time frame of the consolidation events and surges analyzed.

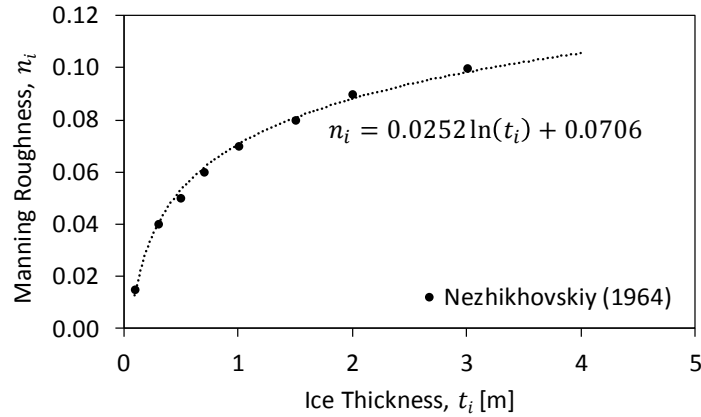


Figure 2.33. Roughness as a function of ice thickness using the data of Nezhikhovskiy (1964) for an accumulation of ice floes at freeze-up.

Approximate initial values of C_p and C_l were deduced from the water level records at successive gauges along the reach. Although the coarse sampling interval of 10 minutes limited the precision of these estimates, they still served as starting points for calibration. The values of C_p , C_l , and m were modified until a satisfactory visual match between the computed and observed rising limbs was obtained. Peak values of streamwise velocity and discharge were computed based on the calculated waveform. Shear stresses on the underside of the ice and channel bed were computed separately due to the differences in roughness between the bed and ice cover. The hydraulic radii of the ice-affected and bed-affected portions of flow beneath the ice cover, R_i and R_b , can be calculated by (Beltaos, 1995c):

$$R_i = R \left(\frac{n_i}{n_c} \right)^{\frac{3}{2}} \quad [2.8]$$

$$R_b = R \left(\frac{n_b}{n_c} \right)^{\frac{3}{2}} \quad [2.9]$$

The shear stress on the ice underside, τ_i , is defined as:

$$\tau_i = \gamma R_i S_f \quad [2.10]$$

A similar equation can be used to calculate the shear stress on the channel bed, τ_b . Of particular interest is the increase in forces during the surge attempting to push the ice cover downstream (i.e. consolidate the ice cover). The consolidating stress (force per unit plan area of the cover), τ_{cons} , is the sum of the shear force acting on the underside of the ice and the streamwise component of the ice and water weight per unit area:

$$\tau_{cons} = \tau_i + \gamma_i t_i S_w \quad [2.11]$$

where γ_i is the specific weight of ice, t_i is the ice cover thickness, and S_w is the water surface slope. The momentum of the mobilized ice contacting the stable ice cover would also add to the consolidation stress, but was not included in the present analysis. During the rising limb of the surge wave, the instantaneous water surface slope can be calculated as (Beltaos, 2013b):

$$S_w = S_o + \frac{1}{C} \frac{\partial y}{\partial t} \quad [2.12]$$

The calculation procedure was performed for each recorded wave, and the computed peak hydraulic parameters were compared at different gauge locations along the reach.

2.3.4 Results and Analysis

2.3.4.1 Water Level Data

The water level records for the 2016-2017 freeze-up period on the Lower Dauphin River are shown in Figure 2.34. As evidenced by the jagged appearance of the water level

data, the ice cover experienced numerous consolidation events as the leading edge progressed upstream. Gauges that recorded water levels that showed an upward step were considered to be within the consolidated reach where mechanical thickening of the ice cover occurred; these water levels were not included in the RLAM. The most downstream water level gauge that recorded a distinct upward step was estimated to be the toe of the consolidation event for subsequent analyses. Several consolidation events created surge waves that were registered by downstream gauges and could be analyzed with the RLAM; these are annotated with a box and identification number in Figure 2.34. For some surges, the water level would simply rise and fall, returning to nearly the same elevation after the surge passed (e.g. Surge 14 as shown in Figure 2.35). However, some consolidation events caused both a positive and negative peak at downstream gauges (e.g. Surge 11 as shown in Figure 2.36). It is hypothesized that this is caused by the superposition of two waves caused by the consolidation event. A positive wave is sent downstream due to the release of water from storage upstream of the consolidated reach (increased flow). Near the toe of the consolidated reach, the thickened ice reduces conveyance and causes a negative wave to travel downstream (decreased flow). This observation was made possible by the high spatial and temporal resolution of collected data in the present study.

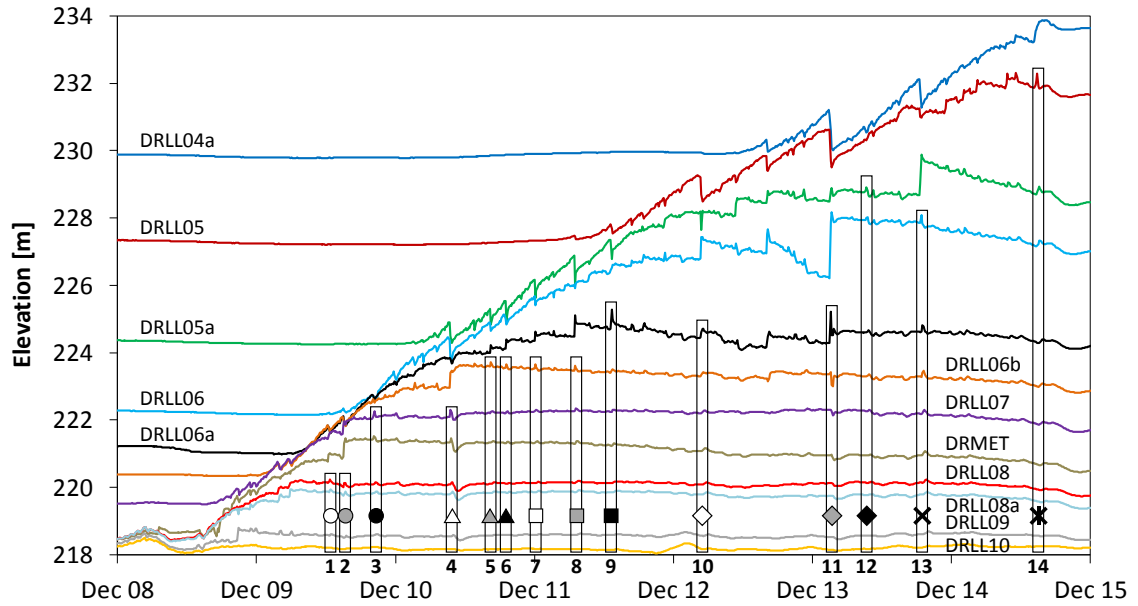


Figure 2.34. Consolidation surges recorded by water level gauges during freeze-up 2016-2017. Surges are numbered sequentially from 1 to 14 for identification. Boxes indicate water levels used in the RLAM for each surge (downstream of consolidated reach).

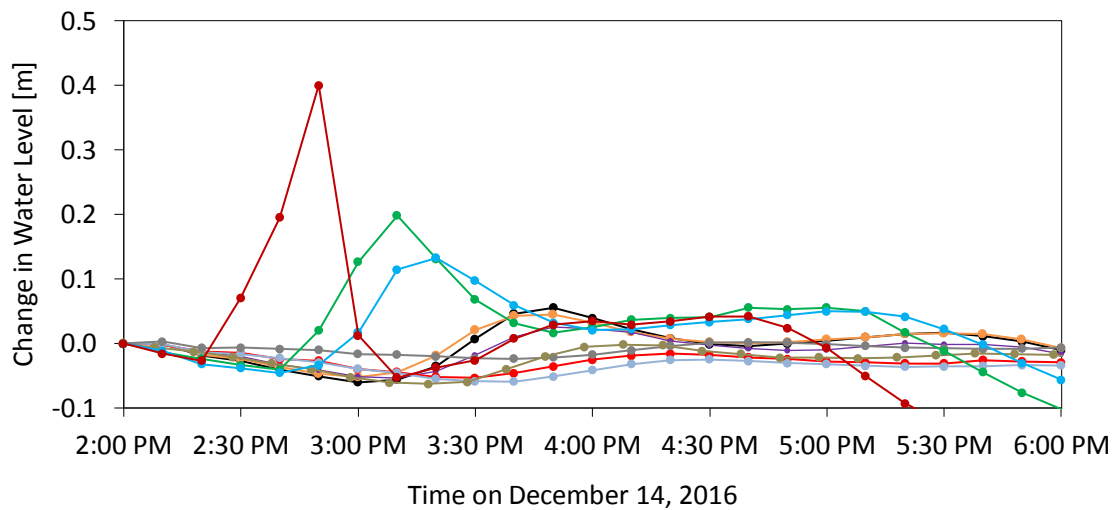


Figure 2.35. Example of positive wave caused by consolidation event (Surge 14). Colours correspond to sites labeled in Figure 2.34.

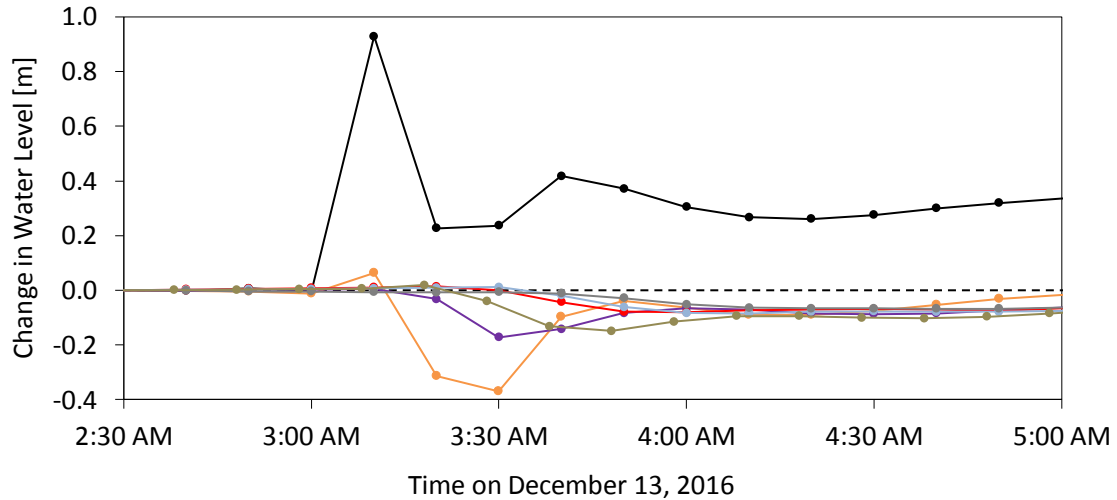


Figure 2.36. Example of negative wave caused by consolidation event (Surge 11). Colours correspond to sites labeled in Figure 2.34.

Water level changes during the surge waves identified in Figure 2.34 are shown in Figure 2.37. It is clear that the peak water level rise is greatest at the gauge immediately downstream of the consolidated reach (i.e. the most upstream data point for each surge). The surge wave attenuates as it moves downstream, resulting in lower peak water level rises at gauges located farther downstream of the consolidated reach. Early in the freeze-up period, a consolidation event upstream of DRLL08 on December 9 (Surge 2 in Figure 2.34) caused water levels to rise by approximately 0.15 m at this location. Separate consolidation events on the evening of December 9 and near noon on December 10 (Surges 3 and 4) caused water levels to rise by over 0.2 m at DRLL07. A consolidation event on December 11 (Surge 9) caused the peak water level to rise over 0.5 m at DRLL06a. The most severe consolidation surge wave recorded occurred in the morning of December 13 (Surge 11). During this event, the water level increased by 2.0 m in less than 20 minutes at site DRLL06, within the consolidated reach. The water level at DRLL06a increased by 0.93 m in less than 10 minutes as the surge wave passed. A consolidation event on

December 14 (Surge 14) near DRLL04a caused water levels to rise by 0.43 m at DRLL05, and this wave was recorded in the water level data all the way downstream to DRLL08a, where it caused the water level to rise by only 3.5 cm. The detection of such small rises in water level attributable to consolidation surges was made possible by the accuracy of the pressure transducers, high spatial resolution of measurement sites, and previous freeze-up experience on this reach of the Dauphin River.

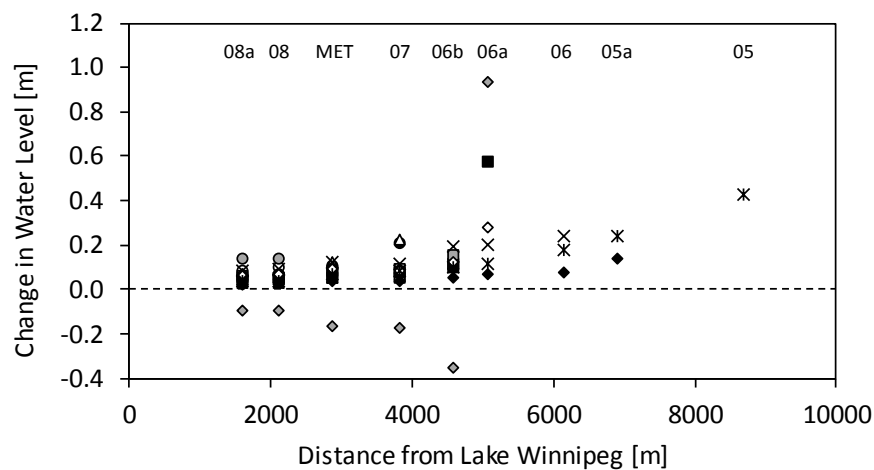


Figure 2.37. Water level rises caused by consolidation surges. Symbols correspond to surges identified in Figure 2.34.

The leading edge and peak celerities, C_l and C_p , were calculated based on the recorded water levels and the measured distance along the centreline of the channel between gauge locations. Due to the high spatial resolution of monitoring sites, the sampling interval of 10 minutes was too coarse to obtain precise values of celerities between each monitoring site (often the peak water level would appear to occur simultaneously at adjacent sites). Therefore, reach-averaged values were calculated for each surge wave by considering the lag time between the water levels at DRLL09 and the farthest upstream gauge that recorded the surge. Figure 2.38 shows that C_l typically ranged

from 2.0-4.0 m/s while C_p ranged from 1.0-2.0 m/s. C_l was found to be greater than C_p for all surges except Surges 1 and 2, which were recorded only at gauges near the downstream end of the reach.

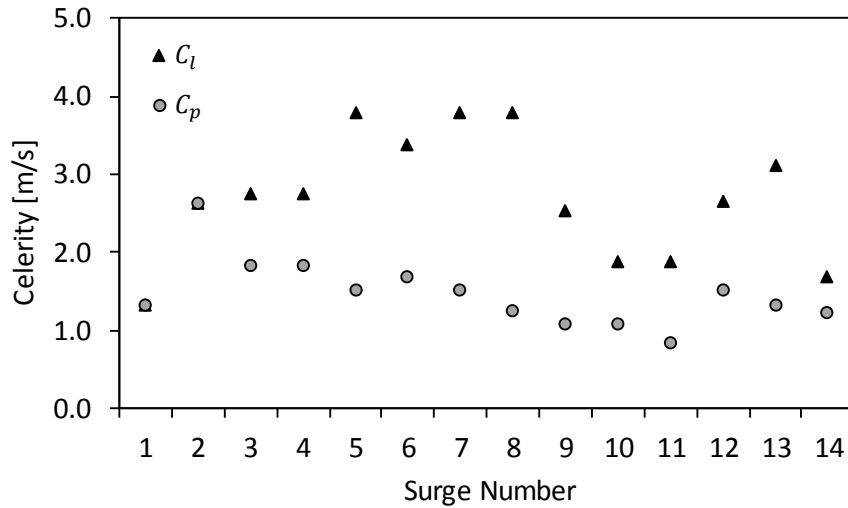


Figure 2.38. Reach-averaged surge wave celerities computed from observed water levels.

2.3.4.2 Application of the Rising Limb Analysis Method

The hydraulic properties during the passage of each surge wave identified in Figure 2.34 were determined using the RLAM. As an example, Figure 2.39 shows the calibrated rising limbs plotted with the observed data for Surge 14. Similar plots for the other surges are included in Appendix A. For each gauge record, the parameters C_l , C_p , and m were modified until a good fit was obtained. Using water levels at sites DRLL09 and DRLL05, reach averaged values for C_l and C_p were determined to be 1.70 m/s and 1.24 m/s, respectively. Best-fit values for C_l ranged from 1.6-2.1 m/s, with the exception of site DRLL06a where C_l was found to be 1.35 m/s. Best-fit values for C_p ranged from 0.8-1.4 m/s. Observed and fitted wave celerities for Surge 14 are shown in Figure 2.40. As noted

by Beltaos and Burrell (2005b), celerity should generally decrease as the wave moves downstream, and C_l should be greater than C_p . While the latter condition was upheld for the best-fit values, the celerity did not consistently decrease with distance downstream, particularly for C_l . This could be due to the changing cross sectional geometry of the river channel and ice cover properties at each gauge site (Kowalczyk Hutchison and Hicks, 2007) and the relatively short reach length over which the surge was recorded. Figure 2.39 and Figure 2.40 are indicative of the goodness-of-fit of the rising limbs and comparison between observed and fitted celerities found using the RLAM for the other surge waves analyzed. All calibrated wave celerities were within the bounds of the kinematic and gravity wave celerities, as defined in Beltaos and Burrell (2005b), with the exception of three instances where the peak celerity was less than 0.05 m/s below the kinematic wave celerity.

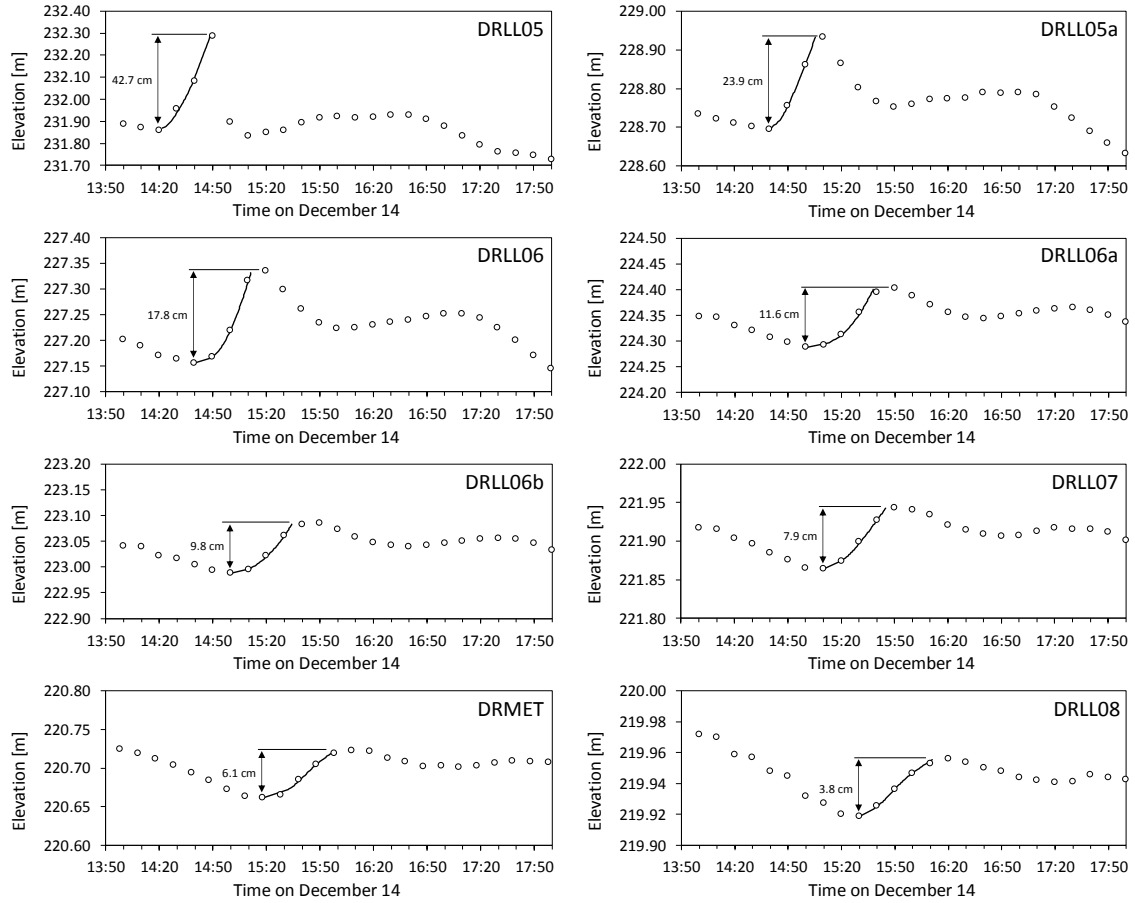


Figure 2.39. Fitted rising limbs of waves caused by Surge 14 (see Figure 2.34).

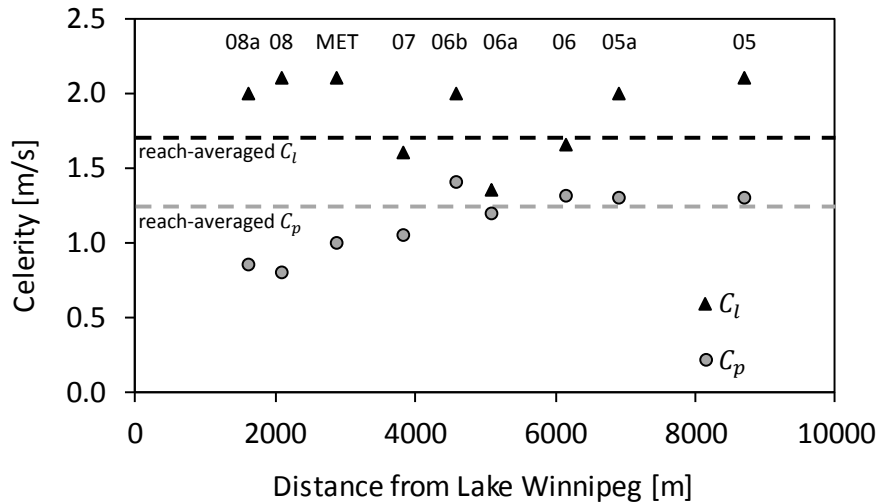


Figure 2.40. Best-fit wave celerities for Surge 14. Dashed lines indicate reach-averaged values calculated from water levels at DRLL09 and DRLL05.

The water surface slope prior to the arrival of each surge was used to calculate the ice thickness at each site; using Equations [2.6] and [2.7], the ice thickness was adjusted such that the friction slope was equal to the water surface slope. The calculated ice thicknesses are shown in Figure 2.41. Typically, the water surface slope at a given site would decrease over time as the ice cover smoothed slightly, thus the calculated ice thickness tended to be greater for surges occurring earlier in the freeze-up period. The calculated values at sites DRLL05, DRLL05a, and DRLL06 are reasonably close to the range of values measured in the field by Wazney et al. (2018).

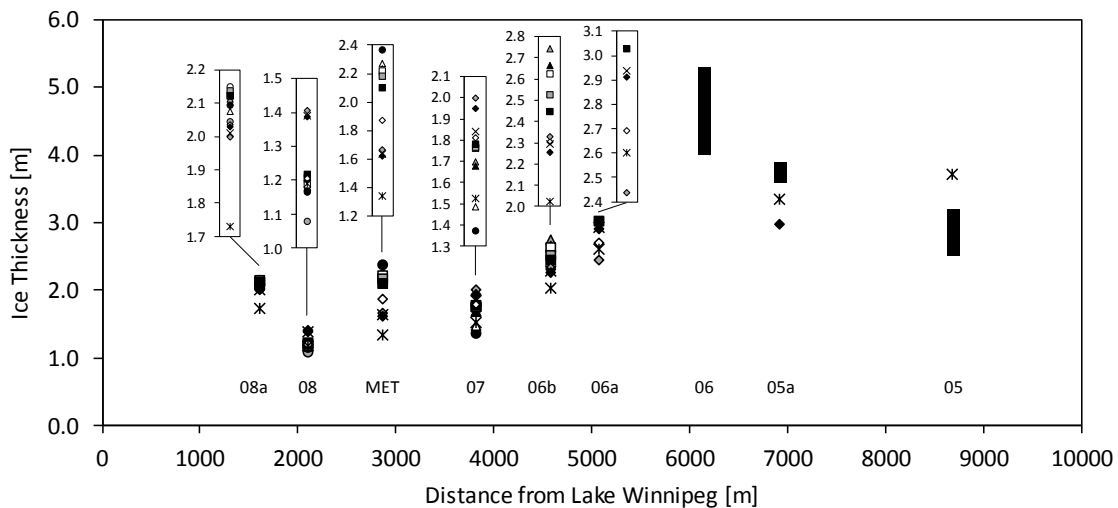


Figure 2.41. Calculated ice thickness prior to passage of surge wave from each consolidation event. Symbols correspond to surges identified in Figure 2.34. Subplots are shown to distinguish closely spaced markers. Shaded ranges indicate the estimated ice thickness at the end of winter as measured by Wazney et al. (2018).

2.3.4.3 Discharge and Velocity

The computed peak discharges at each site during the analyzed consolidation surges in 2016-2017 are shown in Figure 2.42. The steady state unperturbed discharge beneath the ice cover was calculated to be approximately 190 m³/s, reduced from the value of 210 m³/s reported by the Environment Canada gauge upstream due to flow abstraction caused

by the progressing ice cover. As shown, the peak discharge is always greatest at the gauge located just downstream of the consolidated reach, and decreases with distance downstream. The highest peak discharge computed was $577 \text{ m}^3/\text{s}$, occurring at DRLL06a from Surge 11 on December 13. Site DRLL06a experienced a separate peak discharge of $414 \text{ m}^3/\text{s}$ during the passage of Surge 9. The high peak discharges computed at DRLL06a (relative to other monitoring sites) were the result of several significant consolidation events that caused mechanical thickening of the ice cover at site DRLL06. At site DRLL07, peak discharges of $271 \text{ m}^3/\text{s}$ and $284 \text{ m}^3/\text{s}$ were caused by Surge 3 and Surge 4, respectively. Peak discharge reached $241 \text{ m}^3/\text{s}$ at site DRLL08 during Surge 2. Surge 13 caused a peak discharge of $264 \text{ m}^3/\text{s}$ at site DRLL06. The latest freeze-up consolidation event caused a peak discharge of $283 \text{ m}^3/\text{s}$ at site DRLL05 (Surge 14). During Surge 11, the gauges downstream of DRLL06a recorded a negative wave; discharge at DRLL06b was as low as $74 \text{ m}^3/\text{s}$ during this event.

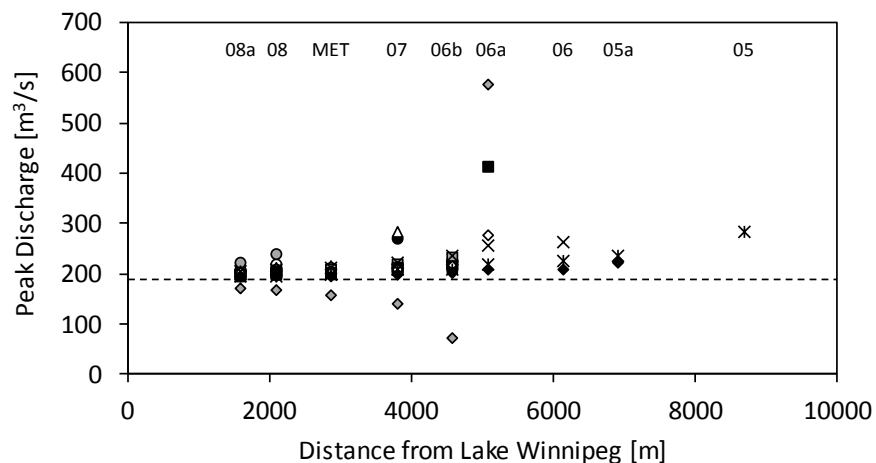


Figure 2.42. Peak discharge caused by consolidation surges. The dashed line indicates the steady discharge of $190 \text{ m}^3/\text{s}$, accounting for flow abstraction. Symbols correspond to surges identified in Figure 2.34.

The relationship between peak discharge and change in water level for all surges is shown in Figure 2.43. The data shows an approximately linear trend; a best fit line suggests that each one metre rise in water level corresponds to an increase in discharge of approximately 362 m³/s. The change in cross sectional geometry and variable ice conditions at each site over time result in some spread in the data, but the relationship provides a good rule-of-thumb estimate. As shown, a water level change of 0 m corresponds to a best fit discharge of 184 m³/s which is within 4% of the steady unperturbed discharge of 190 m³/s, accounting for flow abstraction.

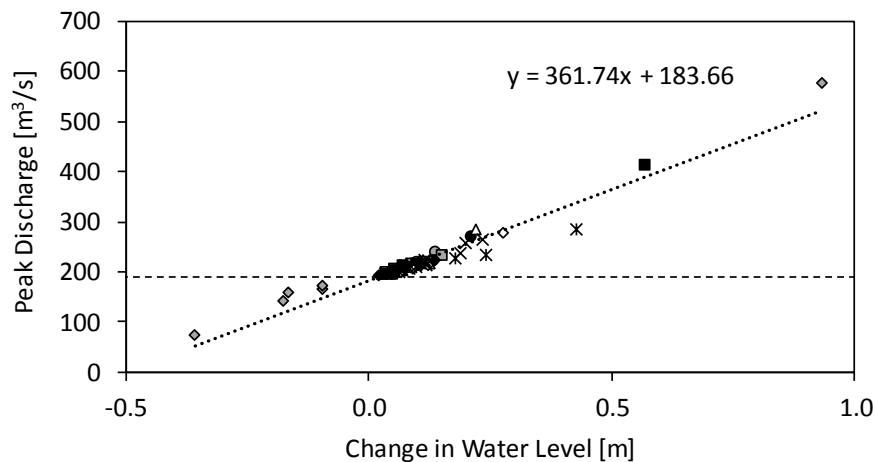


Figure 2.43. Relationship between peak discharge and water level change during surge waves. Symbols correspond to surges identified in Figure 2.34.

The water velocity at each monitoring site during the passage of surge waves was calculated using Equation [2.3]. Figure 2.44 shows the change in water velocity from the pre-surge value for all surge waves analyzed. Once again, the change in velocity is greatest at the sites located just downstream of the toe of the consolidated reach. Site DRLL06a had the largest changes in water velocity of all monitoring sites; Surge 11 and Surge 9 caused velocity increases of 0.53 m/s and 0.36 m/s, respectively. Surge 13 caused a water

velocity increase of 0.16 m/s at DRLL06. Sites DRLL08, DRLL07, and DRLL05 also experienced water velocity increases greater than 0.1 m/s due to surges during the freeze-up period.

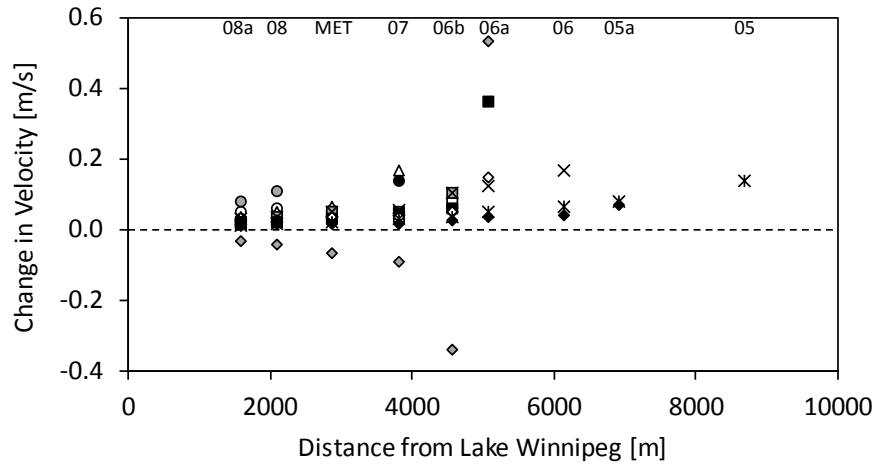


Figure 2.44. Change in water velocity caused by consolidation surges. Symbols correspond to surges identified in Figure 2.34.

2.3.4.4 Shear and Consolidation Stresses

The computed stresses on the ice cover at each monitoring site for a selection of consolidation events that occurred late in the freeze-up period (Surges 11-14) are shown in Figure 2.45. The difference between circular markers indicates the change in shear stress applied to the ice cover during the passage of the surge, while the difference between square markers indicates the change to the total consolidation stress (shear and weight). For each event, the difference between pre-event and peak stress is greatest at the site farthest upstream, where the peak discharge is greatest. For Surge 11, the peak stresses are less than the pre-event stresses at all gauges except the most upstream gauge (see Figure 2.36). This is a result of the negative wave (decrease in water level and discharge) at these sites. At DRLL06a, the shear stress on the underside of the ice increased from 25 Pa to 79 Pa,

while the total consolidation stress increased from 66 Pa to 165 Pa. Thus, the total consolidation stress increase of 99 Pa was due to a 54 Pa increase in shear stress and a 45 Pa increase due to the greater downstream weight component caused by the steeper water surface slope. Surge 12 had a relatively small effect on water levels (see Figure 2.34); the change in stresses on the ice cover were also small. The high pre-event stresses for this surge at DRLL06 were a result of the mechanical thickening of the ice cover that occurred to produce Surge 11. At DRLL05a the ice shear stress increased by only 5 Pa while the total consolidation stress increased by only 10 Pa. Surge 13 was caused by a severe consolidation event that mechanically thickened the ice cover at DRLL05a. Shear stresses on the underside of the ice cover at DRLL06 increased from 48 Pa to 71 Pa (change of 23 Pa), while the total consolidation stress increased from 181 Pa to 247 Pa (change of 66 Pa). Note that the pre-event stresses of Surge 12 and Surge 13 are comparable, as the water surface gradients and ice thicknesses did not change drastically between these two events. Surge 14 was caused by a consolidation event that thickened the ice cover at DRLL04a. At DRLL05, the shear stress on the underside of the ice increased from 30 Pa to 43 Pa (change of 13 Pa), while the total consolidation stress increased from 90 Pa to 114 Pa (change of 24 Pa).

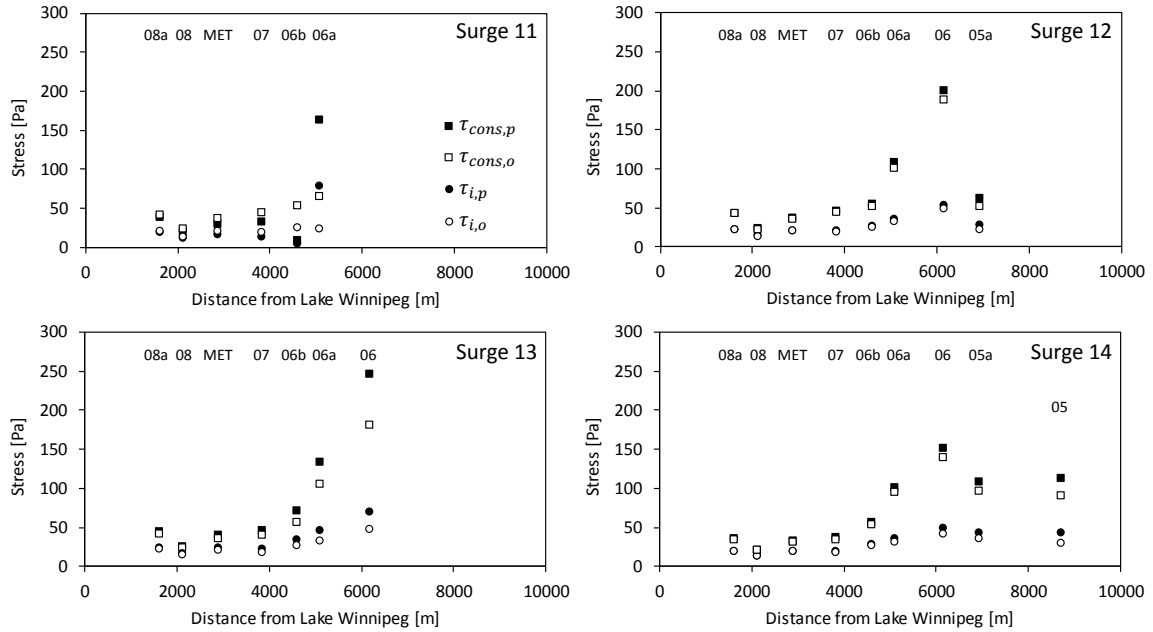


Figure 2.45. Stresses on ice cover during late-season consolidation events. Square markers indicate total consolidation stress, τ_{cons} , and circular markers indicate shear stress on the underside of the ice cover, τ_i . Hollow markers indicate pre-surge values and solid markers indicate peak values during surge.

For all freeze-up consolidation surges analyzed, the surge that produced the greatest peak discharge was identified for each site, and the corresponding pre-event and peak hydraulic properties are summarized in Table 2.3. Consolidation surges caused the discharge at all gauges to increase beyond the steady state discharge of $190 \text{ m}^3/\text{s}$ at one point or another during freeze-up. The ice thickness on the Lower Dauphin River is linked to the severity of consolidation events at a given site; sites that had the largest values of computed discharge in Table 2.3 (e.g. DRLL06a) were just downstream of sites that had high computed ice thicknesses (e.g. DRLL06). Consolidation surges caused total consolidation stresses to increase by 23.3 Pa, 66.2 Pa, and 98.7 Pa at sites DRLL05, DRLL06, and DRLL06a, respectively. Consolidation stresses increased by 6.8-16.0 Pa at the other sites. On average, shear stresses on the underside of the ice cover during the surge

made up 55% of the increase in total consolidation stresses for the surges listed in Table 2.3 (the remaining 45% due to ice weight). The most significant increase in bed shear stress occurred at DRLL06a during Surge 11; bed shear stress increased by 10.0 Pa. The other sites registered bed shear stress increases between 0.9-3.2 Pa. The higher shear stresses on the underside of the ice cover compared to the bed are due to the high roughness values of the ice cover composed of irregularly shaped fragments of ice that protrude into the flow (Wazney et al., 2018, 2017). The increase in bed shear during consolidation surges could increase the potential for bed scour, while the increase in ice shear could cause erosion and smoothening of the ice underside.

Table 2.3. Computed hydraulics of surges producing greatest discharge at each monitoring site in 2016-2017.

Site	Surge	Q_{max} [m ³ /s]	$t_{i,max}$ [m]	Ice Shear Stress [Pa]			Total Consolidation Stress [Pa]			Bed Shear Stress [Pa]		
				$\tau_{i,o}$	$\tau_{i,p}$	$\Delta\tau_i$	$\tau_{cons,o}$	$\tau_{cons,p}$	$\Delta\tau_{cons}$	$\tau_{b,o}$	$\tau_{b,p}$	$\Delta\tau_b$
DRLL05	Surge 14	283.5	3.7	30.4	43.3	12.9	90.2	113.5	23.3	4.7	6.7	2.0
DRLL05a	Surge 14	235.1	3.3	36.6	43.9	7.3	96.5	108.8	12.3	5.9	7.1	1.2
DRLL06	Surge 13	264.1	5.0	48.0	70.5	22.5	181.0	247.2	66.2	6.8	10.0	3.2
DRLL06a	Surge 11	576.6	3.0	24.8	79.4	54.6	66.2	164.9	98.7	4.5	14.5	10.0
DRLL06b	Surge 13	237.2	2.7	27.1	35.1	8.0	56.7	71.0	14.3	5.1	6.6	1.5
DRLL07	Surge 4	284.2	2.0	12.5	21.2	8.7	23.1	39.1	16.0	2.8	5.0	2.2
DRMET	Surge 4	217.3	2.4	31.1	36.1	5.0	64.8	73.4	8.6	5.9	6.8	0.9
DRLL08	Surge 2	240.7	1.4	10.7	15.3	4.6	15.8	22.6	6.8	2.9	4.1	1.2
DRLL08a	Surge 2	223.9	2.0	23.7	29.1	5.4	45.6	54.9	9.3	4.7	5.7	1.0

2.3.5 Discussion

Previous applications of the RLAM to surge waves caused by freeze-up consolidation events on the Peace River by Beltaos and Andres (2005) and Andres et al. (2005) were successful, however the data were limited. Beltaos and Andres (2005) investigated one surge in 2001 that was captured by three water level loggers over a reach length of approximately 60 km. Water levels were recorded at a one hour interval. The consolidation surge was quite significant, causing a peak water level rise of 1.3 m at the most upstream site and heightened water levels that lasted approximately seven hours due to the surge. Andres et al. (2005) applied the RLAM to a surge wave created by a consolidation event in 2003-2004. The surge wave was captured over a 23 km reach by three gauges, which recorded water levels at a 15 minute interval. The peak water level rise was approximately 2.1 m, corresponding to a peak discharge of nearly three times the unperturbed flow.

The Lower Dauphin River is much smaller in scale than the Peace River; thus this study provided a unique application of the RLAM to freeze-up processes that has not been published in the literature to date. Novel to this application was the high quantity and spatial resolution of water levels that captured relatively small scale consolidation surges in a short reach length; the 11.2 km reach of the Lower Dauphin River was instrumented with 13 water level gauges. In general, the magnitude and duration of the surge waves were smaller than the aforementioned events on the Peace River. For example, Surge 14 shown in Figure 2.39 caused a water level rise of 0.43 m and a heightened water level duration of only 40 minutes at the most upstream site. One minor limitation to this study was the sampling interval of the water level loggers. The sampling interval of 10 minutes

is relatively fine for freeze-up monitoring studies of this type, and allowed the identification of several consolidation events and response of water levels. However, the sampling interval was not fine enough to obtain accurate measurements of leading edge and peak wave celerities between each monitoring site. Still, the water level records at the most downstream and upstream gauges that recorded the surge were useful in providing reach-averaged values that served as a starting point for calibration. Other assumptions included an assumed constant top width at each monitoring site, and the estimation of ice thickness based on the water surface slope prior to the arrival of a surge wave. Although these assumptions introduce some uncertainty in the results, the comparison between estimated and measured ice thickness at multiple sites was reasonably good (see Figure 2.41), increasing the confidence in the results of the RLAM.

In general, the RLAM was successful in deducing the transient hydraulic properties of consolidation surges recorded on the Lower Dauphin River. The discharge and water velocity were determined to be highest at the site located nearest to the toe of the consolidated reach. Of particular interest during the period of ice cover development at freeze-up is the change in forces that promote mechanical thickening. The increase in consolidation force caused by the surge wave was also found to be greatest near the toe of the consolidated reach, and decay in the downstream direction as shown in Figure 2.46. The data suggests that a consolidation surge can have a large impact on the stresses on the ice cover within approximately 2 km of the toe of the consolidated reach on the Lower Dauphin River. The separation of the total consolidation stress into its two components is also shown in Figure 2.46. Near the toe of the consolidated reach, the contributions of shear and weight to the total consolidation stress increase are roughly equal (approximately

50% each). With increased distance from the consolidated reach, the shear contribution increases (to approximately 70%), while the contribution of the increased weight decreases (to approximately 30%). In other words, at locations farther downstream from the consolidation toe, the increase in consolidation forces during the surge wave is mainly due to the faster moving water increasing shear stress on the rough underside of the cover. The change in the downstream component of the ice and water weight plays a less important role.

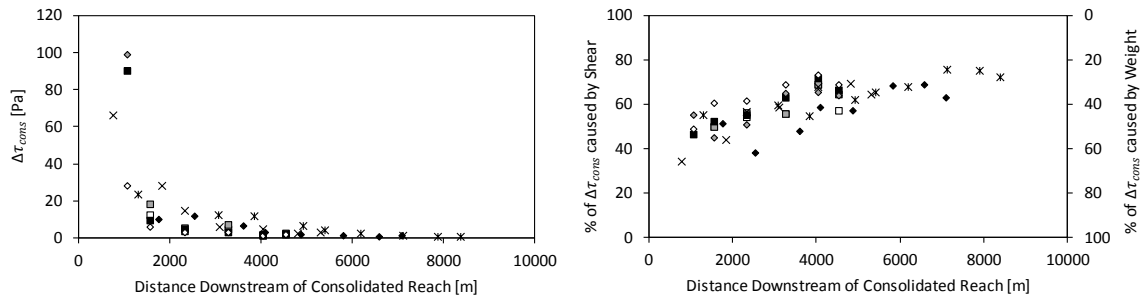


Figure 2.46. Change in consolidation stress downstream of the consolidated reach (left), and percent of the change in consolidation stress resulting from increased shear on the underside of the ice cover (right). Symbols correspond to surges identified in Figure 2.34. Data only shown for Surges 7-14 for clarity.

It should be noted that due to the water level measurement interval of 10 minutes, the peak water level during the passage of steeply rising and falling limbs was likely not captured, particularly near the toe of the consolidated reach. Thus, the values of various hydraulic parameters computed may actually slightly under predict the true values. Future data collection efforts should increase the sampling frequency to avoid this uncertainty.

Results from the application of the RLAM to consolidation surges on the Lower Dauphin River could further the understanding of how the strength of an ice cover changes with time during freeze-up. When an ice cover remains stationary for some duration of

time, interstitial water between the ice floes freezes and adds non-frictional resistance to further consolidation events. Taking a closer look at Figure 2.34, it is clear that the ice cover at site DRL06b mechanically thickened on December 10, resulting in Surge 4. However, all consolidation events occurring after this (upstream) did not thicken the cover. Thus, the configuration of the rubble ice, plus any freezing effects, resulted in a cover that was strong enough to withstand the increased consolidation stresses caused by shoving events during the remainder of the freeze-up period. Andres et al. (2005) postulated that the combination of grounded rubble ice and the presence of a thick thermal ice crust could stop the consolidating front from progressing downstream. Additional field datasets such as the one collected in this study could help investigate the effects of cold ambient conditions on the ability of an ice cover to resist mechanical thickening during periods of unsteady flow.

2.3.6 Conclusions

The Lower Dauphin River was instrumented with 13 water level loggers during the 2016-2017 freeze-up period, which recorded multiple consolidation events as the ice cover progressed upstream. Each event sent a surge of water downstream which temporarily increased water levels and other concomitant hydraulic parameters such as discharge, velocity, and shear stress. The Rising Limb Analysis Method (Beltaos and Burrell, 2005b) was used to determine these parameters and investigate the effects of the unsteady flow on ice dynamics.

Fourteen surge waves created by consolidation events were identified in the collected water level data and were analyzed with the RLAM. All monitoring sites in the Lower Dauphin River showed some perturbations in water level over the course of freeze-

up, indicating that the ice cover was, at one point or another, subjected to hydraulic conditions more severe than those associated with the steady winter discharge. The most severe conditions occurred at site DRLL06a. On two occasions, the computed discharge during a surge wave exceeded $400 \text{ m}^3/\text{s}$, more than twice the recorded steady discharge. These surge waves were the result of consolidation events causing the ice cover near DRLL06 to mechanically thicken, achieving a final ice thickness of approximately 4.0-5.3 m. Increases to consolidation stresses were found to be caused primarily by the increased shear forces of the faster moving water beneath the ice cover, but also in part by the increased weight of the ice cover as the water surface slope steepened as the wave passed. On multiple occasions, the consolidation stress on the ice cover just downstream of the consolidated reach increased by more than 20 Pa; one surge at DRLL06a caused an increase of 99 Pa. The amplification of stresses on the ice cover during these events highlights the interdependence of ice dynamics and channel hydraulics, and elucidates how a consolidation event upstream can trigger further consolidations at downstream locations.

Concurrent to the incidence of consolidation events that can cause heightened stresses on an ice cover during freeze-up, the ice cover gains strength as interstitial water at the surface freezes. Ice floes bond together and the ice cover obtains non-frictional resistance to further mechanical thickening. Further research is required to investigate how the strength of an ice cover varies over the freeze-up season, and what causes a consolidation to stop once it has started (i.e. determining the toe location of the consolidated cover). This will allow numerical models to simulate whether an ice cover will collapse or withstand the changing stresses applied to it during highly dynamic and unsteady conditions as examined in this paper.

2.3.7 Acknowledgements

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2.4 Conclusions and Contributions of this Work

The major conclusions of the field monitoring program and general contributions to the field of river ice engineering are summarized below.

- (i) Ice cover formation and advancement up the Lower Dauphin River was highly dynamic in all three monitoring seasons, involving periods of frontal progression with intermittent collapses of the ice cover. The sudden consolidation of the ice cover caused rapid changes to water levels and ice characteristics, which were observed and documented at a high spatial and temporal resolution compared to previous studies.
- (ii) The final ice profile and ice-affected water levels were different in the three monitoring seasons, presumably due to different combinations of meteorological and hydraulic conditions. In the three seasons of data collected, the highest ice elevations and peak water levels corresponded to the season with a high flow and mild average air temperature during freeze-up. A season with similar flow and colder air temperatures resulted in lower ice elevations and peak water levels. This finding supports the idea that thermal strengthening of a forming ice cover plays a role in the ability of an ice cover to resist deformation and mechanical thickening.
- (iii) Analysis of water level records with the Rising Limb Analysis Method showed that dynamic ice processes and river hydraulics are interdependent. Unsteady

flow conditions caused by dynamic ice events (e.g. shoves) can temporarily change velocity and shear stresses both upstream and downstream of the consolidated reach. This could have implications on subsequent dynamic ice events, or erosion, transport, and deposition of bed sediments. For example, an increase in discharge from a consolidation event could lead to the collapse of the ice cover downstream, or the scouring of bed material as the surge wave passes.

- (iv) The variety of data collected (water surface elevations, time-lapse photographs, top-of-ice elevations, ice thickness measurements, and meteorological data), and the spatial and temporal resolution of the data, comprises one of the most complete and comprehensive freeze-up monitoring programs ever conducted. Detailed field datasets like these are required to evaluate the ability of current numerical models and push their development forward.

Chapter 3 Laboratory Investigation of the Consolidation Resistance of a Floating Ice Cover

3.1 Introduction

Field observations presented in Chapter 2 highlighted the link between meteorological conditions and dynamic ice processes during river ice cover formation; ice-affected water levels varied from season to season, even with relatively similar flow conditions. One of the most significant deficiencies in many state-of-the-art numerical river ice models is the lack of consideration of thermal processes on ice dynamics. Part of the reason for this is the lack of experimental work on the subject. To address this knowledge gap, laboratory tests were undertaken to investigate the effect of the growth of a surface ice crust on the ability of a rubble ice cover to resist mechanical thickening. The paper included in Section 3.2 describes the experimental design, methodology, and results of the tests. The primary goal of the physical experiments was to investigate the change in peak resistance of an ice cover after varying freezing indices, which quantifies time exposed to freezing ambient air temperatures. The paper was accepted to Cold Regions Science and Technology in October 2018 (doi:10.1016/j.coldregions.2018.10.001).

3.2 Paper 3: Laboratory Investigation of the Consolidation Resistance of a Rubble River Ice Cover with a Thermally Grown Solid Crust

3.2.1 Abstract

River ice behaviour during freeze-up depends on a complex relationship between dynamic and thermal processes. This study investigates the effects of cold ambient air temperatures on the ability of a rubble ice cover to resist consolidation (mechanical thickening) under externally applied forces. An experimental apparatus was constructed in a refrigerated room at the University of Manitoba. The apparatus consists of a translating plate that was used to deform a floating rubble ice cover in a waterproof tank. Prior to resistance testing, the ice cover was exposed to an ambient air temperature of -10.5°C for a specified duration. During this time, the interstitial water between the rubble pieces at the surface froze, creating a solid crust that solidified a portion of the rubble surface. Instruments were used to measure the displacement of the plate and resisting force generated by the ice cover during the resistance testing phase. Results indicate that even short durations of freezing (less than two hours) have a considerable impact on the peak resistance of an ice cover. The force required to consolidate a 155 mm thick rubble ice cover exposed to an ambient air temperature of -10.5°C for two hours was over 30 times that of an equivalent unfrozen accumulation of rubble ice. The peak resistance of a cover comprising rubble ice with a solid crust was found to be much greater than the sum of the individual contributions of the rubble and solid ice layers considered separately. Results from this study highlight the significant effect of thermal processes on ice dynamics, and can inform future research efforts in incorporating these processes into numerical river ice models.

3.2.2 Introduction

River ice practitioners have been using numerical models to predict the effects of river ice on channel hydraulics for many years; however, more attention has been given to simulating ice dynamics at breakup (ambient air temperature above 0°C) than at freeze-up (ambient air temperature below 0°C). In a breakup scenario, the ice rubble is traditionally assumed to behave as a floating, granular material that follows a Mohr-Coulomb failure criterion, with strength obtained from frictional interlocking of ice floes within the accumulation (Pariset et al., 1966; Uzunur and Kennedy, 1976). A dynamic ice cover formed in a natural river during the breakup period is subjected to forces attempting to deform the accumulation and push it downstream: the downstream component of the weight of the ice, shear forces caused by water flowing underneath the cover, and shear forces caused by wind on the surface of the cover (Beltaos, 1995a).

During freeze-up, swift flowing rivers develop an ice cover through a process known as telescoping (Beltaos, 1995d). First, an ice cover initiates when frazil ice pans are stopped at a competent thermal ice cover or natural bridging location. Additional incoming pans juxtapose if hydraulic conditions permit, and the stationary front progresses upstream at a thickness of approximately a few pans (due to some ridging or minor packing). As the ice cover lengthens, a portion of the external applied forces are transferred to the river banks, and the remainder is transferred to the cover downstream. At a certain point, the net external forces applied to a section of the ice cover exceed the internal strength, and the cover collapses. This results in mechanical thickening of the cover as pans that were once juxtaposed on the surface consolidate to create an accumulation that is multiple pans thick arranged in a disorderly fashion. During the consolidation phase, the

stationary front of the ice cover retreats downstream. Once the internal resistance of the thickened cover can withstand the external forces, the ice cover returns to a stationary state. The stationary front again progresses upstream as incoming pans continue to juxtapose, and the process repeats itself. An illustration of the formation of a river ice cover at freeze-up is shown in Figure 3.1.

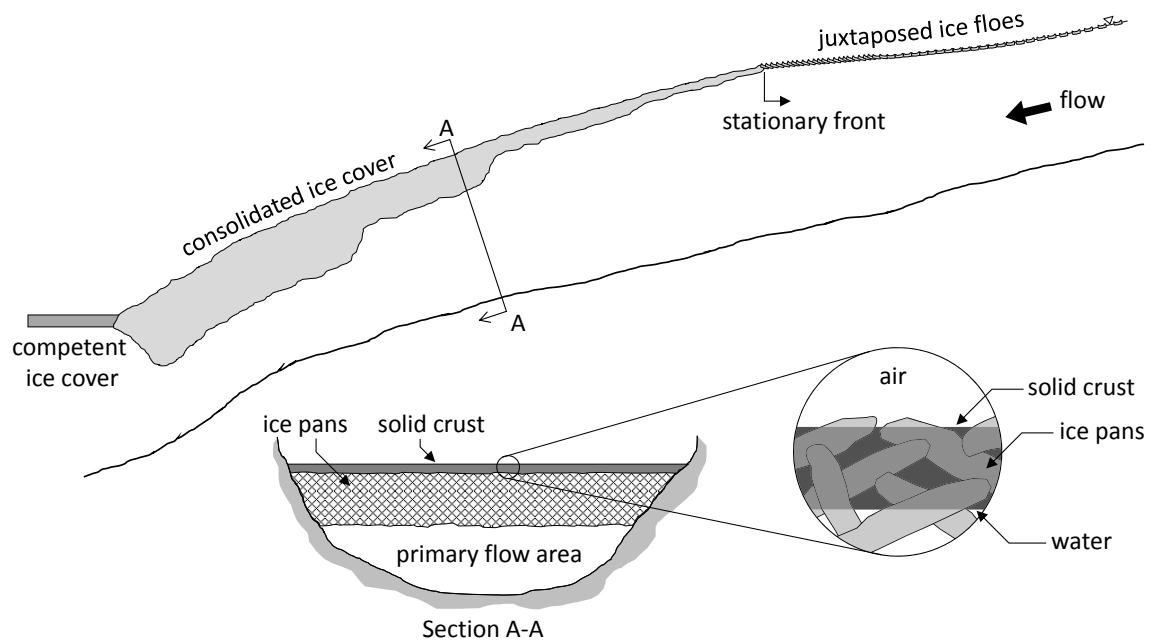


Figure 3.1. Illustration of advancing ice cover with stationary portion growing a solid crust under freezing conditions.

Michel (1991) noted that ice dynamics at freeze-up can differ considerably from breakup. Interstitial water between pans in a stable cover can freeze, creating a solid ice crust that can quickly add significant strength to the cover, increasing its resistance to further consolidation (Michel, 1978). The solid crust is thicker near the toe of the cover and thinnest near the stationary front (Beltaos, 2013a). In accumulations that are multiple pans thick, ice grows at contact points to form freeze-bonds that can also add strength. A number of researchers have identified the effects of freeze-bonding at ice contact surfaces

on the behaviour of ice rubble through laboratory experiments (e.g. Ettema and Urroz, 1989; Schaefer and Ettema, 1985; Tatinclaux et al., 1977; Urroz and Ettema, 1987). However, an experimental investigation of the effects of a thermally grown surface crust on the consolidation resistance of a floating rubble ice cover has not yet been conducted.

Field observations have also corroborated the notion that thermal processes at freeze-up are linked to ice dynamics and can control whether an ice cover is more prone to frontal progression or consolidation events (Andres, 1999; Michel, 1984). Michel (1991) suggested that shoving is less likely during freeze-up due to freezing effects, but can still occur if the ice front progresses quickly. Field observations by Wazney et al. (2017) on the Dauphin River indicate that shove events can cause rapid thickening and consolidation of a previously stable cover, leading to sudden increases in water level within the consolidated reach. The sudden release of stored water during a shove event temporarily increases the discharge, leading to over-thickening of the ice cover beyond that associated with the steady state winter discharge (Andres et al., 2003). As such, the timing and severity of shove events during freeze-up influence the peak ice-affected water levels and final ice thickness profile. The triggering of shove events is related to the rate at which the ice cover gains strength due to freezing effects versus the rate at which external forces increase as incoming ice floes are added to the leading edge (Andres, 1999). Andres (1999) reviewed instances of juxtaposed and consolidated ice covers along the Peace River and found that the effective compressive strength of the solid ice crust was approximately 1.2 MPa. The limited number of field observations and the absence of physical experiments investigating the resistance to consolidation of an ice cover under continual freezing limits the current state of knowledge on this topic.

The objective of this study was to investigate the consolidation resistance of a floating rubble ice cover exposed to freezing ambient conditions through a series of physical experiments. The scope of the experiments was limited to modifying the thickness of the rubble layer and the length of time it was exposed to a freezing ambient air temperature. Variables such as cover length, ice rubble size, and ice composition (frazil slush or competent blocks) were not investigated. Angular ice fragments were used to simulate the accumulation of competent ice floes that form a porous rubble ice cover. An apparatus was designed to apply a uniaxial compressive force to mimic the horizontal thrust experienced by an ice cover resulting from the self-weight and water shear forces that develop as the stationary front progresses upstream. The experiments shed light on how early freezing of a solid crust and freeze-bonds at the surface of a rubble ice cover influence its ultimate strength and ability to resist shoves and further mechanical thickening. Results of this work can be used to inform future physical experiments and field investigations on the link between thermal and dynamic river ice processes. Research on this topic can be incorporated into comprehensive numerical river ice models, allowing them to more accurately simulate processes affecting ice dynamics in a freeze-up scenario.

3.2.3 Methodology

3.2.3.1 Test Apparatus

Experiments were conducted in a refrigerated room at the Hydraulics Research and Testing Facility at the University of Manitoba. The room measures 4 m by 4 m and has two cooling systems that can maintain air temperatures as low as -30°C with a precision of $\pm 0.1^\circ\text{C}$. A tank was constructed from plywood sheets and made waterproof with fiberglass mats, polyester resin, and a coat of flexible rubber sealant. The inner dimensions of the

tank measured approximately 1540 mm long, 560 mm wide, and 750 mm deep. The tank was encased in a rigid frame constructed of extruded aluminum profiles. A vertical compression plate was supported on two linear rails with sleeve bearings to allow the plate to move along the length of the tank with minimal resistance. The plate assembly was threaded to an ACME lead screw which was turned by a servo motor and gearbox. The rotation of the lead screw resulted in linear translation of the plate within the tank. The position of the compression plate was measured with a linear motion transducer (LMT). A thrust block and load cell were positioned at the end of the tank, opposite the motor and gearbox, to measure the forces exerted by the ice cover on the compression plate. Two steel plates and tie rods provided additional support to the frame to withstand the forces required to fail the ice cover. Rigid foam insulation was placed on all four walls of the apparatus to ensure the dominant mode of heat transfer was through the air-ice or air-water interface at the surface. A simplified schematic and photograph of the test apparatus are shown in Figure 3.2.

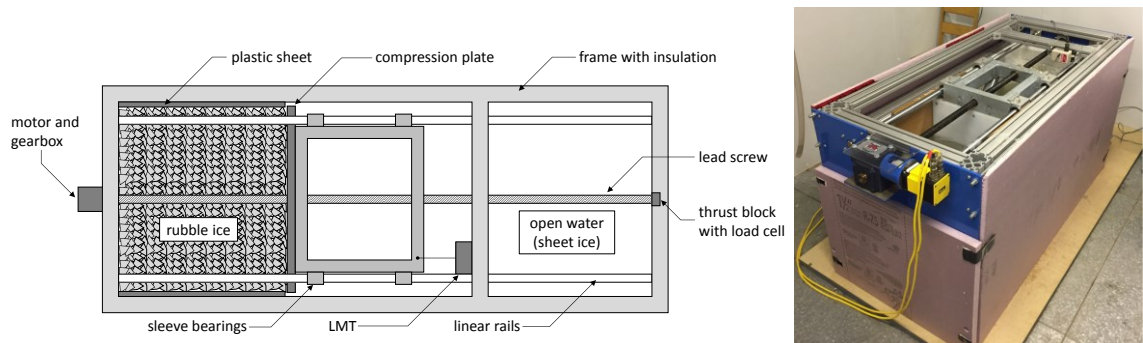


Figure 3.2. Plan view schematic (left) and photograph (right) of test apparatus.

Ice rubble was created by manually crushing plates of ice frozen in an array of trays. The original thickness of the ice plates was approximately 18 mm. When crushed, the ice formed angular pieces with varying dimensions. The ice rubble was sorted based on size

using a series of sieves. The sieves were constructed of wire mesh with square openings with sizes of 51 mm, 38 mm, 25 mm, and 19 mm. The size distribution of ice rubble used for each test was kept constant; rubble sized within the range of 19-25 mm made up 25% of the rubble batch mass, 25-38 mm made up 50%, and 38-51 mm made up the remaining 25%. The air temperature in the room was recorded with a temperature sensor (accuracy of $\pm 0.002^{\circ}\text{C}$). Two additional temperature sensors were positioned in the tank to record the water temperature prior to each test. The rubble ice temperature was measured with thermistors before it was placed in the tank.

3.2.3.2 Experimental Procedure

Each test began by cooling the tank water to 0°C , or slightly lower (typically between -0.007°C and -0.015°C as measured by the temperature sensors). Any skim ice forming at the water surface of the tank was removed, and the water temperatures at two locations at approximately mid-depth were recorded just prior to the introduction of the ice rubble. The initial temperature of the ice rubble was typically between 0°C and -2°C . The ice rubble was constrained to the 500 mm long section of the tank in front of the compression plate. The 1000 mm long section of the tank behind the compression plate remained in an open water condition. Two nylon plastic sheets (20 mm thickness) were placed between the tank walls and the ice rubble to prevent freezing to the walls during the freezing phase. The sheets were carefully removed prior to the resistance testing phase. Thus, the force applied to covers comprising rubble ice with a frozen solid crust was transferred directly from the compression plate through the cover to the end wall, with no shear or freezing effects at the side walls. The plastic sheets were not used for tests with zero freezing duration, as rubble ice on its own cannot remain unsupported in a vertical

plane. Thus, the tests with rubble ice only without freezing included frictional effects from the side walls.

In the freezing phase, the rubble ice cover was left in the cold room for a prescribed duration with the room air temperature set to -10.5°C . During this time, the temperature sensor measured air temperatures at a sampling interval of 30 seconds. Following the freezing phase, the sheet ice in the (previously) open water section of the tank was carefully broken and the thickness was measured with digital calipers. Four ice samples were taken; one from each approximate quadrant of the previously open portion. Three ice thickness measurements were recorded for each sample, and the results were averaged to determine the sheet ice thickness. On the rubble side of the tank, the nylon plastic sheets were carefully removed from their position, leaving a gap of approximately 20 mm between the partially frozen ice cover and the side walls of the tank. Thus, the cover was not laterally constrained during the resistance test. In the resistance testing phase, the motor was commissioned to move the compression plate at a speed of 1 mm/s into the ice cover. This corresponds to an initial strain rate of 0.002 s^{-1} , which is within the range of strain rates of other lab experiments on floating ice rubble (e.g. Cheng and Tatinclaux, 1977; Tatinclaux et al., 1977). During the plate movement, the resisting force generated by the ice cover was recorded with the load cell at a sampling frequency of 500 Hz. The position of the plate was recorded with the LMT, also at a sampling frequency of 500 Hz. The plate was moved a total distance of 250 mm, although the peak resistance typically occurred within the first 5 mm of plate movement. A camera was used to record video of the resistance test to evaluate the failure mode of each ice cover.

Upon completion of a test, the plate was retracted to its original position. Photographs were taken of the partially frozen fragments of the failed cover to qualitatively evaluate the appearance of the cover. The ice was removed with a sieve and a new batch of rubble was used for subsequent tests.

3.2.3.3 Test Series and Scope

The test series focussed on modifying two variables: ice rubble layer thickness and freezing duration. The rubble layer thickness was varied by controlling the mass of ice rubble added to the tank. Five kilograms of ice rubble was used to create a thin rubble layer (50 mm), 10 kg of rubble was used to create a medium thickness (90 mm), and 20 kg of rubble was used to create a thick rubble layer (155 mm). The original test series focussed on rubble layers of 90 mm and 155 mm, but some additional tests were performed with the 50 mm rubble layer to further investigate the effect of rubble layer thickness and presence on peak resistance. A skim ice cover formed with 0 kg of rubble ice (i.e. open water) was also tested. The nominal ice cover characteristics produced by the different rubble masses are shown in Table 3.1. The rubble thickness was measured with a ruler fastened to a wide acrylic base. The ruler was dipped under the rubble and gently pulled up until the acrylic base just touched the underside of the ice, and the distance to the top of the rubble layer was measured. Typically the measurements were within ± 5 mm of each other, with some variation due to the orientation of rubble pieces near the bottom of the cover. Five ice covers for each rubble mass were measured and the average thickness was recorded as the nominal ice cover thickness, reported to the nearest 5 mm. The approximate bulk ice cover porosity was calculated from the rubble layer thickness and rubble mass for each test condition. Photographs of the rubble ice covers prior to freezing are shown in Figure 3.3.

The 50 mm cover had rubble pieces filling the entire surface area of the water, but was only one layer thick. Some rubble pieces were oriented on their sides and extended further beneath the water surface than others. The 90 mm cover was multiple rubble pieces thick, and was visually more tightly packed than the 50 mm cover. The 155 mm cover was also multiple rubble pieces thick, was most tightly packed, and had noticeably more ice rising above the water surface than the 50 mm and 90 mm covers.

A set point air temperature of -10.5°C was always used, but the freezing duration was varied to achieve different freezing indices. Several tests were conducted for nominally identical conditions to evaluate the variability in peak resistance and failure mode.

Table 3.1. Nominal ice cover characteristics.

Rubble Mass [kg]	Cover Thickness [mm]	Cover Porosity
0	0	1
5	50	0.58
10	90	0.54
20	155	0.46

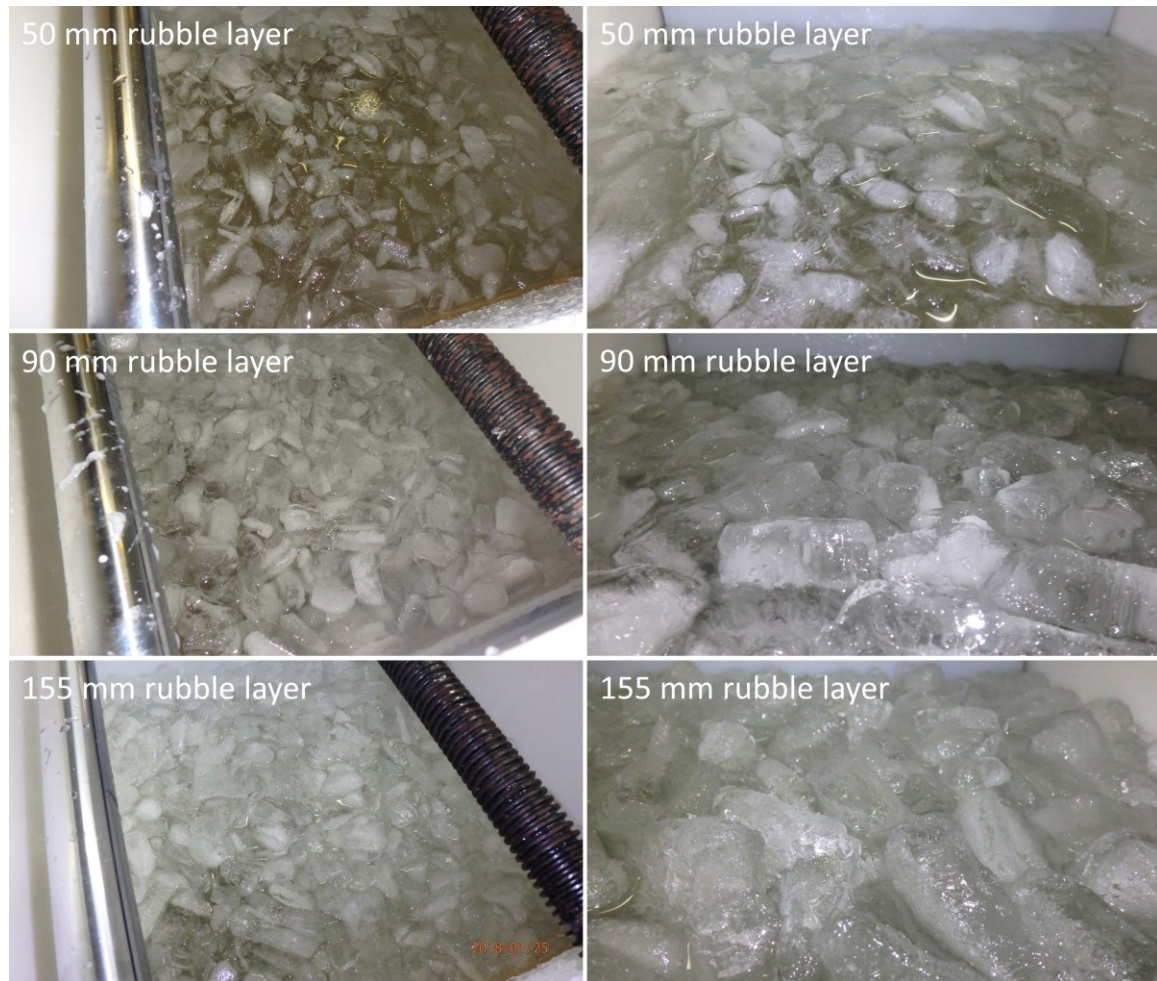


Figure 3.3. Appearance of rubble ice covers prior to freezing for 50 mm rubble layer (top), 90 mm rubble layer (middle), and 155 mm rubble layer (bottom).

3.2.4 Results

The results of all resistance tests are shown in Table 3.2 to Table 3.5. The tests were identified using the convention: rubble mass – freezing duration – test number. The freezing duration is the length of time that the ice cover was exposed to the cold ambient air temperature before its consolidation resistance was tested. The freezing index, measured in degree-minutes of freezing ($^{\circ}\text{C}\cdot\text{min}$), was calculated from the time series of measured air temperatures from the temperature logger. As shown, some variation exists in the freezing index for identical freezing durations due to small deviations in ambient air

temperature from -10.5°C . The sheet ice thickness refers to the average thickness of the ice frozen in the open portion of the tank measured with digital calipers. The peak resistance and displacement are the maximum force recorded by the load cell and coincident displacement of the compression plate as measured with the LMT.

Table 3.2. Skim ice cover results.

Test ID	Freezing Duration [min]	Freezing Index [$^{\circ}\text{C}\cdot\text{min}$]	Sheet Ice Thickness [mm]	Peak Resistance [N]	Displacement at Peak [mm]
000-060-1	60	647	1.61	355	0.09
000-060-2	60	632	1.58	253	0.12
000-060-3	60	629	1.54	278	0.14
000-120-1	120	1272	2.91	1265	0.45
000-120-2	120	1258	3.04	932	0.35
000-120-3	120	1260	2.92	1034	0.53
000-180-1	180	1886	4.26	2534	0.83
000-180-2	176	1852	4.27	2192	0.82
000-180-3	177	1878	4.09	2191	0.92

Table 3.3. Five kilogram (50 mm thickness) rubble cover results.

Test ID	Freezing Duration [min]	Freezing Index [$^{\circ}\text{C}\cdot\text{min}$]	Sheet Ice Thickness [mm]	Peak Resistance [N]	Displacement at Peak [mm]
005-100-1	100	1060	2.33	2949	1.67
005-100-2	100	1021	2.43	2519	1.36
005-100-3	100	1044	2.26	2812	1.66

Table 3.4. Ten kilogram (90 mm thickness) rubble cover results.

Test ID	Freezing Duration [min]	Freezing Index [°C-min]	Sheet Ice Thickness [mm]	Peak Resistance [N]	Displacement at Peak [mm]
010-000-1	0	0	0	13	14.28*
010-000-2	0	0	0	28	42.67*
010-000-3	0	0	0	21	33.34*
010-000-4	0	0	0	11	48.43*
010-030-1	30	322	1.18	205	4.17
010-030-2	30	314	0.93	166	1.94
010-060-1	60	639	1.55	1037	2.50
010-060-2	60	623	1.60	1095	1.53
010-060-3	60	636	1.32	858	0.99
010-060-4	60	629	1.64	785	1.97
010-060-5	60	613	1.47	1624	2.34
010-060-6	60	640	1.59	1486	1.36
010-080-1	80	827	2.55	2375	2.46
010-090-1	90	930	2.56	2870	1.68
010-120-1	120	1278	2.80	3553	2.81
010-120-2	120	1261	2.97	4354	3.01
010-120-3	120	1266	2.71	2954	2.57
010-120-4	120	1241	3.12	4743	1.94

*only plate displacements less than 50 mm considered

Table 3.5. Twenty kilogram (155 mm thickness) rubble cover results.

Test ID	Freezing Duration [min]	Freezing Index [°C-min]	Sheet Ice Thickness [mm]	Peak Resistance [N]	Displacement at Peak [mm]
020-000-1	0	0	0	142	26.24*
020-000-2	0	0	0	115	23.05*
020-000-3	0	0	0	77	33.99*
020-030-1	30	318	0.81	386	0.60
020-030-2	30	320	0.88	240	1.05
020-060-1	60	644	1.50	2059	1.54
020-060-2	60	641	1.53	1168	1.86
020-060-3	60	621	1.71	865	2.07
020-060-4	60	641	1.54	1162	0.99
020-075-1	75	800	1.75	2852	1.81
020-090-1	90	957	2.45	2728	1.96
020-090-2	90	965	2.25	3253	2.49
020-105-1	105	1186	2.78	4118	1.75
020-120-1	120	1288	3.15	5169	3.27
020-120-2	120	1355	3.01	5287	2.38
020-120-3	120	1319	3.17	5549	3.00

*only plate displacements less than 50 mm considered

3.2.4.1 Observed Growth of Ice Layers

Equation [3.1] can be used to calculate the incremental growth of a sheet ice layer in open water, where h is the sheet ice thickness, subscript j indicates a time step, ρ_i and L_i are the density and latent heat of fusion of ice (taken to be 920 kg/m³ and 333,400 J/kg, respectively), t is time, T_m and T_a are the melting temperature of ice (0°C for freshwater) and ambient air temperature, k_i is the thermal conductivity of thermally grown ice (taken to be 2.27 W/m/°C), and H_{ia} is the heat transfer coefficient at the ice-air interface (Ashton, 1986). The relationship between measured sheet ice thickness and time for all test conditions is shown in Figure 3.4. Using Equation [3.1] with a time step of 30 seconds and an assumed constant air temperature of -10.5°C, it was determined that a surface heat transfer coefficient, H_{ia} , of 12 W/m²/°C provided a good fit to the measured data. As there is some variability in the measured data owing to the difficulty in measuring such small thicknesses and the spatial variation across the sheet ice cover, the average sheet ice thickness calculated with Equation [3.1] was used in subsequent analyses rather than the small sample of physical measurements. Note the apparent linearity of the theoretical line in Figure 3.4 caused by the minimal insulating effects of the thin solid ice layer. Heat transfer coefficients for natural rivers range from 10-25 W/m²/°C corresponding to wind speeds of 0-4 m/s (Ashton, 1986). Therefore, the conditions in the cold room were on the lower end of this range, representative of typical freeze-up conditions with low wind speeds. The lab environment did not capture the influence of solar radiation as would be experienced in natural rivers.

$$h_{j+1} = h_j + \frac{1}{\rho_i L_i} \left[\frac{\Delta t (T_m - T_a)}{\frac{h_j}{k_i} + \frac{1}{H_{ia}}} \right] \quad [3.1]$$

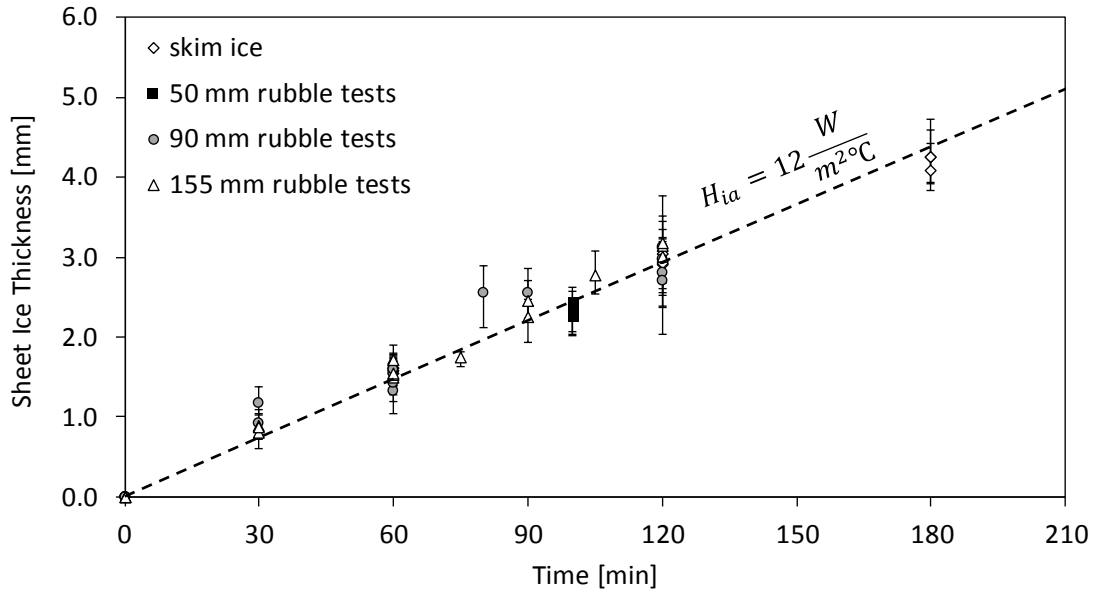


Figure 3.4. Measured sheet ice thickness for all tests. Markers indicate average values while whiskers indicate maximum and minimum single measurements. A heat transfer coefficient of $12 \text{ W/m}^2/\text{°C}$ was found to best fit to the measured data (Equation [3.1]).

When a solid crust grows through a rubble ice layer for an extended period of time (i.e. the thickness of the crust extends over several voids), the rate at which the solid ice layer grows increases by a factor of $1/p_j$, where p_j is the porosity of the rubble accumulation, due to the fact that only water in the interstices must freeze (Ashton and Beltaos, 2013; Calkins, 1979). Typical rubble layer porosities range from 0.4-0.6, thus the growth rate of the solid crust layer can theoretically be about 1.7-2.5 times greater than the growth of an ice sheet in open water. Beltaos (2013a) noted that freezing at the scale of void spaces in a rubble cover can be complex, and that the thickness calculated with the heightened growth rate represents the spatially averaged thickness over a large area. At

very early stages of crust development (when the thickness of the solid crust layer is still less than the average thickness of a void), the factor of $1/p_j$ does not apply uniformly over the entire surface area of the ice cover. Indeed, when the solid crust and rubble were examined following the resistance tests, it was found that the crust thickness varied and was not consistently thicker than the sheet ice grown in the open water area. Instead, the solid crust in the void spaces at the surface, not near the rubble, was approximately the same thickness as the sheet ice (see Figure 3.5). For one test with a 90 mm rubble layer and a freezing duration of 120 minutes, the average sheet ice thickness grown in open water was 2.82 mm while the average solid crust thickness (considering areas away from rubble pieces where the crust had an identifiable thickness) was 2.78 mm. Closer to the rubble pieces, the thickness of the solid crust was slightly greater due to ice growth on the surface of the rubble. There were often pieces of rubble in lower layers bonded to the rubble pieces in the surface layer, but below the depth of the solid crust (see Figure 3.5). In essence, the minimum thickness of the solid crust was approximately equal to the thickness of the adjacent sheet ice. The theoretical increased rate of ice growth due to the porosity of the rubble layer seemed to add some ice thickness near rubble pieces, and allowed freeze-bonding of rubble pieces at contact surfaces in the surface layer and slightly beyond. Due to the difficulties experienced in accurately quantifying the interstitial crust thickness and other freezing processes near the surface of the rubble layer, the freezing index was used in subsequent analyses to characterize each ice cover. The associated thickness of an ice sheet grown over the same freezing index is also reported to provide some context on the relative thickness of the frozen layer to the total ice cover thickness.

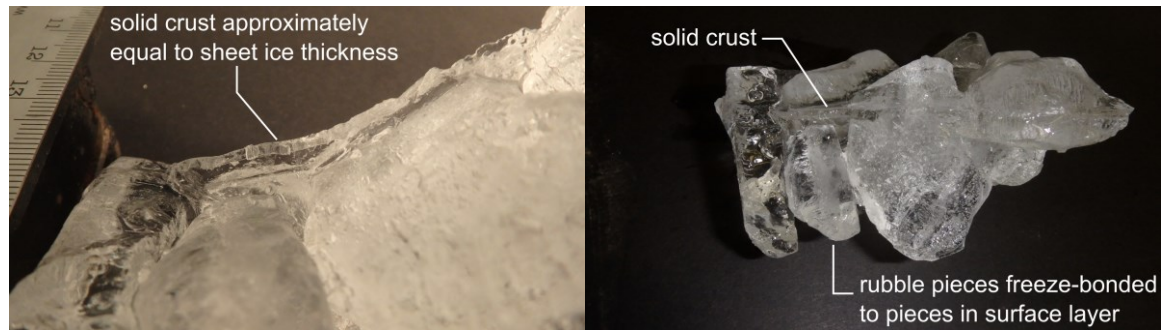


Figure 3.5. Photographs of solid crust and freeze-bonds between rubble pieces.

3.2.4.2 Load-Displacement Measurements

For each experiment, the displacement of the compression plate was plotted with the concurrent load recorded by the load cell. In general, the load-displacement plots have several distinct regions. First, the load increases approximately linearly with displacement when the plate begins to move. As the cover is compressed, the load reaches a peak value. The load will then decrease with continued plate displacement. Ice covers with a competent solid crust may fracture suddenly, causing the load to decrease rapidly, or, if the crust is very thin and somewhat flexible, the load will decrease more gradually. The initial peak represents the most significant contribution of the solid crust. After this peak, the cover has failed and the residual load recorded is caused by frictional sliding of the rubble pieces and the breaking of solid crust remnants that were not destroyed in the initial failure.

It has been shown by Sinha and Frederking (1979) that the stiffness of the test system can result in an actual strain on the ice cover that is less than the nominal strain as recorded from the movement of the compression plate. In the current study, the test apparatus was designed to be as rigid as possible. The compression plate was constructed of steel, and a steel backing plate was positioned directly behind the end wall of the inner tank to minimize any deformation while applying load to the ice covers. However, with

some deformation of the test apparatus, the actual displacement of the ice cover would be slightly less than the plate displacement as recorded by the LMT. Therefore, the actual load-displacement plots (considering true displacement of the ice cover itself) would likely exhibit slightly less deformation before failure at higher loads. Tests performed by Sinha and Frederking (1979) showed that the peak resistance is not significantly influenced by the stiffness of the apparatus, which is the main focus of the current experiments.

Load-displacement measurements are shown in Figure 3.6 for tests conducted with freezing durations of zero and 30 minutes, corresponding to freezing indices of 0 °C-min and 315 °C-min, respectively. To remove noise registered by the load cell at low loads caused by the mechanical system, a moving average filter was applied to the data points. A moving average span corresponding to 5 mm of plate travel was used for the zero freezing duration tests. For the 30 minute freezing duration tests, the moving average was applied to data only after the initial peak in order to preserve the detail near the peak load. The load registered by the load cell when only moving the plate (with no ice cover) was approximately constant near 60 N for the majority of the displacement range; this load was subtracted from the peak load to obtain the values presented in Table 3.2 to Table 3.5.

The load-displacement curves for tests with 315 °C-min of freezing show a more pronounced initial peak than the tests with no freezing. This is due to the breaking of the thin crust and weak freeze-bonds that formed at the surface of the ice cover. Although the peak loads for the 315 °C-min tests appear to occur instantaneously, they are in fact spread over several millimetres of plate displacement and did not correspond to a sudden fracture in the ice cover. Instead, the ice cover slowly and continuously deformed during the peak. The failure location was not consistently in the same location (e.g. just in front of the plate),

but could occur anywhere in the cover. Following the initial peak resistance, the loads for the 315 °C-min tests decreased to approximately the same loads, or slightly greater, as those registered during the 0 °C-min tests. This indicates that the freezing of the ice cover had an influence on the initial peak resistance, but also on the residual resistance occurring after the initial failure. The increase in loads recorded at greater plate displacements (i.e. beyond 100 mm) were caused by the thickening of the ice cover and increased buoyant confining forces.

Loads recorded in the 155 mm rubble layer tests were generally greater than those of the 90 mm rubble layer tests. For tests with 315 °C-min of freezing (corresponding to 0.8 mm of sheet ice growth), the average peak resistance of the 155 mm rubble was 313 N, while the average peak resistance of the 90 mm rubble was 186 N. With no freezing, the average resistance of the 155 mm rubble ice cover was 111 N, while the average resistance of the 90 mm cover was only 18 N; the discrepancy is attributable to the effects of buoyancy on the ability of the rubble to develop internal friction. The 155 mm cover with no freezing also had slightly more pronounced initial peaks than the 90 mm cover, which gradually attained its peak resistance, although one of the 155 mm tests also exhibited this behaviour. Finally, the 155 mm cover had more variable loads recorded later in the test than the 90 mm cover.

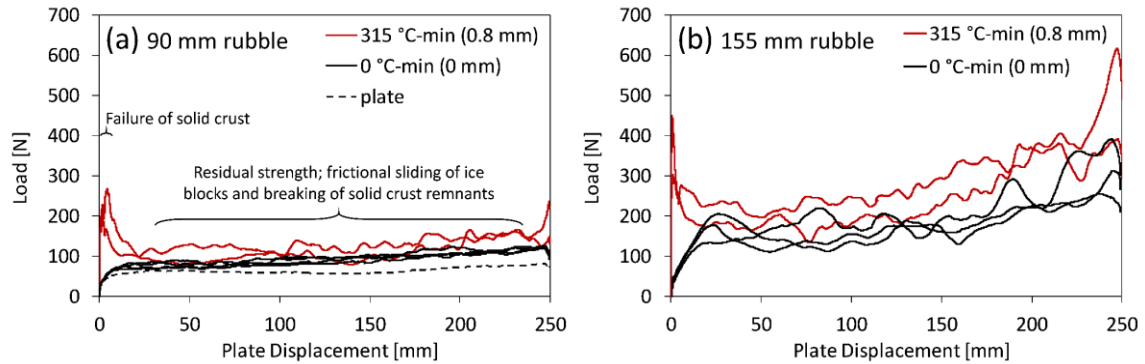


Figure 3.6. Load-displacement plots for tests with 30 minutes of freezing or less; (a) 90 mm rubble layer, (b) 155 mm rubble layer. Legend indicates nominal freezing index and associated sheet ice thickness corresponding to each test.

Load-displacement measurements are shown in Figure 3.7 for tests conducted with freezing durations of 60 minutes or greater. The initial peak resistance obtained for these tests was much greater than the residual resistance provided following failure. In general, the load increased approximately linearly with plate displacement early in the deformation period (typically less than 0.5 mm of plate displacement). As the plate continued to move, the slope of the load-displacement curve decreased. The load continued to increase with plate displacement until failure of the ice cover. At this point, the ice cover achieved its peak resistance and the measured load decreased with further displacement of the plate. Beyond the failure point, the residual load returned to values near or slightly higher than the load corresponding to rubble without freezing (i.e. in the range of 100-300 N). Additional peaks (typically less than 500 N) were recorded as the plate crushed portions of the ice cover with solid crust still intact.

For tests with a freezing index of 630 °C-min, the load-displacement curve flattened out significantly near the peak load; the slope was nearly zero indicating the resistance was relatively constant as the plate continued to deform the cover. Measured loads then

decreased gradually as the ice cover failed. Video recordings indicate that the gradual failure often involved the frozen portion of the cover falling out of plane (either heaving up or down), with some localized crushing of the ice cover and/or sliding of a portion of the cover below the other. For tests with a freezing index of 1260 °C-min, the load-displacement curve also obtained a zero slope near failure, but only for a short time as the cover continued to deform. Failure was signified by a rapid decrease in load (to nearly zero) as the ice cover collapsed dramatically. These sudden failures are shown by X markers in Figure 3.7. Visual observations indicate that the cover would suddenly fracture at the surface; the fracture was typically not one single straight line extending laterally across the cover, but instead followed an irregular path between the surface rubble and could be composed of multiple segments. Generally, ice covers subjected to longer freezing durations (i.e. greater freezing indices) failed at greater plate displacements and achieved higher peak resistance values. The slopes of the load-displacement plots for skim ice covers did not plateau before failure as they did for the rubble ice covers. The failure mode of the skim ice covers was buckling, typically including two lateral fractures that extended across the cover approximately parallel to the compression plate.

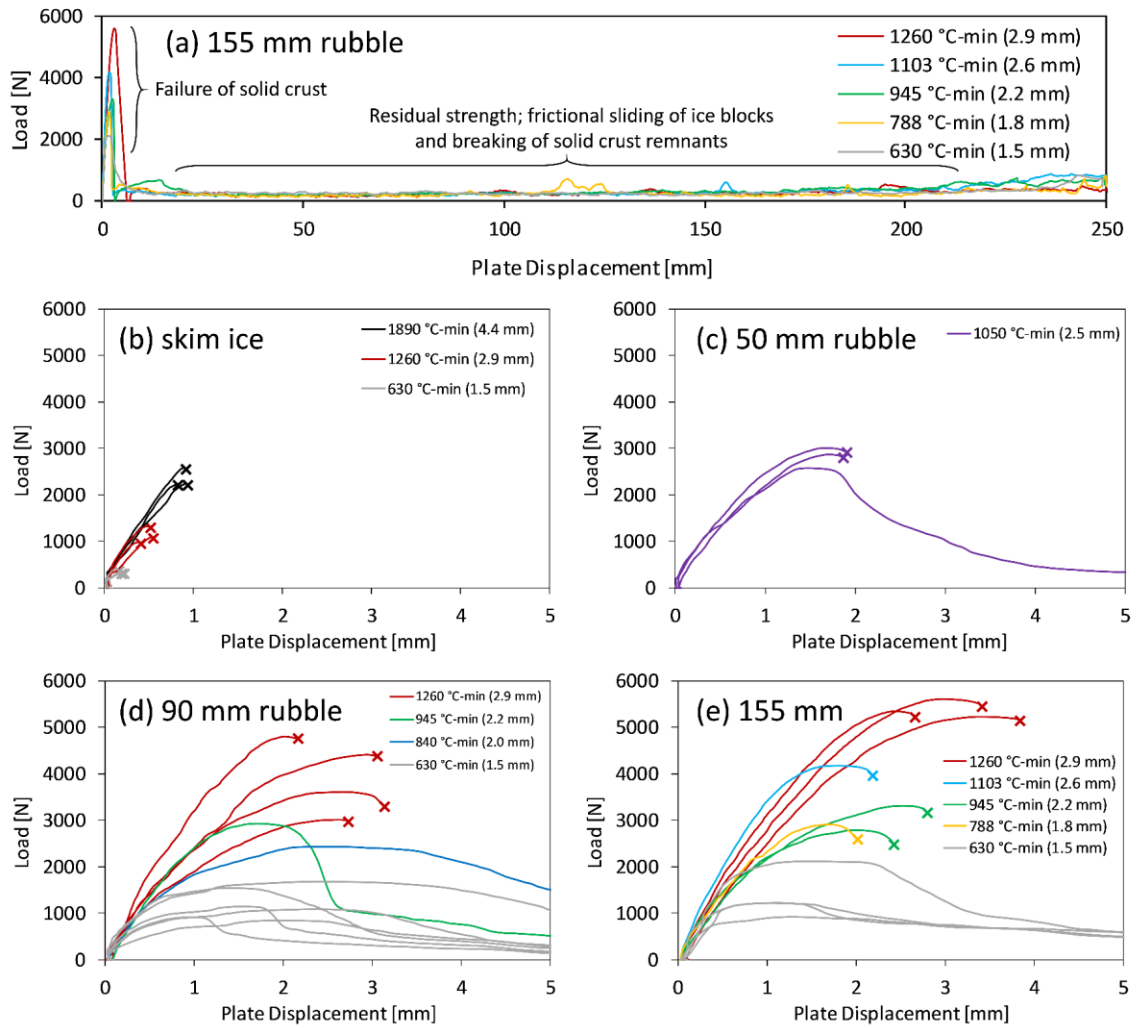


Figure 3.7. Load-displacement plots for tests with 60 minutes of freezing or greater. Top plot (a) shows subset of 155 mm rubble tests for entire stroke of plate displacement. Bottom plots shows zoomed in view near failure for (b) skim ice, (c) 50 mm rubble layer, (d) 90 mm rubble layer, and (e) 155 mm rubble layer. Fracture failures are indicated by X markers. Legend indicates nominal freezing index and associated sheet ice thickness corresponding to each test.

3.2.4.3 Peak Resistance to Consolidation

The relationship between peak resistance and freezing index is presented in Figure 3.8. As shown, the peak resistance of an ice cover increases with prolonged exposure to cold ambient air temperatures. The relationship does not appear perfectly linear over the entire range of freezing indices tested. Consolidation resistance gained from 0 °C-min to

315 °C-min was found to be approximately 0.6 N/°C-min. From 630 °C-min to 1260 °C-min, the slope increases to approximately 5.0 N/°C-min.

The thickness of the rubble ice layer had a smaller effect on the peak resistance. Near a freezing index of 630 °C-min, peak resistances for 90 mm rubble ranged from 785-1624 N (average of 1148 N), while peak resistances for 155 mm rubble ranged from 865-2059 N (average of 1313 N). Although there is overlap in the ranges, the highest peak resistance obtained for the 155 mm rubble was greater than that of the 90 mm rubble and the lowest value of resistance measured for the 90 mm rubble was less than that of the 155 mm rubble.

In the range of 800-1100 °C-min, all three rubble layer thicknesses recorded peak resistances between 2375 N and 3253 N. Tests performed with thicker rubble appear to have marginally higher peak resistances. For example, tests 020-075-1 and 010-080-1 both corresponded to a freezing index near 800 °C-min, and the 155 mm rubble recorded a peak resistance almost 500 N higher than the 90 mm rubble. Tests 020-090-1, 020-090-2, and 010-090-1 corresponded to freezing indices near 950 °C-min; the 155 mm rubble recorded resistances of 2728 N and 3253 N while the 90 mm rubble recorded a peak resistance of 2870 N which is between the values of the 155 mm rubble. The 50 mm rubble layer tests corresponded to a slightly higher freezing index of 1050 °C-min, so a direct comparison is not possible. However, based on the slope of the best fit line it appears as though the peak resistance values of the 50 mm tests would sit approximately 500 N below the resistances obtained by the 155 mm rubble covers.

At a freezing index of approximately 1260 °C-min, the 90 mm rubble recorded resistances from 2954-4743 N (range of 1789 N), showing more variability than the 155 mm rubble loads which were more concentrated from 5169-5549 N (range of 380 N). It is not clear why this is the case, but it is possible that the thicker rubble ice with higher buoyant forces grew a more consistent solid crust at its surface (i.e. less possibility of a large gap between rubble pieces for the crust to bridge). The peak resistances obtained for the skim ice covers are all much lower than those obtained by a rubble ice cover at the same freezing index. For example, near 630 °C-min, the average skim ice resistance was approximately 300 N, while the average resistance for a rubble cover with a solid crust (combination of 90 mm and 155 mm tests) was approximately 1200 N. This discrepancy increases with higher freezing indices; near 1260 °C-min, the average skim ice resistance was approximately 1100 N, while the average resistance for tests with a rubble layer with a solid crust was approximately 4500 N. Resistance gained by the skim ice cover from 630 °C-min to 1890 °C-min is approximately 1.6 N/°C-min, much less than the rate of resistance gained by the rubble ice covers (5.0 N/°C-min) over the same range.

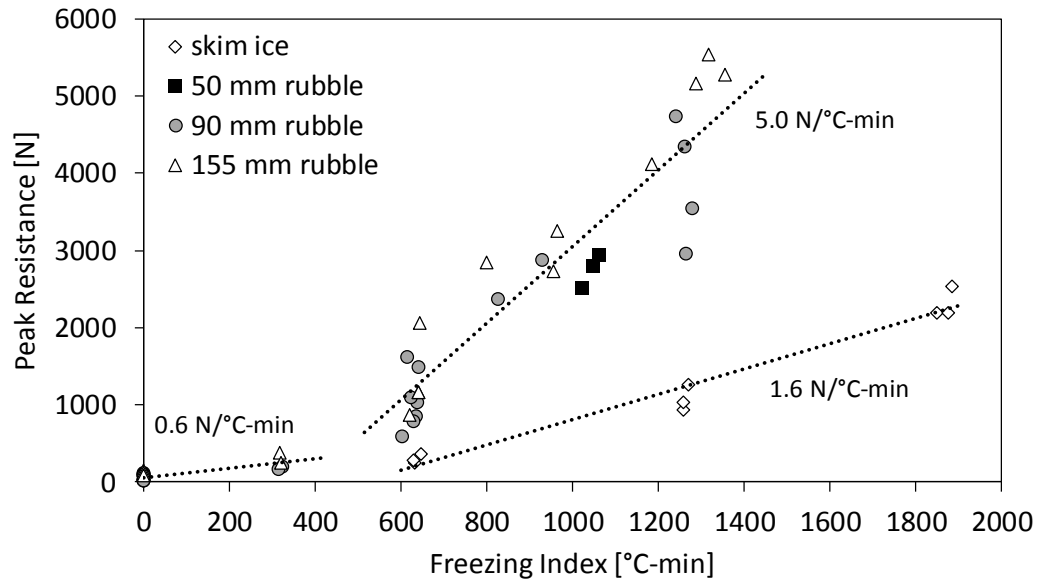


Figure 3.8. Peak resistance variation with freezing index.

Following publication of the initial test series, 22 additional experiments were conducted to confirm the trends of the original data, and to investigate the effect of changing the ambient air temperature to -5.0°C and -20.0°C . Figure 3.9 shows the combined dataset; the additional data confirms the trends in the original test series. Although only seven tests were conducted at a different set point air temperature, the data suggests that the peak strength is most influenced by the freezing index, independent of the air temperature that was used to achieve it. Additional tests over the entire range of freezing indices for various set point air temperatures could investigate this further.

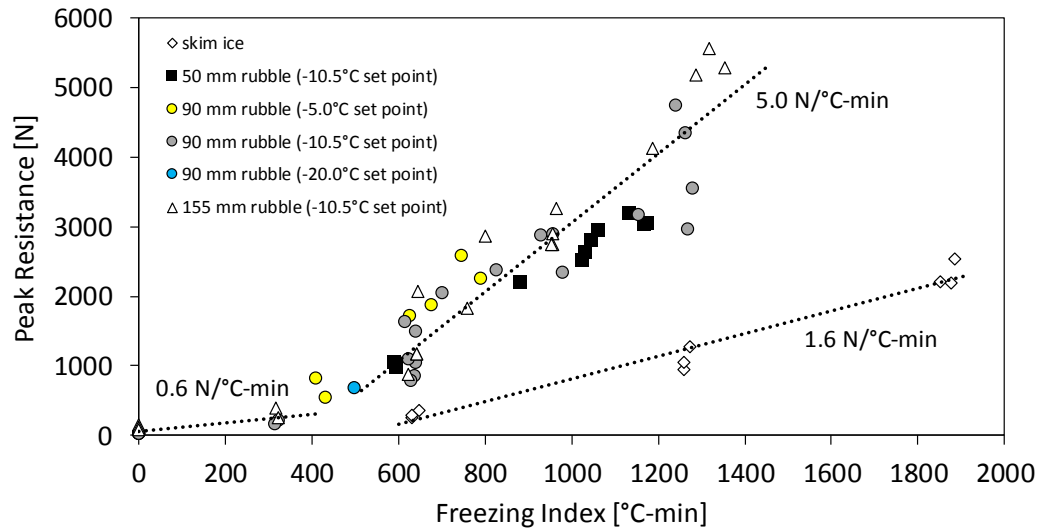


Figure 3.9. Peak resistance variation with freezing index, including additional tests after original publication.

3.2.5 Discussion

3.2.5.1 Effect of Exposure to Freezing Ambient Conditions

Experimental results indicate that an ice cover exposed to cold ambient conditions for longer durations will have a higher peak resistance; however, the apparent gain in resistance was not constant over the range tested. The ice cover was observed to behave differently depending on the freezing index and the corresponding frozen crust layer that was able to form. Without any freezing, the rubble pieces slid against each other and rearranged almost continuously as the plate applied a load. The resistance to deformation was provided solely by the frictional interlocking and sliding between rubble ice pieces. A distinct peak load was not obvious.

At freezing indices from 315-630 °C-min, the thin crust layer and freeze-bonds added some non-frictional resistance, but were still too early in their development to allow the cover to behave rigidly. Rather, the ice cover appeared to be flexible, and could heave

up or down slightly without cracking. Localized failure of the thin crust gradually allowed some relative movement of the rubble. Failure typically occurred when a portion of the cover gradually slid underneath the other portion. The slow failure was captured in the load-displacement measurements as an extended peak where the slope was nearly zero. The resistance of the cover slowly decreased following the peak.

At freezing indices near 1260 °C-min, the thicker solid crust and freeze-bonds comprised a cover that was more rigid than flexible. As the plate applied force to the cover, there was no initial apparent movement of rubble pieces relative to one another; instead, the cover compressed as a single unit. Failure was characterized by a sudden fracture that appeared at the surface of the ice cover through the solid crust. The fracture reduced the resistance provided by the cover to nearly zero immediately.

The different behaviours of the ice cover at various stages of freezing could explain the apparent non-linear gain in peak resistance shown in Figure 3.8. The peak resistance of the ice cover increases approximately linearly in tests with freezing indices above 630 °C-min; this is approximately the same freezing index at which the failure mode changes from gradual crushing/heaving to sudden fracture in Figure 3.7. This suggests that once the solid crust and freeze-bonds have developed sufficiently to allow the ice cover to behave rigidly, the added resistance with additional freezing becomes approximately constant.

The variability in peak resistance for identical tests with similar freezing indices can be attributed to some randomness in the orientation of the rubble pieces between tests (particularly near the ice-air interface), which could result in changes to surface contact

between ice pieces and the open water areas between them. The ice cover was often observed to fail by fractures to the surface crust layer that were not consistent in shape or orientation; they presumably followed the weakest path through the ice cover, which always went around individual pieces of ice rubble and not through them. Variability could also result from small changes to ambient atmospheric conditions (i.e. air currents, relative humidity, etc.) between tests that influence the freezing of the ice cover at the surface; however, these were kept as consistent as possible during testing and are expected to have contributed minimally to the observed variability. Despite the variability in tests with nominally identical conditions, the trends in Figure 3.8 are clear and highlight the relationship between peak ice cover resistance and the length of time exposed to freezing ambient conditions.

3.2.5.2 Effect of Rubble Ice Presence

The peak resistances of ice covers comprised of rubble with a thermally grown surface crust were consistently greater than the sum of their parts. For example, as shown in Figure 3.8, the resistance of a 155 mm rubble cover with no freezing was approximately 110 N. A skim ice cover approximately 2.9 mm thick (corresponding to a freezing index of 1260 °C-min) recorded a peak resistance of about 1100 N. However, when the 155 mm rubble ice cover was exposed to the freezing ambient conditions for 1260 °C-min, the peak resistance was approximately 5300 N, much greater than the summation of 110 N (the rubble-only contribution) and 1100 N (the skim ice-only contribution).

This is likely due to the added stability of the ice cover provided by the rubble. At the early stages of solid crust growth investigated in this study, it was observed that the minimum thickness of the solid crust was approximately the same thickness as the sheet

ice. In a skim ice scenario (no rubble ice), the entire streamwise length of the cover is unsupported (except for the small buoyant force keeping it floating), and the thickness is limited to the thickness of the skim ice. The characteristic length of the skim ice sheets ranged from 0.12-0.27 m (for thicknesses of 1.5-4.4 mm). This results in a cover that is prone to instabilities and buckling failure when a load is applied. All failures for skim ice covers involved buckling with relatively straight fractures that extended laterally across the cover, as illustrated in Figure 3.10. With the presence of a rubble ice layer, the growth of the solid crust bridges the voids between the competent rubble pieces at the surface. This reduces the maximum span of a given section of solid ice from the entire cover length in the skim ice scenario (500 mm) to only a few millimetres with the presence of rubble ice. Although the rubble pieces are not fixed in place, they are held in place by the buoyant forces acting on the rubble layer below, thus provide some form of support to the solid crust. Furthermore, the depth of influence of the frozen layer increases with the presence of ice rubble; not only is the solid crust fixing the cover in place, but so too are the freeze-bonds that develop in the upper layers of the rubble. This increases the characteristic length of the ice cover, as well as the thickness-to-width and thickness-to-length ratios of the rigid (frozen) portion of the ice cover. In effect, the presence of the ice rubble creates a cover that is less prone to instabilities and buckling failure. Failure occurred when the surface fractured in an irregular pattern. Often the fracture involved a number of segments, presumably following the weakest path between the rubble pieces (see Figure 3.10). This could explain why the peak resistance of a cover comprising rubble ice with a surface crust was much greater than the sum of its parts considered individually.

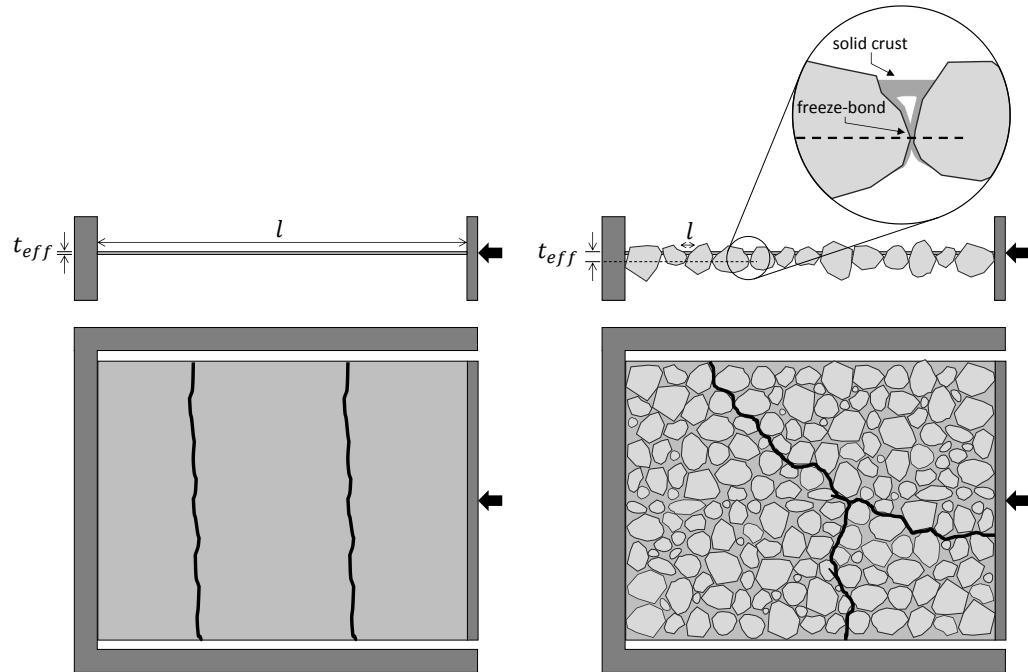


Figure 3.10. Illustration of observed failure modes of skim ice cover (left) and rubble cover with solid ice crust (right). The profile view (top) shows the length of unsupported solid ice, l , and the effective depth to which surface freezing has influenced the ice cover, t_{eff} . The plan view (bottom) shows the observed fracture patterns in ice covers of each type.

3.2.5.3 Effect of Rubble Layer Thickness

The peak resistance of an ice cover comprising rubble ice with a solid crust was less dependent on the thickness of the rubble layer than the freezing index. However, ice covers with thicker rubble layers tended to record slightly greater peak resistances. The 90 mm rubble layer at 630 °C-min of freezing achieved an average peak resistance of approximately 1150 N, while the 155 mm rubble with the same freezing index recorded an average peak resistance of 1310 N. At 1260 °C-min of freezing, the 90 mm rubble layer achieved an average peak resistance of 3900 N, while the 155 mm rubble layer recorded an average peak resistance of 5340 N. Peak resistances of the 155 mm rubble layer were also marginally greater than the 90 mm rubble layer for tests conducted between 800 °C-min and 1100 °C-min. The trend of the data in Figure 3.8 indicates that the resistances

recorded by the 50 mm rubble layer would fall slightly below those of the 90 mm and 155 mm rubble layers at a freezing index near 1050 °C-min.

Examination of the ice cover after testing indicated that most of the freezing effects were localized at the surface. In this region, the rubble ice pieces were strongly frozen together by the solid crust. Some freeze-bonding was observed at contact points between rubble pieces in the surface layer. Beneath the depth of the solid crust, weaker freeze-bonds allowed some rubble pieces to adhere to the rubble in the surface layer. These pieces were more easily pulled apart by hand and would likely not add significant consolidation resistance compared to the solid crust. The rubble near the bottom of the ice cover in the 155 mm rubble layer tests was observed to be completely loose and unfrozen. This observation, along with the rubble resistance tests with no freezing, suggest that the freezing at the surface in the form of solid crust and freeze-bonds between rubble pieces was responsible for the majority of the resistance gained by the cover. The presence of more ice rubble beyond this layer had a small effect on the peak resistance over the range of freezing indices tested. The higher peak resistances obtained by thicker rubble covers can be attributed to the slightly lower porosity; the rubble is more tightly packed with larger buoyant forces. The solid crust and freeze-bonds near the surface can more easily solidify the rubble pieces together, resulting in a higher peak resistance.

It is possible that the thickness of the rubble layer would have more importance for greater freezing indices. This may allow freeze-bonds further below the surface layer to gain additional strength. More ice rubble would result in more contact points for these freeze-bonds to grow. It is also possible that the rubble layer thickness would have a larger

effect on peak resistance at lower initial rubble ice temperatures, as colder rubble could increase the effects of sub-surface freeze-bonding.

3.2.5.4 Application to Field Studies

Solidification of a rubble ice cover via thermal processes has been observed on the Dauphin River in central Manitoba (Wazney et al., 2018). The ability of a natural ice cover to grow a surface crust is hindered by the seepage velocity of water moving through the voids. The seepage velocity is related to the water surface gradient as well as the porosity, shape, and size of the ice floes (Beltaos, 1999). It is most important near the toe of an ice jam, where the water surface slope is high. In the case of the Lower Dauphin River, assuming the water surface gradient is equal to the bed slope (0.16%), the seepage velocity through the rubble layer would be approximately 2-3 mm/s. The critical velocity for skim ice to grow at the water surface is approximately 70 mm/s (Liu and Shen, 2011), thus it is expected that a solid crust can form in these conditions.

During freeze-up, the ice cover progresses upstream as frazil ice pans juxtapose or ridge up on the leading edge of the stationary cover. Stable portions of the ice cover continually lose heat to the cold ambient conditions. The resulting ice cover no longer behaves as an accumulation of individual pans that resists external forces solely by frictional interlocking between ice pieces. Instead, the cover behaves as a rigid, uniform mass. Figure 3.11 shows photographs of a thin ice cover on the Dauphin River taken with an unmanned aerial vehicle (UAV).



Figure 3.11. Photographs of a rubble ice cover with a thermally grown solid ice crust on the Dauphin River, Manitoba. The left photograph shows an oblique view of the cover composition; frazil pans are frozen together by a solid crust at the surface. The right photograph shows this type of cover fracturing as it moves through a meander during a shove event.

First-hand observations of consolidation events by the authors on the Dauphin River indicate that the stationary front advances at a thickness of one or two frazil pans. Since the ability to generate internal friction is limited by the buoyant forces acting on the cover, this region (called the actively consolidating zone) undergoes frequent localized shove events every 20-30 minutes or less. The short duration of time between shoves hinders the growth of a complete solid ice crust, however portions of the cover can still remain frozen together and behave as larger rafts rather than individual pans (see Figure 3.11). With each shove event, the ice cover thickens, and its ability to resist subsequent shoves is increased. On top of this, the ability to resist shoves means that the cover will be stable for a longer period of time, allowing interstitial water to freeze and further increase the resistance to shoves. Secondary consolidations occur when a mechanically thickened region of the ice cover (not near the leading edge) shoves and consolidates even further. Several hours or days can elapse between secondary consolidation events, meaning that the rubble ice cover has a longer duration to grow a solid ice crust. For example, a series of photographs taken at a one-minute interval with a trail camera on November 22, 2017

indicated that at a site approximately 1 km downstream of the stationary front, a secondary consolidation event occurred at 6:21 am, followed by another event at 3:00 pm. During the approximate 8.5 hours between secondary consolidation events, the cover at this location was stationary, with no relative movement of ice pieces at the surface. Air temperature records indicate that approximately 5000 °C-min accumulated during this time. Assuming a heat transfer coefficient of 12 W/m²/°C (conservative estimate for field conditions), the 120 m wide cover would gain approximately 5000 kN of resisting force due to surface freezing effects. Note that this value is a rough estimate based on the limited experiments conducted in the lab; it is likely that there are scaling effects due to the complexity of the processes involved that prohibit the direct application of lab results to real world scenarios.

Despite the limitations, the results of this experimental study shed light on mechanisms by which the consolidation resistance of an ice cover is enhanced under cold ambient conditions. Subroutines can be added to comprehensive numerical river ice models that compute the growth of a solid crust layer in a stationary ice cover. Ice dynamic calculations would then consider the time-varying strength of the ice cover, which is dependent on the ambient weather conditions and the length of time it has been stationary. This would be an improvement on the current state of practice of calibrating a few constant strength parameters such as friction angle and ice cohesion which are supposed to uniformly represent all ice in the model. Parameters such as the failure load of the solid crust layer, which was computed in the laboratory tests but would likely vary due to field-scale effects, can be calibrated to field datasets as they are acquired.

3.2.6 Conclusion

Ice dynamics at breakup and freeze-up are often treated identically in many numerical river ice models, even though several important differences exist between the two cases. This study aimed to investigate the effect of a thermally grown solid crust on the ability of a rubble ice cover to resist consolidation. In general, the force required to consolidate a rubble ice cover increased with prolonged exposure to cold ambient conditions due to the growth of additional ice in the form of a solid crust at the surface as well as freeze-bonds between rubble pieces. At the early stages of crust development investigated in this study, the thickness of the solid crust layer was found to be approximately equal to the thickness of an adjacent ice sheet grown in open water. At 630 °C-min of freezing (30 minutes exposure to -10.5°C air), the thin crust and freeze-bonds near the surface were not sufficiently developed to provide rigidity to the cover. Failure occurred gradually and involved localized failure of the solid crust and sliding of rubble pieces relative to one another. With prolonged exposure to cold ambient conditions the solid crust was able to grow thicker and more securely bond the surface rubble pieces together. Freeze-bonds also developed at contact points between the rubble; these bonds were strongest at the surface and became weaker with depth. Beyond a freezing index of 630 °C-min, the ice cover behaved more rigidly and failed via sudden fracture of the surface layer. Peak resistance in this range increased by approximately 5 N/°C-min. The peak resistance of a cover comprising rubble ice with a solid crust was found to be much greater than the sum of the individual resistances of the rubble and solid ice layers considered separately. The effect of rubble layer thickness on peak resistance was small over the

conditions tested, but generally thicker rubble covers obtained slightly higher peak resistances than thinner covers.

This study highlights the strong influence of thermal processes on river ice dynamics at freeze-up. The results confirm that only a short duration of time is required for a solid crust and freeze-bonds to form at the surface layer of a rubble ice cover, which can add considerable non-frictional resistance to a consolidation event. Additional research in the form of laboratory tests and field measurements will further the understanding on this subject.

3.2.7 Acknowledgements

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3.2.8 References

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3.3 Conclusions and Contributions of this Work

Results of the laboratory experiments show the influence of the freezing of interstitial water near the surface of a rubble ice cover on its ability to resist mechanical thickening. At the laboratory scale, even short durations of exposure to cold ambient conditions resulted in significant increases to the consolidation resistance of the ice cover. This suggests that an ice cover formed under colder ambient temperatures may have a final thickness that is less than one that forms with warmer ambient temperatures. Again, this highlights the dependence of ice dynamics on thermal processes, and necessitates the consideration of the link between the two in numerical models. Models that account for thermal strengthening may be more robust and provide better estimates of ice-affected water levels and ice thicknesses between seasons at a particular site. The crust strengths, which were found to vary from 1.0-1.5 MPa at freezing indices at the upper end of the range tested, but could be as low as approximately 0.2 MPa at low freezing indices, can be used as a first approximation of the failure stress of the surface crust in numerical models.

However, it is possible that these values represent an upper limit to values applicable to the field due to the ideal laboratory conditions such as no flow through the ice rubble, no relative motion of ice rubble during the freezing stage, and uniaxial application of the force.

Chapter 4 Numerical Modeling of River Ice Cover Formation at Freeze-up

4.1 Introduction

Section 1.2.6 provided an overview of common numerical river ice models. CRISSP2D was selected as the numerical model for this research as it is one of the most sophisticated and advanced two-dimensional models available. Two-dimensional models can calculate the internal stresses experienced by a simulated ice cover more accurately than one-dimensional models, particularly for complex water networks and rivers that exhibit meanders – an important consideration when modeling the resistance of an ice cover to mechanical thickening. CRISSP2D is also capable of simulating unsteady flow conditions which are common during the freeze-up period, especially on regulated rivers. The objective of the research presented in this chapter was to incorporate a relationship between thermal and dynamic ice processes into the CRISSP2D model. Following an overview of the model, new model formulations are presented which simulate the thermal strengthening of a forming ice cover exposed to cold ambient conditions. Simulations were performed on an idealized channel to demonstrate that the formulations function as intended on a conceptual level. Finally, the CRISSP2D model was applied to the case study of the Lower Dauphin River to evaluate its ability to simulate the ice cover formation and associated water levels.

4.2 Overview of CRISSP2D Model

Development of the two-dimensional numerical river ice model, CRISSP2D, began in 2000 as part of the Comprehensive River Ice Simulation System Project (CRISSP). The project was initiated to address engineering issues common to numerous hydropower utilities in Canada and the United States. As mentioned in Section 1.2.6, CRISSP2D was developed as a generalization of DynaRICE, including thermal ice modules and an improved numerical scheme for ice dynamics (Shen, 2002). The model is programmed in structured modular form using Intel Visual FORTRAN to allow for module updates to be performed as new theories on ice processes are developed. CRISSP2D is capable of simulating a variety of ice formation processes in complex river geometries. The following sections provide a high level overview of the CRISSP2D program to give the reader some context on the treatment of different ice processes in the model. Attention will be given to the treatment of ice dynamics, as this is the model component that will be modified in this research. Detailed descriptions of each module can be found in the CRISSP2D User's Manual and CRISSP2D Programmer's Manual (Liu and Shen, 2011).

4.2.1 Hydrodynamic Module

River hydrodynamics are determined by solving the depth-averaged Navier-Stokes equations using the explicit Petrov-Galerkin finite element method (Liu and Shen, 2004). The equations consider the effects of surface ice on the flow of water (Shen and Chen, 1992), as well as seepage flow in the void spaces of a porous ice cover (Shen et al., 2000a). A modified explicit time integration technique called the leapfrog method is employed which is stable provided the user selects a sufficiently small time step. Treatment of dry nodes is performed using the small imaginary depth method (Malenchak, 2011; Wu, 2007).

In this approach, the user specifies a drying depth and a rewetting depth. If the water depth on a node drops below the drying depth, that node is considered dry and the velocity will be set to zero. A land boundary treatment is applied to the dry nodes to create an internal dynamic boundary; the flow on adjacent nodes is rotated such that the velocity is tangential to the dry nodes. If the depth on a node increases beyond the rewetting depth, the node is considered wet again and the velocity is no longer constrained to zero. Typically the rewetting depth will be specified larger than the drying depth by some amount to avoid constant switching of a node between wet and dry states.

4.2.2 Ice Dynamics

CRISSP2D employs the Discrete Parcel Method (DPM) (Shen et al., 1993) to simulate the transport and dynamic interactions of surface ice. In this method, the surface ice is considered a continuum that can be represented by a sufficiently large number of parcels, each having properties of mass, momentum, and energy. Each parcel comprises a collection of ice pieces with open water areas at the surface. The thickness of the ice covered portion is made up of a solid ice layer and a frazil ice layer, the latter having a porosity specified by the modeler. A conceptual illustration of an ice parcel is shown in Figure 4.1.

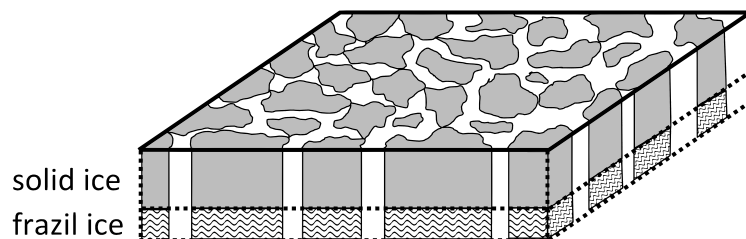


Figure 4.1. Illustration of an ice parcel used in DPM for treatment of ice dynamics in CRISSP2D.

Advective properties, such as ice mass, are tracked for each parcel as it moves through the model domain. The surface concentration, N , and ice thickness, t_i , of each parcel are updated in the ice dynamics portion of the code as parcels move and interact with each other. The surface concentration is limited by a maximum value, N_{max} , specified by the modeler (typically 0.6). Non-advective properties (e.g. strain rate and stress gradient) are calculated using the Smoothed Particle Hydrodynamics (SPH) method, where properties of the tracing parcel (i.e. the parcel whose properties are being calculated) are interpolated from its neighbouring parcels using kernel density estimation. In general, the kernel estimation of a function can be expressed as Equation [4.1] (Liu and Shen, 2011).

$$\tilde{f}(\vec{r}_k, l) = \sum_{j=1}^N f_j \frac{m_j}{M_j} W(\vec{r}_k, \vec{r}_j, l) \quad [4.1]$$

where $\tilde{f}(\vec{r}_k, l)$ is the function estimated at position $\vec{r}_k$ of the tracing parcel, l is the smoothing length which determines the range of influence of the interpolation kernel, W , f_j is the function value at location $\vec{r}_j$ of a neighbouring parcel, and m_j and M_j are the mass and mass density of neighboring parcel j , respectively. The Gaussian kernel is used for the interpolation kernel, W , in the CRISSP2D model. The mass density at each parcel location (mass per unit plan area) is calculated at each time step using the SPH method. Substituting M for the function f in Equation [4.1], the mass density at the location of parcel k can be found using Equation [4.2].

$$M_k = \sum_j m_j W_{kj} \quad [4.2]$$

where W_{kj} is the average value of the interpolation kernels of parcels k and j . Using the calculated mass density and the known mass of ice within a parcel (which remains constant

in the ice dynamics module), the surface concentration is calculated. As parcels converge, the mass density increases and the surface concentration of ice within the parcel increases. When the surface concentration reaches the limit N_{max} , any further increases in mass density result in an increase to the thickness of the parcel (i.e. mechanical thickening). In essence, mechanical thickening of surface ice occurs as parcels become closer together.

The momentum equation of the surface ice can be written in Lagrangian form, shown in Equation [4.3] (Shen et al., 1990).

$$M_i \frac{D\vec{V}_i}{Dt} = \vec{R} + \vec{F}_a + \vec{F}_w + \vec{G} \quad [4.3]$$

where M_i is the mass density, $\frac{D\vec{V}_i}{Dt}$ is the acceleration of the ice parcel, $\vec{V}_i$ is the velocity of the ice parcel, t is time, $\vec{R}$ is the force due to internal ice resistance (stress gradients), $\vec{F}_a$ is the force due to wind shear, $\vec{F}_w$ is the force due to water shear, and $\vec{G}$ is the gravitational force due to the sloped water surface.

The internal ice resistance vector is calculated from the stress gradients as shown in Equations [4.4] and [4.5].

$$R_x = \frac{\partial}{\partial x} (\sigma_{xx} N t_i) + \frac{\partial}{\partial y} (\sigma_{xy} N t_i) \quad [4.4]$$

$$R_y = \frac{\partial}{\partial y} (\sigma_{yy} N t_i) + \frac{\partial}{\partial x} (\sigma_{yx} N t_i) \quad [4.5]$$

where σ_{xx} and σ_{yy} are normal stresses, $\sigma_{xy} = \sigma_{yx}$ are shear stresses, N is the areal concentration, and t_i is the thickness of the parcel. The stresses on the ice parcels are determined with a constitutive law, discussed later in Equation [4.13].

The wind shear at the ice-air interface is calculated with Equations [4.6] and [4.7].

$$F_{ax} = \rho_a c_a |\vec{W}| W_x N \quad [4.6]$$

$$F_{ay} = \rho_a c_a |\vec{W}| W_y N \quad [4.7]$$

where $\vec{W} = W_x \hat{i} + W_y \hat{j}$ is the wind velocity at a 10 m height above the ice surface, ρ_a is the density of air, and c_a is the wind drag coefficient for ice, which incorporates both frictional and form drag effects.

The water shear force is calculated with Equations [4.8] and [4.9].

$$F_{wx} = \rho c_w |\vec{V}_w - \vec{V}_i| (V_{wx} - u) \frac{N}{N_{max}} \quad [4.8]$$

$$F_{wy} = \rho c_w |\vec{V}_w - \vec{V}_i| (V_{wy} - v) \frac{N}{N_{max}} \quad [4.9]$$

where $\vec{V}_i = u \hat{i} + v \hat{j}$ is the ice parcel velocity, $\vec{V}_w = V_{wx} \hat{i} + V_{wy} \hat{j}$ is the depth-averaged water velocity, and c_w is the water drag coefficient which is a function of the Manning roughness of the underside of the ice cover.

Finally, the gravitational force due to the slope of the water surface is calculated with Equations [4.10] and [4.11].

$$G_x = -M_i g \frac{\partial \eta}{\partial x} \quad [4.10]$$

$$G_y = -M_i g \frac{\partial \eta}{\partial y} \quad [4.11]$$

where g is gravitational acceleration and η is the water surface elevation.

If a parcel is close to a land boundary, an additional frictional force will be applied to simulate the resistance provided by the river bank. This frictional force is calculated based on the dynamic Coulomb yield criterion (Hanes and Inman, 1985) with the assumption of zero cohesion, shown in Equation [4.12].

$$F_f = F_N \tan \phi \quad [4.12]$$

where F_f is the friction force and F_N is the normal component of the ice force acting on the boundary. A similar expression is used to model the bed friction of a grounded ice accumulation.

The internal stresses on the ice parcels are calculated with a Kelvin-Voigt viscoelastic constitutive model (Ji et al., 2005) with a Mohr-Coulomb yield criterion. The stress-strain relationship employed by this model in two dimensions is written in tensor notation in Equation [4.13].

$$\sigma_{ij} = 2\eta_v \dot{\epsilon}_{ij} + (\zeta_v - \eta_v) \dot{\epsilon}_{kk} \delta_{ij} + 2G \epsilon_{ij} + (K - G) \epsilon_{kk} \delta_{ij} - P_r \delta_{ij} \quad [4.13]$$

where η_v and ζ_v are the shear and bulk viscosities, G and K are the elastic shear and bulk moduli, ϵ and $\dot{\epsilon}$ are the strain and strain rate, and P_r is the horizontal hydrostatic pressure. The first two terms on the right side of Equation [4.13] constitute the viscous effects; stresses are dependent on the strain rates and viscous material properties. The next two terms on the right side of Equation [4.13] constitute the elastic effects; stresses are dependent on the elastic material properties and total strains.

The model uses the strain and strain rates as calculated with the SPH method to determine the stress state of the parcel. The strain rates within the ice continuum at the

location of a given parcel k are estimated with the SPH method, shown in Equations [4.14] to [4.17].

$$\left(\frac{\partial u}{\partial x}\right)_k = \frac{1}{M_k} \sum_j m_j (u_j - u_k) \frac{\partial W_{kj}}{\partial x} \quad [4.14]$$

$$\left(\frac{\partial u}{\partial y}\right)_k = \frac{1}{M_k} \sum_j m_j (u_j - u_k) \frac{\partial W_{kj}}{\partial y} \quad [4.15]$$

$$\left(\frac{\partial v}{\partial x}\right)_k = \frac{1}{M_k} \sum_j m_j (v_j - v_k) \frac{\partial W_{kj}}{\partial x} \quad [4.16]$$

$$\left(\frac{\partial v}{\partial y}\right)_k = \frac{1}{M_k} \sum_j m_j (v_j - v_k) \frac{\partial W_{kj}}{\partial y} \quad [4.17]$$

where u and v are the parcel velocities in the x and y directions, respectively, m and M are the mass and mass density of the parcel, and $\frac{\partial W_{kj}}{\partial x}$ and $\frac{\partial W_{kj}}{\partial y}$ are the average values of the derivative of the interpolation kernels of parcels k and j with respect to x and y , respectively. The strain rate is multiplied by the time increment to determine the additional total strain on an ice parcel over the time step. The strain rates and strain can then be used in the constitutive model to determine the new internal stress on each parcel (considering plastic behaviour if applicable). The SPH method is used again to calculate the internal ice resistance force vector on each parcel. The momentum equation is then used to calculate the new parcel velocity. The parcel is shifted within the model domain to a new position and the mass density is recalculated with the SPH method.

The Mohr-Coulomb criterion is used to determine if an ice parcel, in a given stress state, will be viscoelastic or plastic. Ice cohesion is not considered in the model. The average vertical buoyant stress, P_0 , is assumed to be the principal stress in the vertical

direction, σ_3 , and acts as the confining pressure on the ice cover. This follows the theory of Pariset et al. (1966) and Uzuner and Kennedy (1976) when σ_3 is the least principle stress, which is most often the case. The yield surface is composed of three sections: shear yielding, divergent yielding, and convergent yielding. The yield surface in two-dimensional principal stress space is shown in Figure 4.2. The divergent stress, σ_D , and convergent stress, σ_C , are also shown with their respective equations.

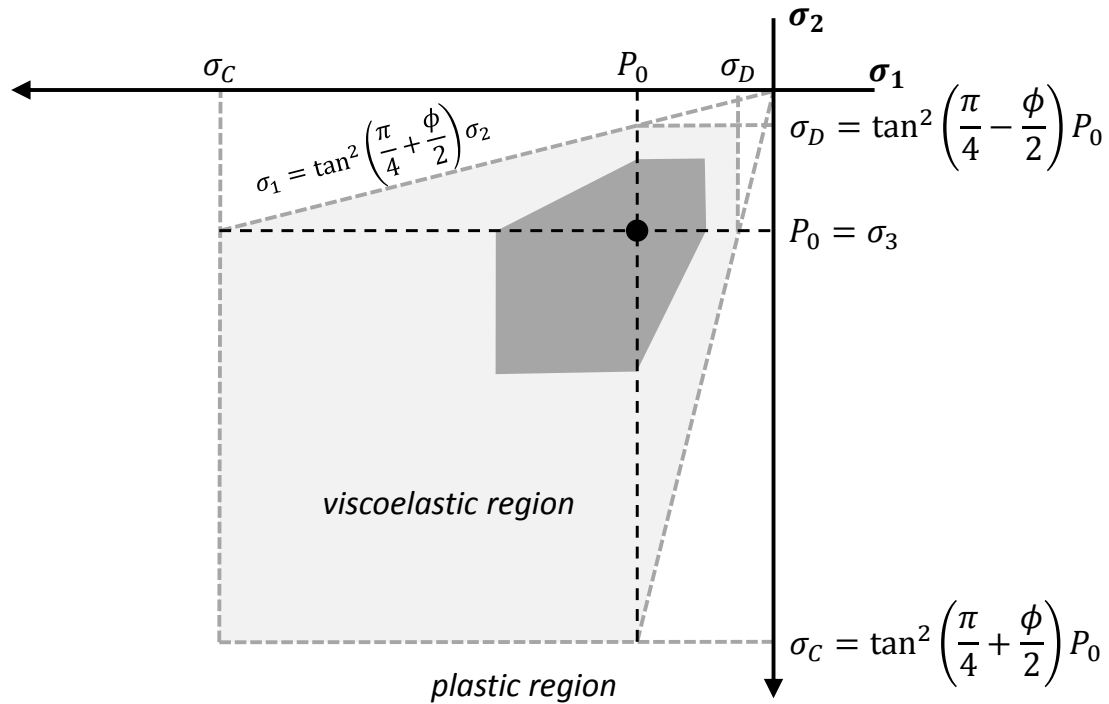


Figure 4.2. Mohr-Coulomb yield surface in two-dimensional principal stress space (adapted from Liu and Shen, 2011). The smaller dark shaded region indicates the yield surface defined by a lower value of internal friction angle, ϕ .

When the stress state of the parcel falls within the bounds of the yield surface, the entire strain of the parcel over the time step is considered elastic deformation of the ice cover, and the stresses are calculated from the strains and strain rates in the viscoelastic constitutive law. When the stress state falls outside the yield surface (the plastic region),

the total strain of the parcel over the time step is divided into two portions: elastic strain and plastic strain. Only the elastic strain is used to calculate the incremental change in internal stress over the time step with the viscoelastic equation. The plastic strain essentially allows ice parcels to become closer to neighbouring parcels (thereby causing mechanical thickening) without any change to the internal stress.

In order for an ice parcel to become stationary under a set of external forces (water shear, weight, wind shear), a stress gradient must be developed with the ice cover. However, in order for an ice parcel to generate internal stresses, it must be adequately thick to exist within the Mohr-Coulomb yield boundary. Thus, greater external forces will result in the formation of a thicker ice accumulation.

The user has the ability to change the material properties of the ice cover which will modify the constitutive equations and change the relationship between stresses and strains for the ice. Most commonly, the angle of internal friction, ϕ , is modified to adjust the Mohr-Coulomb yield surface. In Figure 4.2, the effect of decreasing ϕ is shown by the darkly shaded region which indicates the suppressed viscoelastic region. Therefore, for an ice parcel with a given σ_3 (related to thickness), a higher value of ϕ will allow the ice cover to develop the required stresses to become stable at lower values of ice thickness, resulting in the formation of a thinner ice cover.

Current state-of-the-art models (including CRISSP2D) rely on the modeler to select a single set of parameters that are used to calculate ice dynamics throughout the entire simulation period. There is a missing link between the thermal growth of a surface crust in an ice accumulation and the associated time-dependent ability to resist mechanical

thickening. The interdependency of these processes is important to consider during the formation of an ice cover at freeze-up.

4.2.3 Heat Exchange

CRISSP2D is capable of simulating the thermodynamics of river ice processes by considering energy exchanges between the atmosphere, ice cover, and water. Energy exchanged between the riverbed and water are not included. The heat exchange at the air-water or air-ice interface due to meteorological conditions is an important consideration for modeling river ice. The model considers this heat exchange in one of three ways. First, the modeler can choose to use the linear heat transfer approach with the consideration of air temperature only (see Equation [1.2]). This is a common option due to the simplicity of the calculations and the need for only air temperature as an input. Satisfactory results can be obtained with a site-calibrated heat transfer coefficient over long-term simulations. The second option is to apply the linear heat transfer approach with an additional term which incorporates the diurnal effect of solar radiation, as shown in Equation [4.18].

$$\phi_{wa} = H_{wa}(T_s - T_a) - \phi_R \quad [4.18]$$

where ϕ_{wa} is the heat flux at the water-air interface. An additional term, χ , can be added to Equation [4.18] to account for other heat exchange processes not captured in the first two terms. A similar expression is used for the heat flux at the ice-air interface. This approach, when used with an appropriate time step, accounts for diurnal variation in heat exchange due to solar radiation. This can be important for some ice processes such as the growth and release cycles of anchor ice, or the growth and decay of the surface crust on an ice cover. The third option is to apply the full energy balance approach, considering the individual effects of solar radiation, longwave radiation, evaporative heat transfer,

precipitation, and conductive heat transfer (see Equation [1.1]). This approach relies on several empirical relationships to quantify the effects of each term (Liu and Shen, 2011). The major disadvantage to this approach is the additional meteorological data requirement, thus the form of Equation [4.18] is most commonly used.

Heat exchange at the bottom surface of the ice cover is considered in the model using a linear approach as shown in Equation [4.19].

$$\phi_{wi} = H_{wi}(T_w - T_m) \quad [4.19]$$

where ϕ_{wi} is the heat flux between the river water and ice and H_{wi} is the linear heat transfer coefficient at the water-ice interface. The CRISSP2D program calculates a heat transfer coefficient based on the Reynolds number of the flow and the temperature of the river water. The value of H_{wi} is much larger than H_{wa} , and varies depending on the roughness of the underside of the ice cover. For smooth ice covers, H_{wi} may be approximately 2,000 W/m²/°C, while for rough covers with ice blocks protruding into the flow, H_{wi} can be in excess of 10,000 W/m²/°C (Ashton, 2013; Beltaos and Burrell, 2006). However, note that the temperature difference is much smaller than at the air-ice interface. The heat flux at the water-ice interface will first affect the bottom layer of the ice parcel. In the case of a frozen parcel with a surface crust layer and rubble ice layer, ϕ_{wi} will increase or decrease the thickness of the rubble ice layer before affecting the crust layer. Thus, heat exchange processes at the air-ice interface are more important for the formation and growth of the surface crust.

4.2.4 Water Temperature and Suspended Frazil Ice

Calculation of water temperature and suspended frazil ice concentration is performed using an equation describing the transport of thermal energy in a mixture of ice and water (Liu and Shen, 2011). The full equation includes advection and diffusion of thermal energy, and source and sink terms encompassing heat exchange at the water surface and river bed, heat exchange between suspended frazil ice and river water, and mass exchanges between the suspended and surface ice layers. The model breaks the computation into two separate stages. First, in the water temperature module, the conservation equations of water temperature and suspended frazil concentration are solved using the Galerkin finite element method considering advection, diffusion, and (for water temperature) heat exchange at the top and bottom boundaries (Liu and Shen, 2011). In the second stage, the model aims to return the water column to a state of thermal equilibrium by solving for the change in suspended frazil ice concentration due to thermal growth of frazil particles with a Lagrangian finite difference approach. If the water temperature is below 0°C (supercooled), the suspended frazil concentration increases such that the latent heat released brings the water temperature closer to 0°C. Conversely, if the water temperature is above 0°C, the suspended frazil ice concentration will decrease. Mass exchange between the suspended and surface ice layers are also considered in this stage. The user can specify parameters such as frazil particle rise velocity and probability of deposition/entrainment to control this process. The model assumes that the size of a frazil particle is constant (user-specified) through the simulation. Thus, thermal growth of suspended ice simply increases the volumetric concentration of the frazil, rather than changing the size of the particles themselves (Malenchak, 2011).

4.2.5 Skim and Border Ice

The skim ice formation criteria used within the CRISSP2D model are adapted from those developed by Matousek (1984), which state that skim ice can form when the following conditions are met:

- (i) the temperature of the water surface is supercooled (i.e. $< 0^{\circ}\text{C}$); and,
- (ii) the buoyant velocity of the ice particle exceeds the vertical component of the turbulence velocity.

Since the water temperature module considers the depth-averaged temperature of the water column, an empirical relationship developed by Matousek (1984) is used to estimate the water surface temperature. The relationship considers the depth-averaged water temperature, heat flux at the air-water interface, depth-averaged water velocity, and a wind velocity term. The vertical turbulence velocity is calculated using the expression proposed by Lal and Shen (1989) which considers effects of wind and bed shear stress. If the above criteria are met on an element, an ice parcel is created with an area equal to the element area, a surface concentration of 1.0, and a thickness specified by the user (typically 0.001 m). Alternatively, if ice parcels exist in the element, and their areal concentration is less than 1.0, the mass of skim ice grown will be distributed to the parcels in that element and their areal concentrations will be set to 1.0. If a skim ice parcel is transported to a portion of the river where the turbulence is high, or is subjected to increased internal stress from other parcels or a land boundary, the maximum areal concentration of the parcel will be reset to that of a normal surface ice parcel (typically 0.6). This represents the fragmenting of a weak skim ice cover in dynamic conditions (Liu and Shen, 2011).

The static border ice formation routine functions in a similar way; if the thermal and mechanical conditions for static border ice growth are met on a node adjacent to a land boundary or existing border ice node, that node will instantaneously develop a border ice cover. The conditions checked within CRISP2D for static border ice formation were originally proposed by Matousek (1984), as follows:

- (iii) the temperature of the water surface is less than a critical value, as specified by the modeler;
- (iv) the buoyant velocity of the ice particle exceeds the vertical component of the turbulence velocity; and,
- (v) the depth-averaged velocity is less than the critical velocity, as specified by the modeler.

If the above conditions are satisfied, border ice will be created on the node with a user-defined initial thickness and a surface concentration of 1.0. Accretion of surface ice on the edge of existing border ice is known as “buttering”, and is incorporated into the model via the dimensionless relationship developed by Michel et al. (1980) which includes a critical water velocity that limits when a surface ice parcel can adhere to existing border ice (Liu and Shen, 2011).

4.2.6 Mass Exchange between Suspended and Surface Layers

Suspended frazil ice can rise to the water surface to create new ice parcels, or add to the thickness of a frazil ice layer in existing parcels. Conversely, frazil ice from the surface layer can be entrained in the flow, adding to the mass in the suspended layer. Mass exchange processes between the suspended frazil ice and the surface layer are controlled

by two user-defined parameters. These parameters specify the probability of deposition of ice particles reaching the surface layer from the suspended layer, and the coefficient of re-entrainment of surface frazil ice per unit area. A similar mass exchange process is considered between the suspended layer and the channel bed, simulating the accretion of frazil ice to create anchor ice. In highly turbulent sections of the river, large surface floes may break apart and become entrained in the flow. This process is handled in the model by allowing the user to designate sections of the river as “rapids” and specify a parameter to quantify the fraction of ice parcel mass that is entrained over each coupling time. Both the frazil and solid portion of the surface ice parcel can be entrained, and the entrained particles will have a separate class of mass exchange parameters as defined by the user. The total change to the volumetric concentration of suspended ice due to the mass exchange processes is combined with the finite element advection and diffusion equations to obtain a complete solution for the suspended ice.

4.2.7 Thermal Growth and Decay of Surface Ice

Heat transfer from ice to the atmosphere and river water will cause thermal growth or decay of surface ice parcels. The model treats the thermal growth of moving ice floes and stationary ice covers in the same way. Depending on the water and air temperatures, an ice cover can melt from the bottom, freeze from the bottom, melt from the top, or any combination of the three. The model also accounts for the accelerated growth of cover thickness due to the presence of a frazil ice layer beneath the cover. Growth is accelerated because of the insulating effect of the frazil ice to the solid cover and due to the fact that only the interstitial water between frazil particles must freeze in order for the solid ice layer to grow downwards.

4.2.8 Ice Cover Breakup

The purpose of this module is to simulate the timing of mechanical breakup of a stable ice cover. A river can experience a mechanical breakup when the driving forces that promote breakup increase while the resisting forces remain relatively constant. This can occur with a sudden increase in discharge due to spring snowmelt or from peaking operation of a hydroelectric generating station. Empirical predictive methods are used within the model because of the difficulty in formulating an analytical solution for predicting mechanical breakup. In the model, an ice cover can break up in five different ways. The first is calculated automatically within the model, where a breakup is initiated if the user-defined compressive strength of the ice cover is exceeded by the external compressive forces acting on the parcels that make up the cover. The remaining four options provide the modeler more control over how and when an ice cover will break up, and include the following:

- (i) breakup of ice cover at a user-specified time;
- (ii) breakup of ice cover if water surface elevation exceeds a critical value specified by the user;
- (iii) breakup of ice cover if change in water surface elevation over specified time interval exceeds a critical value specified by the user; and,
- (iv) breakup of ice cover if change in water surface elevation over specified time interval exceeds a critical value related to the thickness of the cover.

Any combination of the above breakup options can be applied to individual sub-reaches within the model domain. In addition to this, the user is able to link sub-reaches such that the breakup of one reach can automatically trigger the breakup of another reach.

4.2.9 Undercover Ice Transport and Accumulation

The progression of an ice cover will continue until the cover reaches a section of river with hydraulic conditions that preclude further progression. The critical Froude number approach is adopted in CRISSP2D. If the Froude number at the leading edge of an ice cover exceeds the critical value as specified by the modeler (typically between 0.08 and 0.14), incoming ice parcels will overturn. It is assumed that overturning ice floes are accumulations of frazil particles and will break up as they entrain in the flow (Liu and Shen, 2011). The newly entrained frazil particles are able to deposit under the ice cover in addition to the suspended frazil particles already in the flow. The movement of frazil particles under an ice cover is driven by the flow and resisted by the buoyant and interparticle forces. In the present model, the ice transport capacity concept (Shen and Wang, 1995) is used to calculate the accumulation of frazil ice particles under an ice cover, allowing for the simulation of hanging ice dams.

4.3 Incorporation of Thermal Effects on Ice Dynamics

4.3.1 Overview

As discussed in Section 1.2.4 and Chapter 3, the growth of a solid ice crust through a porous layer of ice floes comprising an ice cover adds stability and strength to the cover (Michel, 1991; Shen et al., 1995). The non-frictional resistance provided by the solid crust will cause the ice parcels to deviate from the Mohr-Coulomb yield behaviour; the ability

of the ice accumulation to develop internal stresses will no longer be limited by the confining stress caused by buoyancy. The crust will allow greater internal stresses to develop within the cover without significant movement and mechanical thickening of the ice parcels.

This has previously been incorporated into CRISSP2D model applications by artificially stopping ice parcels when they meet some “stoppage criteria”, typically defined as a critical ice parcel velocity. Artificially stopping ice parcels (setting their velocity to zero) will stop them from mechanically thickening, as the mass density of the parcels will not change thereafter. However, stresses and strains will no longer be captured properly with the constitutive model. Furthermore, there should be some stress limit at which the frozen portion of the ice cover fails and mechanical thickening can resume; this stress limit should be related to the amount of freezing that has occurred to the ice cover. The breakup subroutines within the CRISSP2D code only allow the user to input a single value for the strength of the ice cover (e.g. 5000 N/m). In reality, the maximum internal stress a parcel is able to generate will be limited by the thickness of the solid crust layer in addition to the rubble ice layer. If the stress exceeds this limit, the ice cover will collapse and the solid crust will fracture and behave as discrete ice fragments again. The ice parcels will return to the traditional Mohr-Coulomb behaviour and will mechanically thicken to achieve a new stable configuration. Once stable (i.e. ice velocity and rate of thickening are low), the solid crust will grow and begin adding strength again.

The following sections will outline a revised formulation to incorporate the effects of freezing on ice dynamics in the CRISSP2D model, which is a direct contribution of this research. The revised model will enhance the capabilities of the CRISSP2D model, and

provide a more physically based treatment of the ability of a forming ice cover to freeze in place versus collapse and mechanically thicken during the freeze-up period.

4.3.2 Model Formulations

The link between thermal and dynamic processes are incorporated into the CRISSP2D model by distinguishing different types of ice parcels, depending on their stage of evolution from free-drifting ice floes to a jammed accumulation of ice rubble. The three parcel classifications are shown in Figure 4.3.

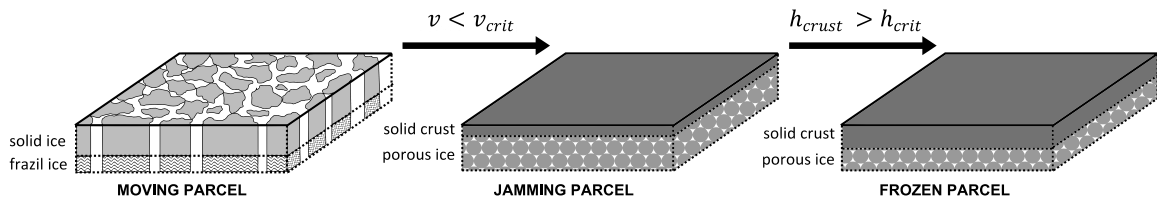


Figure 4.3. Parcel classification to incorporate thermal effects on ice dynamics.

When ice parcels are moving in the area upstream of the ice front, they are nearly free-drifting; some open water areas exist between ice floes and internal stresses are low. These “moving parcels” have two layers: the frazil ice layer grows as frazil is transferred from the suspended layer to the underside of the parcel, while the solid ice layer grows as the interstitial water between frazil particles freezes. Note that there are still open water areas between the surface ice floes.

As the parcels approach the ice front, their velocity will decrease while the internal stresses increase. If the parcel velocity drops below a user-defined critical value (i.e. $v < v_{crit}$), the parcel transitions to a jamming parcel. At this point, the ice parcel represents ice floes that are packed together to create a porous ice layer with a porosity specified by the modeler. Since the “jamming parcel” is moving very slowly, it will be subject to the

growth of a solid crust layer as the water in the void spaces freezes the ice floes together. Moving parcels and jamming parcels follow the viscoelastic-plastic constitutive model with Mohr-Coulomb failure criteria as previously described.

The calculation of the incremental growth (or melt) of solid crust thickness is performed each coupling interval (Δt_{intvl}) using a finite difference approach. If the direction of heat transfer is from the river to the atmosphere (i.e. heat lost from the surface of the ice cover), the crust will freeze downwards through the porous layer below it, as calculated with Equation [4.20]. Note the presence of the ice rubble porosity, p_j , in the denominator of the right side of the equation. As noted in Chapter 3, at early stages of crust development (when the crust thickness is less than the size of a void), the growth rate of the crust would be better represented by the growth rate of a level ice sheet through open water. The growth rate of the crust will only approach the heightened rate after some duration of time, and still the crust thickness would represent a spatially averaged thickness. This detail is omitted in the current formulation for surface crust growth.

$$\Delta h_{crust} = \frac{1}{p_j \rho_i L_i} \left(\frac{H_{ia}(T_m - T_a) - \phi_R}{\frac{H_{ia} h_{crust}}{k_i} + 1} \right) \Delta t_{intvl} \quad [4.20]$$

Conversely, if the direction of heat transfer is such that the river is gaining heat from the atmosphere, melting will occur at the surface and the crust thickness will decrease, as calculated with Equation [4.21]. Note that the rubble porosity is not included in Equation [4.21] because the entire surface of the cover is modeled to melt evenly (i.e. rubble pieces and crust will melt together). This may be an oversimplification, and may not fully capture the deterioration of crust thickness (and associated resistance) that occurs

in a scenario where the ice cover is gaining heat from the atmosphere, such as a sunny day with mild air temperatures only slightly below 0°C.

$$\Delta h_{crust} = \left(\frac{H_{ia}(T_m - T_a) - \phi_R}{\rho_i L_i} \right) \Delta t_{intvl} \quad [4.21]$$

When the solid crust layer thickness exceeds a user-defined value (i.e. $h_{crust} > h_{crit}$), the parcel becomes a “frozen parcel.” At this point, it is assumed that the solid crust has grown sufficiently to restrict the ice rubble from thickening (rearranging) any further. The resistance of the ice parcel to consolidation is the summation of the rubble strength and solid crust strength, shown in [4.22].

$$R_{crit} = \sigma_{crust} h_{crust} + \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \sigma_y t_i \quad [4.22]$$

where σ_{crust} is the failure stress of the solid crust, h_{crust} is the thickness of the solid crust layer, σ_y is the average buoyant stress on the ice cover, and t_i is the thickness of the rubble layer. The value of σ_{crust} was estimated to be approximately 1.2 MPa by Andres (1999) from observations of consolidating and juxtaposing ice covers on the Peace River, Alberta. Laboratory experiments in Chapter 3 found values ranging from 0.2-1.5 MPa, with thicker crust layers exhibiting higher strength values. It is clear from Equation [4.22] that the first term on the right side of the equation, representing the contribution of the solid crust layer, dominates the total consolidation resistance. For example, using typical values of $\phi = 46^\circ$ and $p_j = 0.4$, an accumulation of ice rubble 0.5 m thick will have a resistance of 340 N/m. Assuming a crust strength of 0.5 MPa, a solid crust of only 1 mm would provide an additional 500 N/m of resistance to mechanical thickening.

The frozen parcel will remain in the elastic region until the internal force, F , exceeds the consolidation resistance (Equation [4.22]) of the ice cover. In this way, small relative movement of ice parcels result in increased internal stresses; thus the cover can develop the required stress gradient to remain effectively stationary. The maximum internal force per unit width of ice cover is calculated with Equation [4.23].

$$F = \sigma_1 t_i \quad [4.23]$$

where σ_1 is the maximum principal stress. If $F > R_{crit}$, the parcel will return to the “moving parcel” state, and the mass of ice in the solid crust layer will be added to the porous layer. This physically represents the applied stresses exceeding the internal strength of the ice cover, leading to a consolidation event. With the sudden absence of the solid ice crust, the parcel will undergo plastic deformation and sudden mechanical thickening. When the consolidated parcels reach a state of quasi-equilibrium and the velocity drops below the critical velocity, the process repeats itself.

For illustrative purposes, consider an ice parcel moving downstream toward a stable ice cover, as shown in Figure 4.4. Three stages of evolution of the highlighted parcel are shown: Stage A, B, and C.

Stage A: When the parcel is moving with the flow upstream, the surface ice concentration is low and the ice thickness will be the user-specified single-layer ice thickness. The strength of the parcel will be constant and low. As the parcel is nearly free-drifting, the internal force of the parcel will be only from lateral hydrostatic pressure.

Stage B: As the parcel approaches the stable ice cover, it converges with other parcels. The increase in mass density causes the rubble ice thickness to increase (with

attendant decrease in plan size of the parcel to conserve ice mass), and the velocity of the parcel decreases. The strength of the ice parcel increases with the increase in rubble thickness. Meanwhile, the internal force of the ice parcel increases. The external forces of water shear and weight are pushing the parcel downstream, but now there is a velocity gradient (i.e. strain over each time step), which increases the internal stress on the parcel via the viscoelastic constitutive law. If the parcel velocity decreases below the critical velocity, the parcel will change to a “jamming parcel” and the model will begin to track the growth of the surface ice crust.

Stage C: When the solid ice crust thickness exceeds the critical value, the parcel will change to a “frozen parcel”. The total strength of the parcel is the summation of the rubble strength and the crust strength. The rigid solid crust will prevent further mechanical thickening of the rubble accumulation, so the rubble strength will remain constant. The total strength will continue to increase as the solid crust grows thicker. The internal force of the parcel will continue to increase as more ice is added to the leading edge and the cover lengthens. Eventually the internal force will plateau due to frictional effects of the channel banks.

If the resistance of the ice parcel to consolidation is ever exceeded by the internal force, the solid crust will fail, and the strength of the parcel will decrease suddenly. The parcel will shove downstream and mechanically thicken. When the parcels approach a state of equilibrium and their velocity decreases, the solid crust is permitted to grow again.

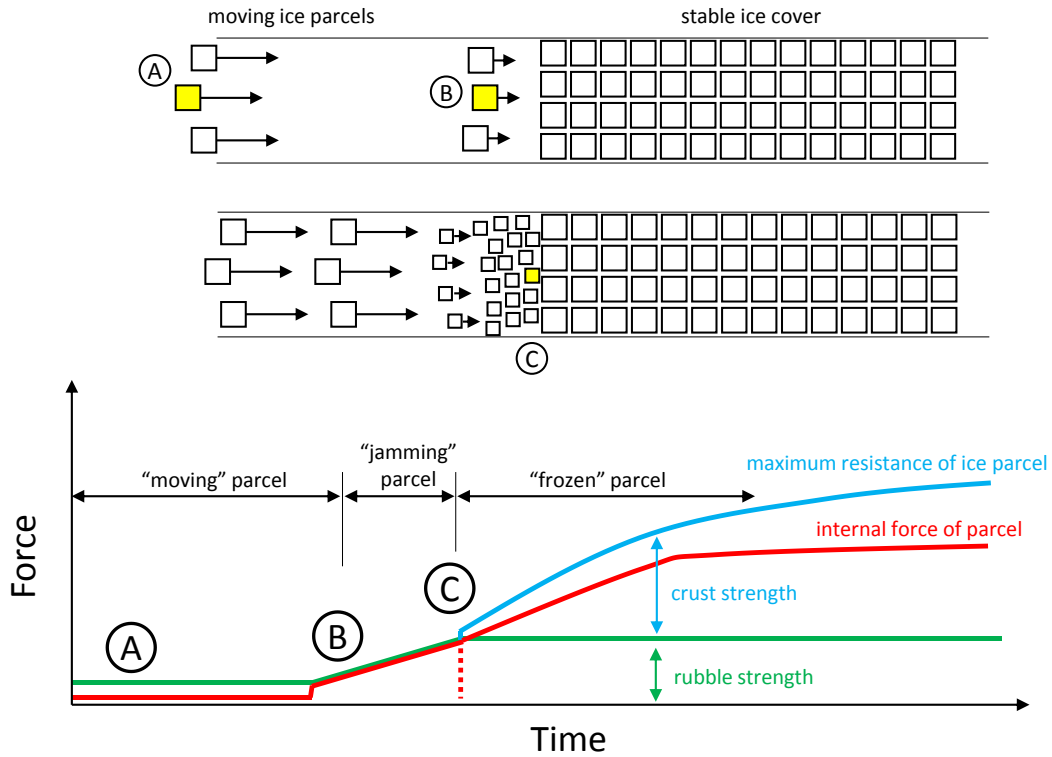


Figure 4.4. Idealized evolution of parcel stresses in a forming ice cover. Top panel shows plan view of the parcel of interest at different times. Bottom panel shows the variation of parcel strength and internal force with time.

There are several scenarios that can promote consolidation of a forming ice cover, illustrated in Figure 4.5. First, if the cover is forming in a reach with a high slope, the rate of stress increase on the parcels at the leading edge will be higher than the rate of strength addition by the solid crust. The cover undergoes multiple consolidations until the backwater effect of the thickened ice accumulation is sufficient to reduce the rate of stress increase caused by the lengthening of the ice cover. Second, if a period of cold weather is followed closely by a period of warm weather, the rate of strength addition from the solid crust decreases while the rate of stress increase from the lengthening cover remains high (as incoming surface ice generated during the cold weather makes its way to the leading

edge). Finally, if hydraulic conditions change due to a storage release, the stresses could increase beyond the strength of the cover and cause a consolidation.

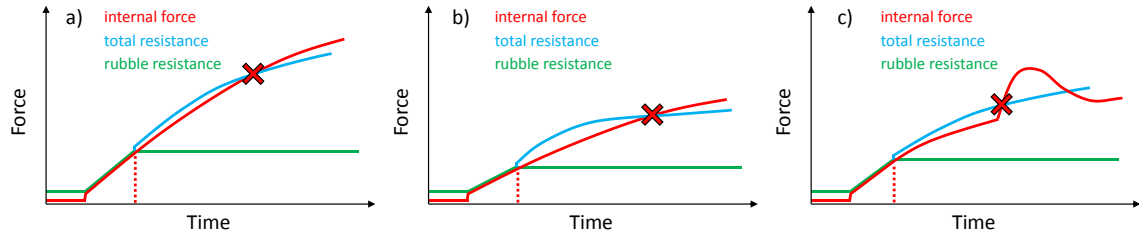


Figure 4.5. Scenarios that promote ice cover consolidation: a) high bed slope, b) mild weather, c) flow increase.

4.3.3 Assumptions of the Proposed Treatment

A number of assumptions were made in the revised model formulations, listed below.

- (i) The solid crust grows uniformly at the surface of the ice cover at a rate influenced by the porosity of the ice accumulation (which is prescribed by the user). The surface freezing calculation is simplified and does not consider processes such as freeze-bonding between rubble pieces that can occur below the depth of the solid crust.
- (ii) The solid crust will begin to grow when the parcel velocity meets the prescribed criteria, and will continue to grow until the stress state of the parcel results in failure. In reality, there are numerous factors that could influence the ability of the surface crust to grow (and remain intact). Relative motion of rubble ice, even in slowly moving areas of the cover, could prohibit the growth of a uniform crust, particularly early in the cover formation. Water velocity through the voids within a rubble accumulation

may also play a role in the development of the crust, but is not considered in the model.

- (iii) Frozen parcels, which include a surface crust with underlying rubble, are treated as a single unit. Resistance to consolidation of the rubble and crust are additive, and representative of the ability of the parcel to withstand applied stresses or fail. In other words, the rubble ice under an intact ice cover will not thicken; the crust must fail first, then the entire parcel will mechanically thicken as a unit.
- (iv) Only in-plane stresses are considered when determining if the frozen parcels will fail. This assumption has been adopted by previous studies (e.g. Andres, 1999) to characterize the effect of the surface crust on the resistance of the ice cover to mechanical thickening. In reality, bending stresses may play a role near the consolidating front, particularly in cases where ice rubble submerges and piles up beneath a section of the frozen cover and applies a lifting force which induces bending failure (Andres et al., 2005).

These assumptions simplify the complex processes occurring while a river ice cover is forming during freeze-up, resulting in a model that relies on only three additional user inputs. Details of surface freezing are all lumped into the tracking of a solid ice crust, which effectively enables parcels to resist mechanical thickening.

4.3.4 Implementation in CRISSP2D Model

This section will present a high-level outline of how the revised formulations were implemented in the CRISSP2D program. The general order of computations in a CRISSP2D simulation is shown in Figure 4.6. As previously mentioned, the

hydrodynamics and ice dynamics modules are loosely coupled. After model initialization (Step 1), the hydrodynamics are simulated first (Step 2). The hydrodynamic time step, Δt_h (specified by the user) is used, and the calculations are performed on the finite element mesh until the coupling time interval, t_{intvl} , is reached. Prior to the ice dynamics calculations, initial coupling calculations are performed to update the properties of the parcels in the model (Step 3). Among other calculations, new parcels are created, and ice mass is added or removed from parcels based on thermal and mass exchange processes. In Step 4, the transport and interaction of ice parcels are simulated within the model domain, using the latest hydrodynamic result (from Step 2). During this step, the parcels are moved downstream and their thickness, velocity, and internal stresses are computed based on interactions with other parcels in the model. These calculations are performed using the ice dynamic time step, Δt_i . When the coupling time interval is reached, the properties of the ice parcels are interpolated to the nodes of the finite element mesh. Hydrodynamic calculations are then performed over the next coupling time interval, using the latest ice properties from the ice dynamics module. This sequence repeats until the end of the simulation.

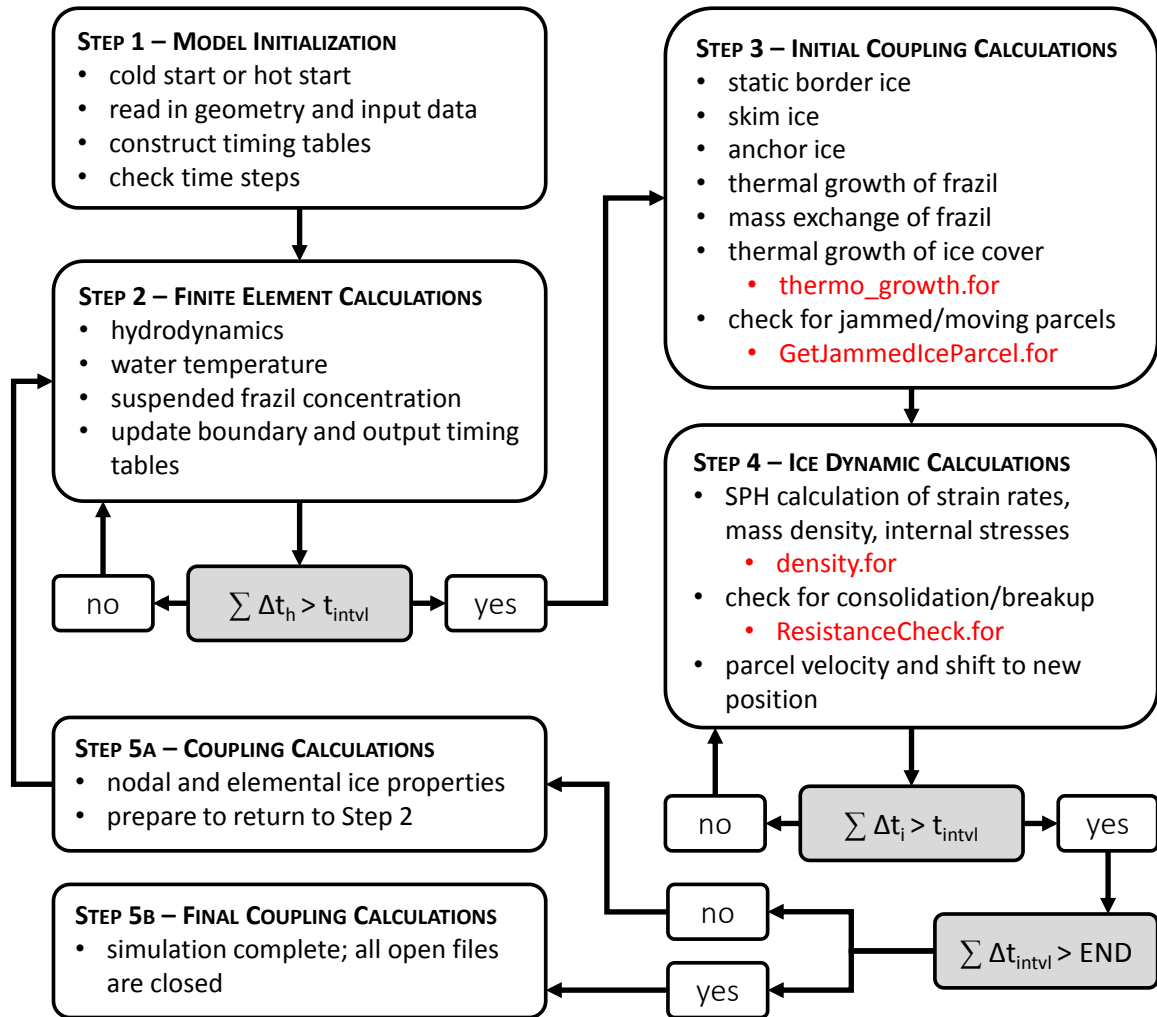


Figure 4.6. CRISSP2D computational procedure.

Within the computational procedure outlined in Figure 4.6, four specific subroutines are highlighted, namely `thermo_growth.for`, `GetJammedIceParcel.for`, `density.for`, and `ResistanceCheck.for`. It is within these subroutines that considerations are made for the growth of the surface crust and the effects on the resistance of ice parcels to mechanical thickening.

Figure 4.7 shows a flowchart of the `thermo_growth.for` subroutine. In this subroutine, the heat exchange at the ice-air and ice-water interfaces is used to calculate the

incremental growth of surface ice in each parcel. If the parcel is a “moving” parcel, the growth/melt of surface ice will modify the solid ice thickness of the parcel (see Figure 4.3). If the parcel is a “jamming” or “frozen” parcel, the growth/melt of surface ice will modify the crust thickness of the parcel instead. Italicised words in Figure 4.8 indicate variable names within the CRISSP2D code, for programming reference.

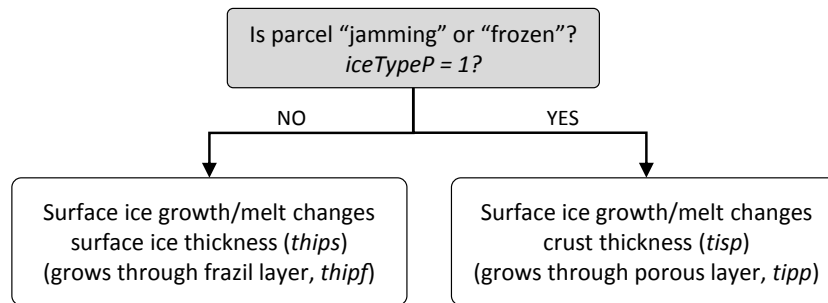


Figure 4.7. Flowchart of thermo_growth.for subroutine.

Shown in Figure 4.8, the GetJammedIceParcel.for subroutine determines the class of each parcel. The subroutine will first check if the velocity of the ice parcel is less than the critical velocity, and if so, classify the parcel as a “jamming” parcel. At this point the surface ice crust will be initialized and subsequently tracked for the parcel. The subroutine also checks if the crust thickness is greater than the critical value, and classify those parcels as “frozen” parcels.

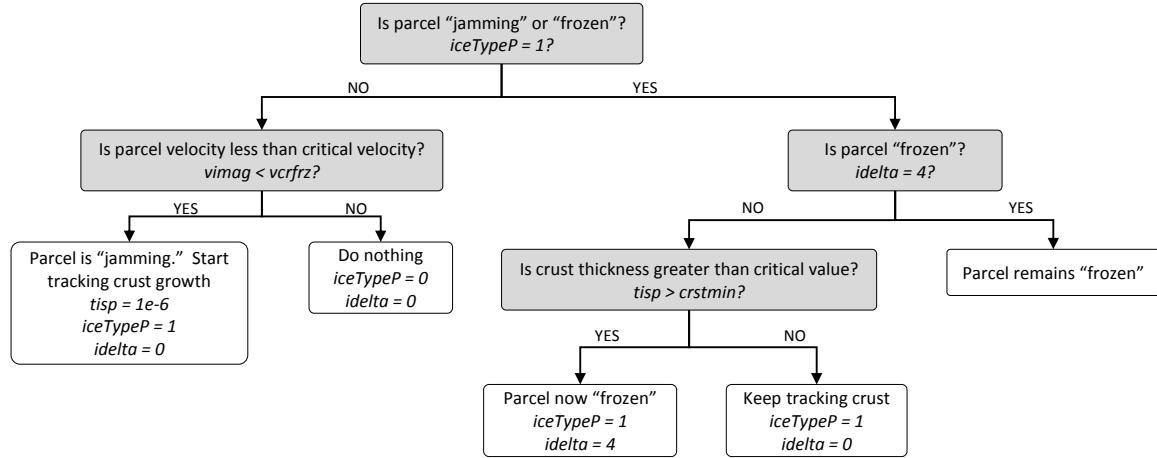


Figure 4.8. Flowchart of GetJammedIceParcel.for subroutine.

The flowchart of the density.for subroutine is shown in Figure 4.9. In this subroutine, the SPH method is used to determine the velocity gradients, strains, and mass density of each parcel. The mass density is subsequently used to calculate the thickness and areal concentration of the parcel. The subroutine then checks if the parcel is “frozen.” If it is not, the viscoelastic constitutive model with Mohr-Coulomb failure criterion for granular materials is used to update the internal stresses on the parcel. If the parcel is “frozen,” the viscoelastic model is used to update the stresses, but failure of the parcel is not checked until the ResistanceCheck.for subroutine.

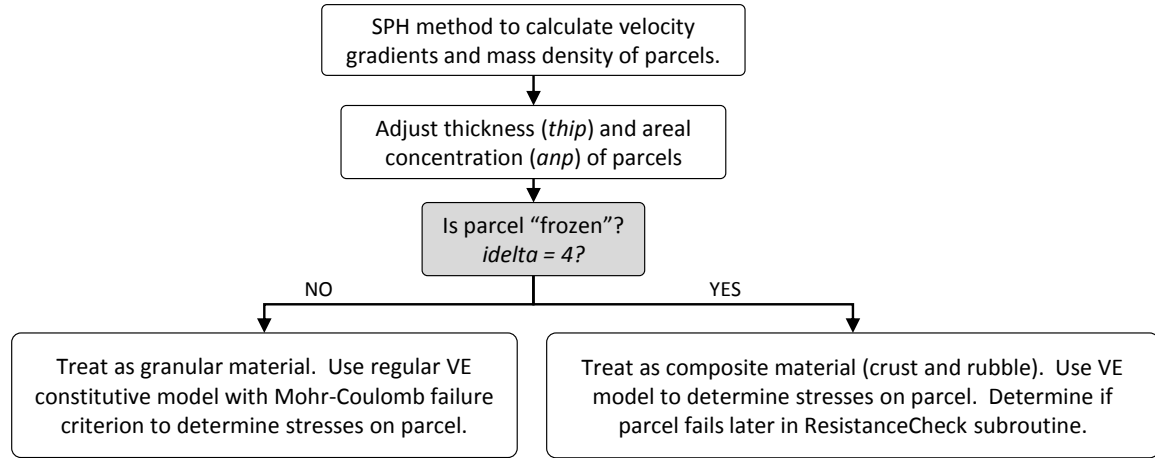


Figure 4.9. Flowchart of density.for subroutine.

The flowchart of the ResistanceCheck.for subroutine is shown in Figure 4.10. In this subroutine, “frozen” parcels fail if the maximum internal force is greater than the strength of the rubble and crust. The failed parcel returns to the “moving” parcel state and the mass of ice in the crust layer is added to the solid ice layer of the moving parcel. The internal forces are set to the lateral hydrostatic pressure.

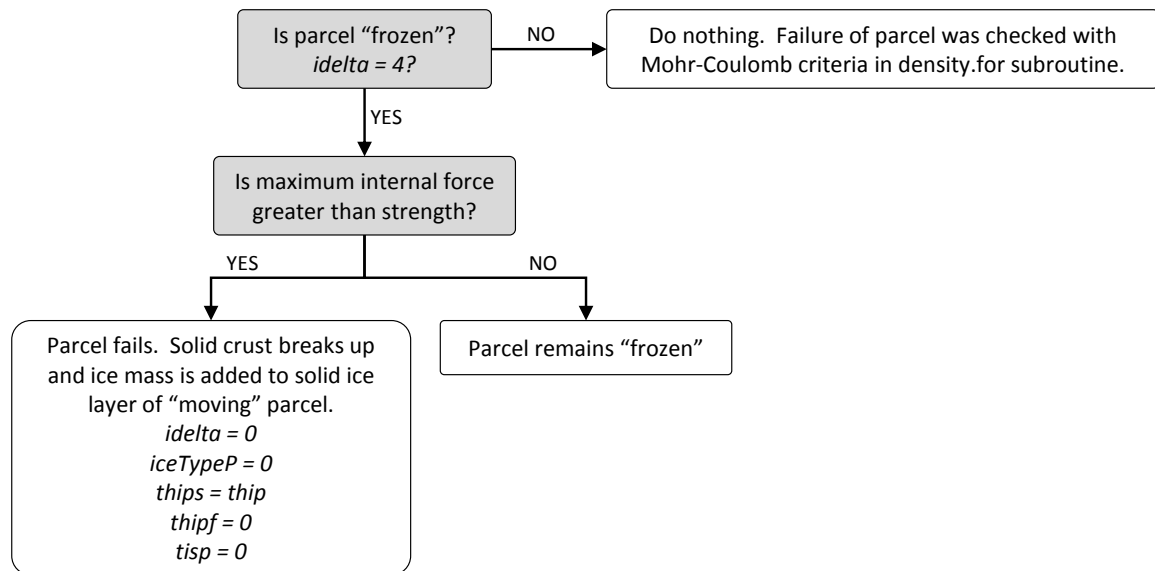


Figure 4.10. Flowchart of ResistanceCheck.for subroutine.

Several parameters were added to the parameter input file (.par file) for use within the modified code. These parameters are listed in Table 4.1, along with a short description of each and a range of recommended values that were found to work well for the model applications in this thesis.

Table 4.1. Parameters added to parameter input file.

Parameter	Description	Recommended Value(s)	Subroutines
vcfrfz [m/s]	Critical parcel velocity to begin crust thickness calculation. When the parcel velocity drops below this value, iceTypeP is set to 1, and the surface crust of the parcel is tracked.	0.001-0.05 m/s	GetJammedIceParcel
ncrsttype [0,1]	Parameter that characterizes how the user wants to input the strength of the surface crust (crst). If ncrsttype = 0, then the model assumes that the strength of the surface crust is a constant value in [N/m]. If ncrsttype = 1, the model assumes the strength of the surface crust is input as a failure stress in [Pa].	1	ResistanceCheck
crst [Pa] or [N/m]	Strength of surface crust. Can either be input as a constant [N/m] if ncrsttype = 0, or as a failure stress in [Pa] if ncrsttype = 1.	0.1-0.5 MPa	ResistanceCheck
crstmin [m]	Minimum crust thickness to classify parcel as “frozen.”	0.001-0.05 m	GetJammedIceParcel
nez [0,1]	Parameter that modifies the ice cover roughness. If nez = 0, the default method of determining roughness from single layer thickness, parcel thickness, base Manning n, and maximum Manning n is used. If nez = 1, the best fit line of Nezhikhovskiy (1964) for accumulations of ice floes at freeze-up is used: if $t_i \leq 0.2$ m: $n_i = 0.02$ if $t_i > 0.2$ m: $n_i = 0.0252\ln(t_i) + 0.0706$ $n_{i,max} = 0.15$	1	parvelocity qforce qforce0

Several variables were added to the parcel output file (.hot file), and are listed in Table 4.2. These variables allow the user to track parcels in areas of interest to evaluate their stability and likelihood of failure. For example, the user could introduce a flow peaking operation and evaluate which areas of a river ice cover are most susceptible to failure.

Table 4.2. Variables added to parcel output file.

Variable Name	Description
strength_rub [N/m]	Maximum resistance of rubble ice layer. When parcel transitions to “frozen” parcel, the principal stress on the parcel at the time will be converted to a load in N/m and stored in strength_rub (if strrub = 1).
strength_crust [N/m]	Maximum resistance of crust layer of parcel. This variable is calculated based on the user-defined crust strength (crst input in .par file) and the thickness of the crust (i.e. the crust resistance of a frozen parcel changes with time).
strength_tot [N/m]	Total resistance of the “frozen” parcel; summation of strength_crust and strength_rub. If the maximum internal force exceeds this value, the frozen parcel fails and becomes a moving parcel.
force [N/m]	Maximum internal force (per unit length) felt by the parcel, based on the maximum principal stress.
cfactor [-]	Variable that quantifies how close a frozen parcel is to consolidating. The consolidation factor is calculated as the maximum in internal force divided by the total resistance of the parcel. Values close to 1 indicate the parcel is nearing failure.

4.4 Idealized Channel Simulations

4.4.1 Purpose and Model Setup

Simulations were performed on an idealized channel to assess the ability of the revised model formulations to simulate the thermal strengthening of a forming ice cover under cold air temperatures, and the ability of the model to simulate dynamic consolidation events common to fast-moving rivers as observed in the field. The effects of dynamic ice events on water levels and discharge were also evaluated.

The model domain consisted of a 5 km long straight, rectangular channel. An ice boom was placed 4.5 km from the upstream boundary to initiate the formation of an ice cover. A water discharge of $200 \text{ m}^3/\text{s}$ and depth of 5 m were prescribed as the upstream and downstream boundary conditions, respectively. The channel had a width of 130 m and a slope of 0.16%. The channel properties were set to resemble the Lower Dauphin River, where consolidation events have been observed to affect the final ice thickness and water levels (Wazney et al., 2018). The depth at the downstream boundary created a backwater effect in the model that extended approximately 2.5 km upstream. The open water depth, velocity, and Froude number in the uniform flow region of the model were 1.1 m, 1.4 m/s, and 0.4, respectively. The finite element mesh consisted of 3500 elements and 2008 nodes, which allowed for seven elements to span laterally across the channel as shown in Figure 4.11.

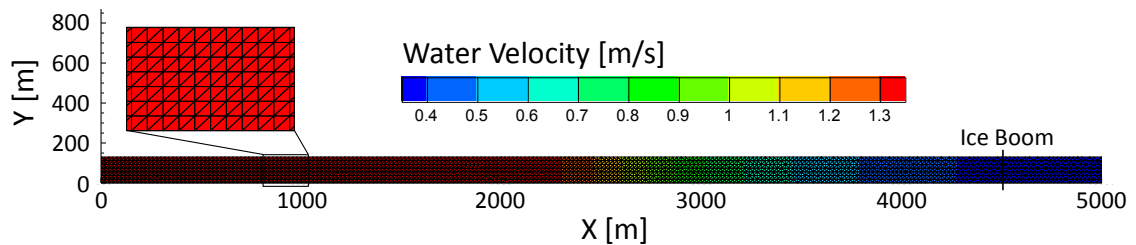


Figure 4.11. Idealized channel model domain.

Parameters relevant to the thermal strengthening and formation of the ice cover that were held constant over all runs are listed in Table 4.3. The values for v_{crit} and h_{crit} were selected based on a best estimate of conditions that could initialize the growth of the solid ice crust and at what point it should begin to influence ice dynamics within the model. The values for ϕ , p_j , and H_{ia} were based on the default settings within the CRISSP2D model

which have been shown to produce satisfactory results in other model applications. The roughness of the ice cover, n_i , was selected to be a constant value near the upper end of the range of values reported by Nezhikhovskiy (1964). The bank friction coefficient was selected to be slightly greater than the friction angle between ice pieces to represent some added resistance provided by vegetation and roughness at the channel bank.

Table 4.3. Relevant parameters held constant for idealized channel runs.

Parameter	Description	Value
v_{crit}	critical freezing velocity	0.05 m/s
h_{crit}	critical crust thickness	2 mm
ϕ	internal friction angle of ice rubble	46°
p_j	porosity of ice rubble	0.4
H_{ia}	linearized heat transfer coefficient at ice-air interface	12.2 W/m ² /°C
n_i	Manning roughness of ice cover	0.10
$\tan \phi_b$	bank friction coefficient	1.5

Five runs were performed to evaluate the effect of air temperature and crust strength on the formation of an ice cover in the prismatic channel. Run 1 was set up with an air temperature of 0°C, thus there was no consideration of thermal effects on ice dynamics. This run can be thought of as the base case, which represents the current state of practice in modeling the formation of a river ice cover at freeze-up in CRISSP2D (i.e. Mohr-Coulomb failure criterion with constant friction angle). In the four other runs, combinations of air temperatures of -10°C and -30°C and crust strengths of 0.5 MPa and 1.0 MPa were prescribed to evaluate the effect on the ice cover formation. For all runs, ice was introduced at the upstream boundary at a constant discharge of 4 m³/s and thickness of 0.2 m. In reality, a given river would have ice discharge correlated to air temperature, but the length of the river would affect the exact relationship. Each simulation was run for

50 hours (model time), or until the ice cover progressed to the upstream boundary. Table 4.4 summarizes the ideal channel run setups.

Table 4.4. Run setups for idealized channel simulations.

	T_{air} [°C]	σ_{crust} [MPa]
Run 1	0	-
Run 2	-10	0.5
Run 3	-10	1.0
Run 4	-30	0.5
Run 5	-30	1.0

4.4.2 Results

4.4.2.1 Water Levels

Water level time series were extracted from each simulation at the 2.0 km and 4.0 km river stations, as shown in Figure 4.12.

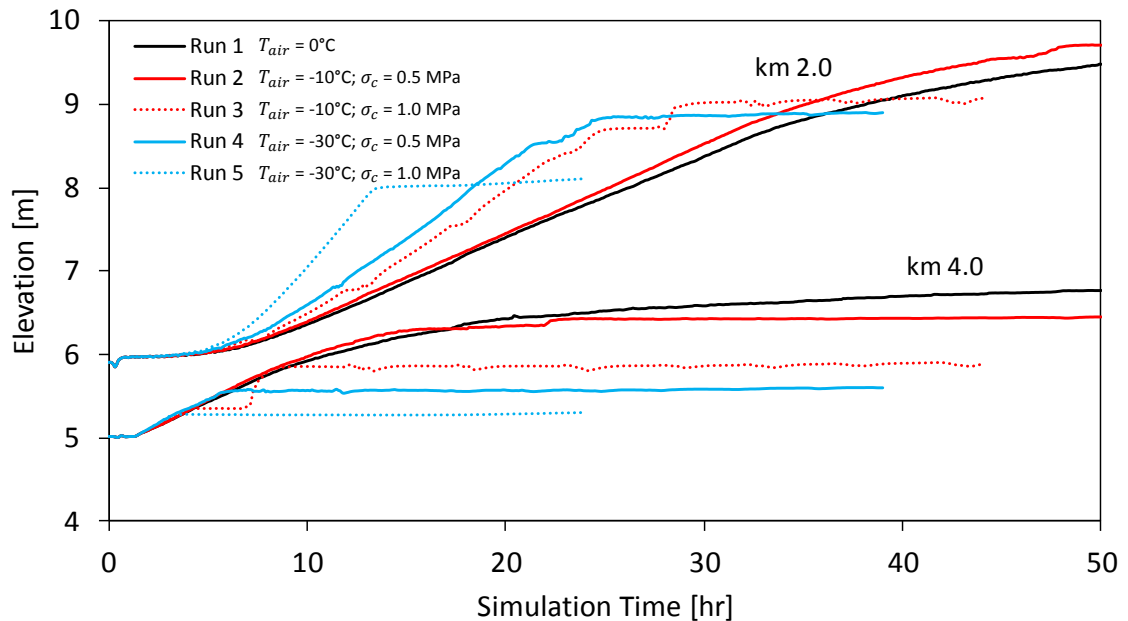


Figure 4.12. Water levels at two locations along channel centreline for ideal channel simulations. The end of each line indicates the time at which the leading edge of the ice cover reached the upstream boundary.

Run 1 was compared to all other runs to evaluate the general effect of incorporating thermal strengthening of the ice cover. In all runs with thermal strengthening (Runs 2-5), water levels at both locations rose earlier and at a higher rate than Run 1. This is because the ice front was able to progress upstream at a higher rate, causing backwater effects earlier in the simulation than in Run 1. The final stable water levels of Runs 2-5 are noticeably lower than Run 1. Thermal strengthening of the ice cover allowed the cover to become stable at a reduced thickness, thus causing less flow constriction and staging. The exception to this is Run 2 at km 2.0; at the end of the simulation the water level at this location was approximately 0.2 m greater than Run 1. This is because the ice cover in Run 1 was still thickening downstream of this location, and the water level would continue to rise and eventually exceed the water level of Run 2, which would begin to plateau as the ice cover stabilizes. This is similar to the water level records at km 4.0; early in the simulation period (hour 10-15), the water level of Run 2 was greater than Run 1. However, as the simulation continued, the ice cover of Run 2 stabilized due to thermal effects. The ice cover of Run 1 continued to thicken as more ice was added to the leading edge and forces continued to increase on the accumulation of rubble.

Table 4.5 summarizes the stable water levels in each of the five runs at km 4.0 and km 2.0. Comparing Run 2 to Run 3, and Run 4 to Run 5, shows the effect of increasing the strength of the solid ice crust. A higher failure stress of the solid crust will effectively decrease the potential for a consolidation event to occur. In both cases, increasing the crust strength from 0.5 MPa to 1.0 MPa resulted in a faster progression of the ice cover upstream (i.e. earlier rise in water levels at km 4.0). The higher crust strength also decreased the final stable water level. For example, at km 4.0, Run 2 obtained a stable water level that

was 0.6 m higher than that of Run 3, and occurred approximately 12 hours later in the simulation. Similarly, Run 4 obtained a stable water level that was 0.3 m greater than Run 5, occurring 3 hours earlier in the simulation.

Table 4.5. Comparison of ice-affected staging in idealized channel simulations.

	T_{air} [°C]	σ_{crust} [MPa]	km 4.0		km 2.0	
			Stable Water Level [m]	Time of Stable Ice Cover [hr]	Stable Water Level [m]	Time of Stable Ice Cover [hr]
Run 1	0	-	>6.8	>50	>9.5	>50
Run 2	-10	0.5	6.5	22	9.7	48
Run 3	-10	1.0	5.9	9	9.1	29
Run 4	-30	0.5	5.6	6	8.8	25
Run 5	-30	1.0	5.3	3.5	8.1	14

The effect of a lower air temperature was evaluated by comparing Run 2 to Run 4, and Run 3 to Run 5. Lower air temperatures increase the rate of freezing of the solid ice crust, and thus the rate at which strength is gained by the forming ice cover. By keeping the ice discharge constant in all simulations, it is expected that a lower air temperature would result in a thinner ice cover and less staging. The stable water level at km 4.0 in Run 4 was approximately 0.9 m less than that of Run 2. Likewise, the stable water level at km 4.0 in Run 5 was 0.6 m less than that of Run 3. The difference in stable water levels was greater at the location further from the ice boom. For example, at km 2.0 the stable water level of Run 5 was 1.0 m less than that of Run 3.

The time series of water levels in Run 1 and Run 5 are relatively smooth, and show less fluctuations than the water levels of Runs 2, 3, and 4. The fluctuations indicate the occurrence of consolidation events in the simulations. In Runs 1 and 5, the parameter values resulted in no sudden collapses of the ice cover. Run 1 did not account for thermal

strengthening, thus the parcels in the ice cover continually and gradually thickened as stresses increased. In Run 5, the fast growth and high failure stress of the solid crust added sufficient strength to prevent any collapses of the ice cover from occurring. In Run 2, the growth rate and failure stress of the solid crust was relatively low. Only minor shoving occurred near the leading edge of the cover. In Runs 3 and 4, the combinations of growth rate and failure stress allowed the ice front to progress upstream at a low thickness (the crust temporarily stabilized the ice cover). As the cover lengthened and stresses on the frozen parcels increased, eventually the failure stress was exceeded and the parcels consolidated and mechanically thickened.

4.4.2.2 Ice Profiles and Thickness

The ice cover thickness and profiles along the centreline of the channel for all runs are shown in Figure 4.13 and Figure 4.14. Results were extracted at the end of the simulation (hour 50) for Runs 1 and 2, and at the time the ice front reached the upstream boundary for Runs 3, 4, and 5.

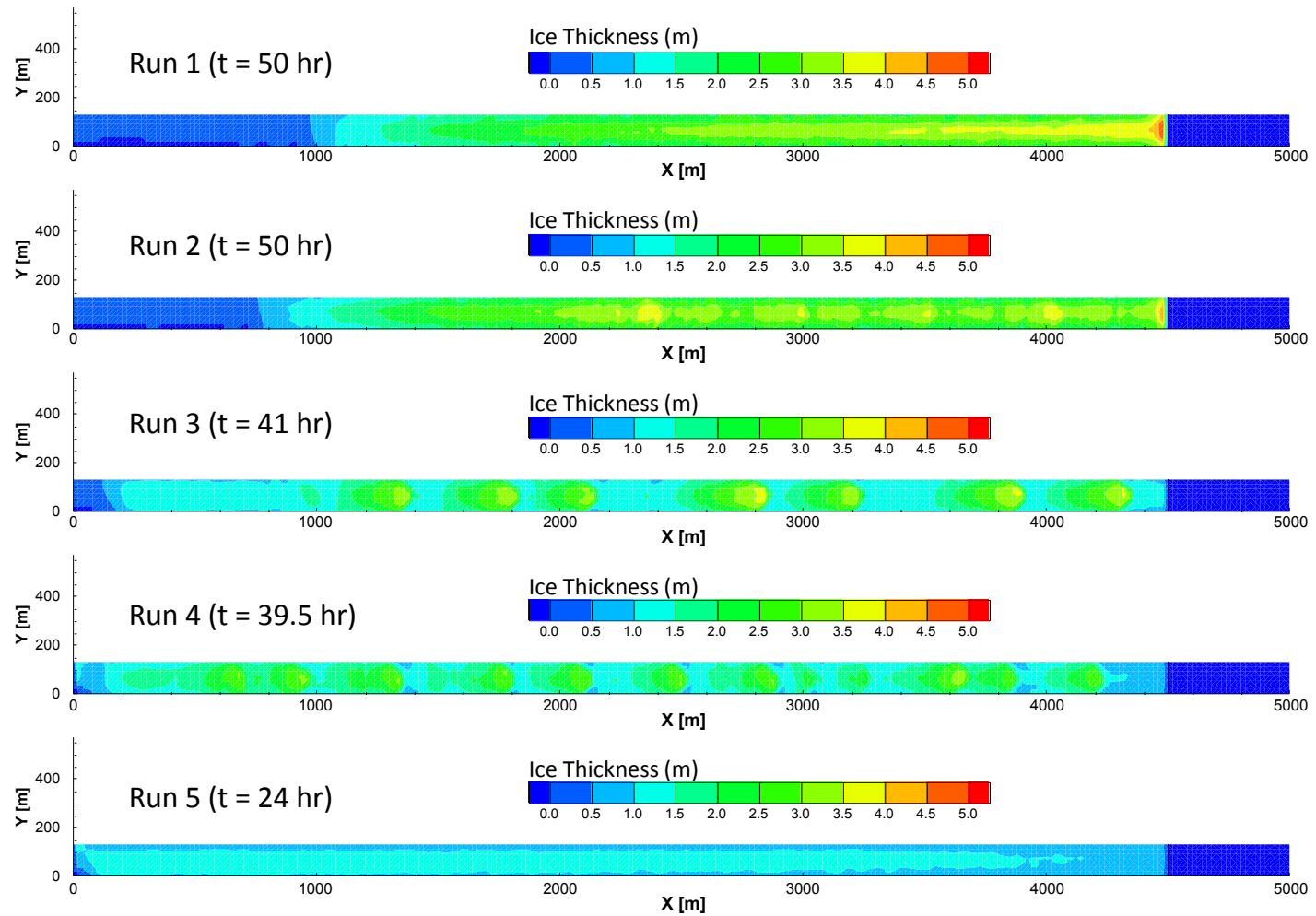


Figure 4.13. Simulated ice cover thickness in idealized channel simulations.

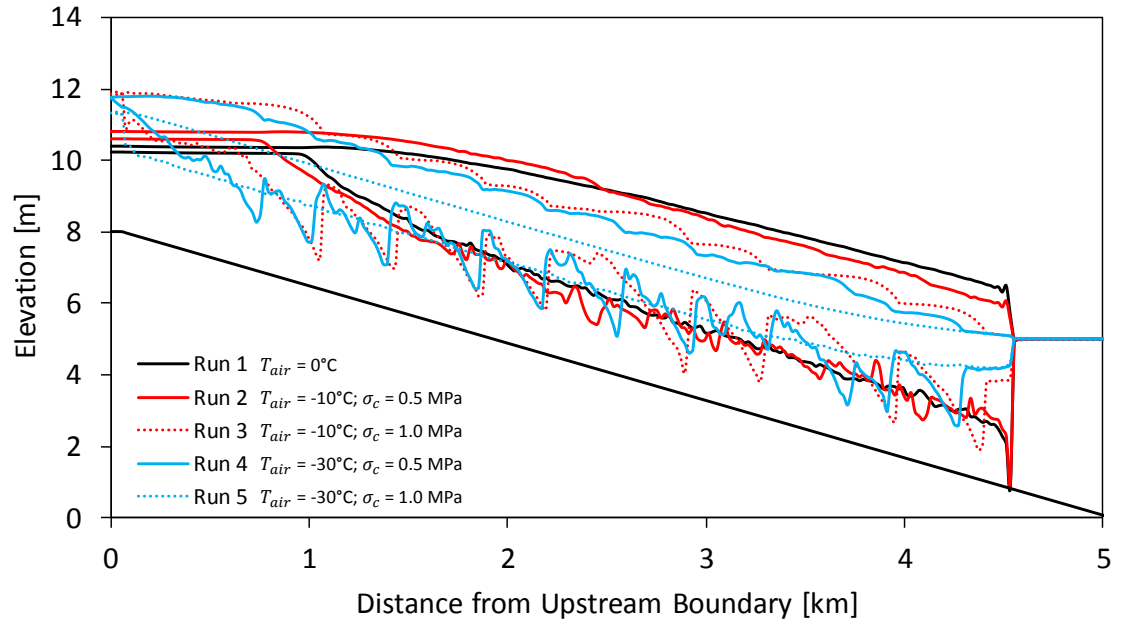


Figure 4.14. Ice profiles along centreline of channel for idealized channel simulations.

Incorporating effects of thermal strengthening has a noticeable impact on the final ice profile and thickness. The ice cover of Run 1 is thinnest at the upstream extent, and gradually gets thicker with distance downstream. The cover thickness does not vary locally along its length, except for near the ice boom where it is thickest. The average centreline thickness of the ice cover of Run 1 is approximately 3.4 m; near km 4.0 the thickness is approximately 3.6 m (likely near the equilibrium thickness). This is approximately the same ice cover thickness calculated using one-dimensional theory with a strength coefficient of 1.2 (thickness of 3.53 m calculated with Equation [1.13]).

The ice cover of Run 2 is relatively similar to Run 1; the average centreline ice thickness of Run 2 is approximately 3.3 m. This is because the rate of crust growth and failure stress are quite low, and do not provide much additional resistance to mechanical thickening. The bottom-of-ice profile of Run 2 is more variable than Run 1 due to the

small shove events that occur near the upstream edge as the cover progressed upstream. The top-of-ice profile of Run 2 is approximately 0.3 m lower than Run 1 near km 3.5. Upstream, near km 2.0, the top-of-ice profile of Run 2 is higher than Run 1, but this is because the cover in Run 1 is still thickening at this location. With a longer reach and longer simulation time, the top-of-ice profile of Run 2 would be consistently lower than Run 1.

Comparing Run 2 to Run 3, where crust strength was increased from 0.5 MPa to 1.0 MPa, shows a further decrease in the top-of-ice profile. In some areas, the top-of-ice profile of Run 3 is more than 0.5 m below Run 2. The ice thickness changes locally along the length of the ice cover of Run 3. This is a result of intermittent cycles of frontal progression and collapse of the cover. The average ice cover thickness of Run 3 was approximately 2.1 m, with some regions near the toes of consolidated portions near 3.5 m. The ice profile of Run 4 is similar to Run 3, with an average thickness of approximately 1.9 m and consolidation toe thicknesses up to 3.3 m. The ice thicknesses of Run 3 and Run 4 do not increase drastically at the ice boom as they do in Run 1 and Run 2. The ice cover of Run 5 is much thinner than the other simulations, with an average thickness of 1.1 m and no variation in thickness along its length. Due to the colder air temperature and high failure stress of the crust, thermal strengthening of the forming ice cover was sufficient to prevent significant mechanical thickening.

4.4.2.3 Discharge Variation caused by Ice Cover Consolidations

To investigate the effect of ice cover formation on flow within the channel, discharge was extracted at four sections upstream of the ice boom: km 0.8, km 2.0, km 3.2, and km 4.4. The time series of discharge for Run 1 and Run 3 are shown in Figure 4.15.

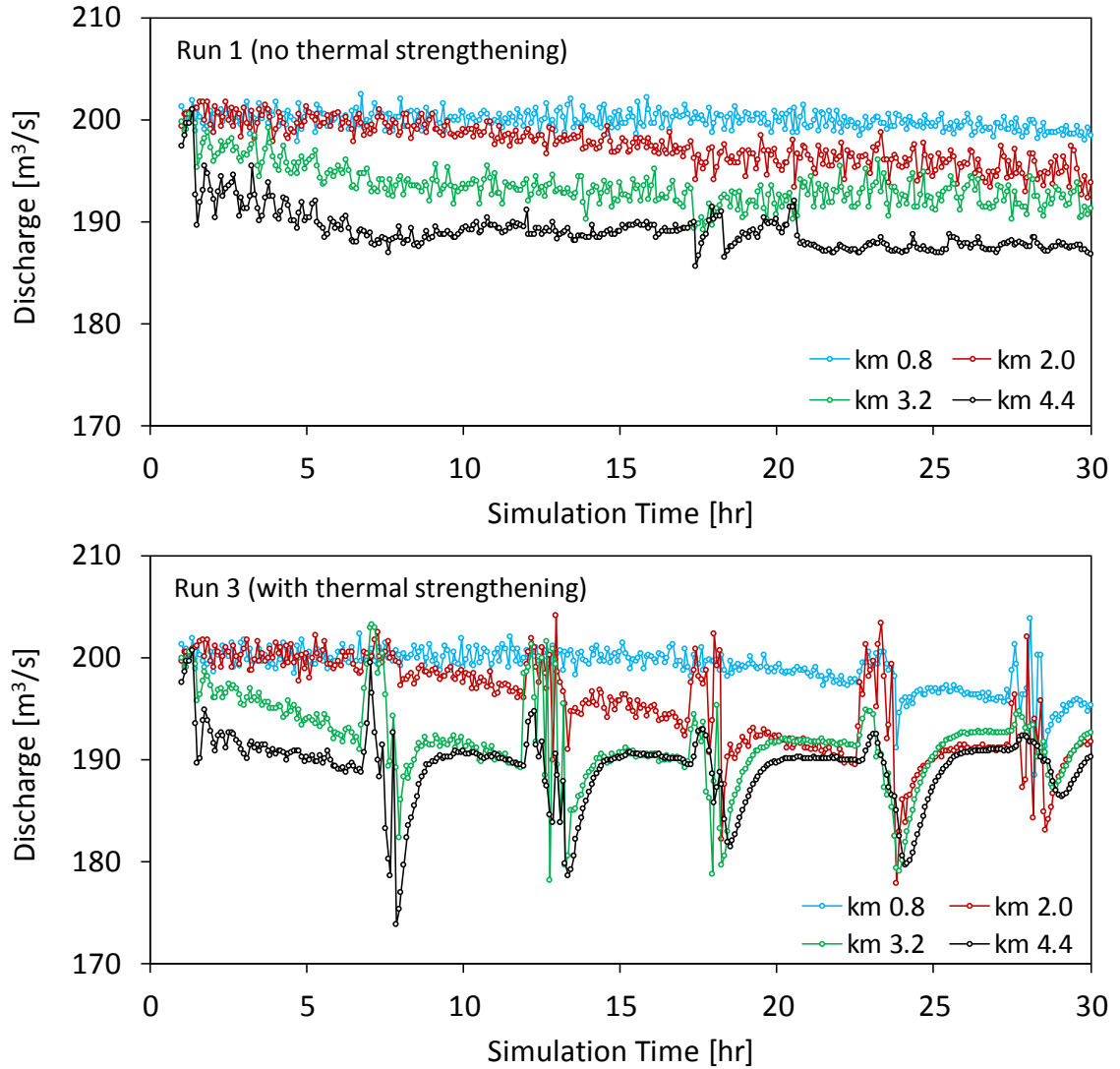


Figure 4.15. Discharge at sections along idealized channel. Top panel shows Run 1 (no thermal effects) and bottom panel shows Run 3 (air temperature -10°C).

In Run 1, the discharge decreased from $200 \text{ m}^3/\text{s}$ at km 4.4 and km 3.2 early in the simulation due to the thickening of ice at the boom which caused flow abstraction upstream. The progression of the ice cover upstream caused the discharge at km 2.0 to slowly decrease at approximately hour 10 of the simulation, while the discharge at km 0.8 remained near $200 \text{ m}^3/\text{s}$. With the exception of some noise associated with the numerical

calculation of discharge at each cross section, the flow changed steadily at each site as the effects of the ice cover were felt and flow was constricted.

The effect of ice cover consolidations on flow are immediately evident by examining the discharge extracted for Run 3. Similar to Run 1, each site experienced a steady decrease in discharge as the ice cover forced flow into storage. However, the discharge periodically fluctuated due to ice cover consolidations. During each event, the discharge at a site located within the consolidated reach, or downstream, increased abruptly as the surge of water passed by, then decreased as the thickened ice cover forced water into storage again. As a specific example, consider the consolidation that occurred between hour 20 and hour 25. As shown in Figure 4.16, the event caused the collapse and thickening of the ice cover approximately between km 2.0 and km 3.0 (toe located at km 2.9). As shown in Figure 4.15, the discharge first increased at km 2.0 by approximately $10 \text{ m}^3/\text{s}$. The discharge at km 3.2 and km 4.4 (downstream of the consolidation toe) increased as the surge moved downstream, but then decreased as the cover thickened near km 2.9 and forced upstream flow into storage, reducing flow at the downstream stations. The discharge at km 2.0 remained elevated until the backwater effect from the thickened ice cover extended back to the site. As the water level upstream increased and the consolidation of the ice cover stopped, the flow beneath the ice cover returned to approximately the pre-event values at each of the three sites. Evidence of the changing discharge can also be seen in the small perturbations of water level in Run 3 shown in Figure 4.12. These numerical results are in agreement with field observations of changing water levels and flow hydraulics that occur during a consolidation event (e.g. Andres et al., 2003; Wazney et al., 2018), albeit to a smaller scale.

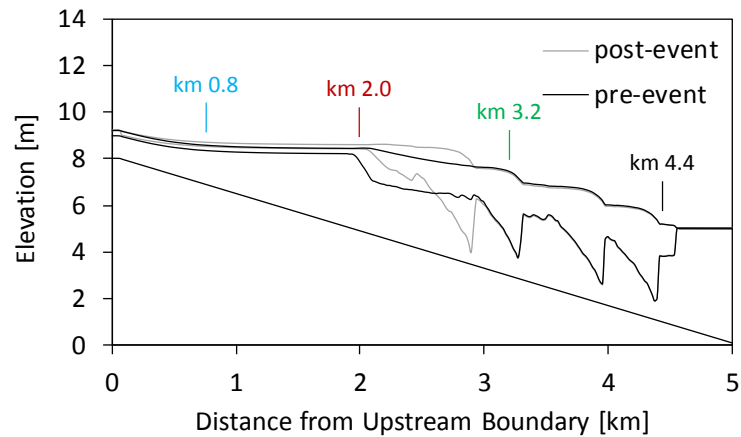


Figure 4.16. Ice cover profiles before and after consolidation event occurring at hour 23 in Run 3.

4.4.2.4 Parcel Response to Consolidation

To demonstrate the functionality of the added model outputs, pertinent variables are shown in Figure 4.17 for a parcel located just downstream of the toe of the consolidation event that occurred at hour 23 in Run 3. As shown in the top panels, the parcels within the consolidated reach thickened to over 3.5 m near the toe. The bottom panel shows the properties of the bolded parcel related to stresses and resistance to consolidation. The bolded parcel did not fail during the event. The rubble strength remains approximately constant over the event, indicating that the parcel did not thicken significantly. The contribution of the crust to the total consolidation resistance increases over the timeframe shown, as the thickness of the crust increases with exposure to the cold ambient air. During the process of mechanical thickening of the ice cover upstream (involving the failure of several previously frozen parcels), the internal force on the bolded parcel increases abruptly. As the upstream ice accumulation thickens, the forces of water shear and ice cover weight increase, and these forces are transferred to the intact, frozen cover downstream (where the bolded parcel is located). Since the increase in internal force did

not exceed the total resistance of the bolded parcel, it remained “frozen” and resisted mechanical thickening.

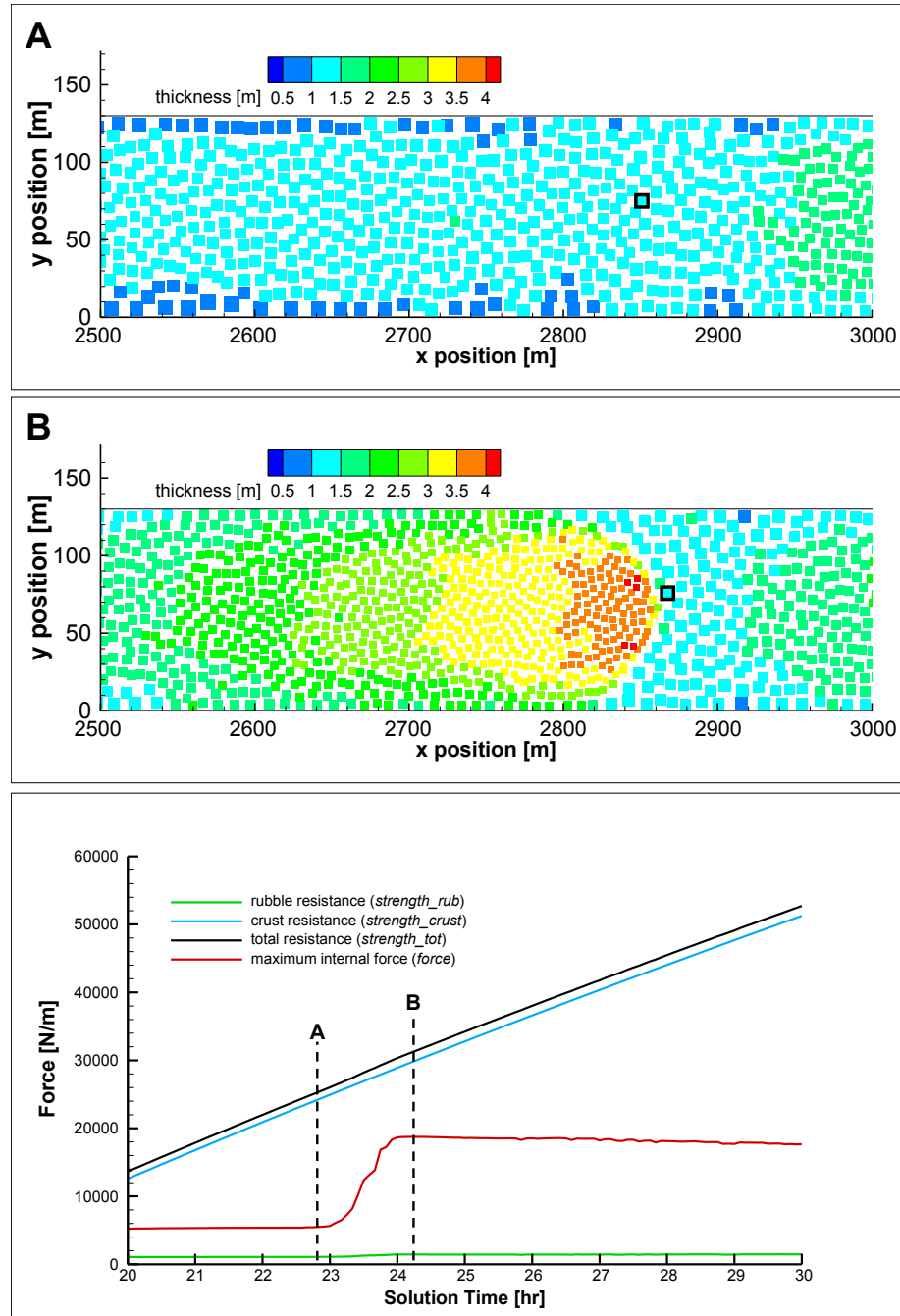


Figure 4.17. Parcel output during consolidation at hour 23 in Run 3. Top panels show plan view of parcel thickness near the toe of the consolidation. Bottom panel shows consolidation resistance and forces acting on bolded parcel in top panels during the consolidation.

4.4.3 Sensitivity to Model Parameters

Additional simulations were performed to evaluate the sensitivity of the model results to the user-defined values of the parameters v_{crit} and h_{crit} . Run 3 was used as the base case, and the parameters v_{crit} and h_{crit} were varied over a reasonable range to investigate the impact on the simulated ice profile. The suite of sensitivity runs is shown in Table 4.6.

Table 4.6. Sensitivity runs performed on idealized channel, using Run 3 as base case.

	T_{air} [°C]	σ_{crust} [MPa]	v_{crit} [m/s]	h_{crit} [m]
Run 3a	-10	1.0	0.10	0.002
Run 3b	-10	1.0	0.08	0.002
Run 3c	-10	1.0	0.01	0.002
Run 3d	-10	1.0	0.002	0.002
Run 3e	-10	1.0	0.05	0.03
Run 3f	-10	1.0	0.05	0.01
Run 3g	-10	1.0	0.05	0.001
Run 3h	-10	1.0	0.05	0.0005

The effect of varying the critical freezing velocity, v_{crit} , is shown by the ice profiles of Runs 3a, 3b, 3c, and 3d, in Figure 4.18. In general, higher values of v_{crit} allowed the ice front to progress upstream marginally faster, and at a slightly reduced thickness. As the ice parcels slow down and begin to mechanically thicken at the leading edge (or ice boom), a higher critical freezing velocity will allow the solid crust to start growing earlier. Thus, at a higher v_{crit} , the parcel will transition to a frozen parcel earlier. The top-of-ice profiles are similar for all runs, but the ice thickness variation along the cover is different. In Runs 3a, 3b, and 3c, the ice cover includes areas of thicker ice separated by areas of thinner ice. In these runs, the parcels approaching the leading edge of the ice cover (in the backwatered region) can more easily meet the velocity criteria and begin forming the solid

crust. This creates areas of thin, thermally strengthened ice cover. In Run 3d (lowest value of v_{crit}), the thickness of the ice cover is more consistent and does not have areas as thin as the other runs. The low value of v_{crit} prevents parcels from meeting the velocity criteria until they are very close to the leading edge, thus they will mechanically thicken more before obtaining strength from thermal processes. The average ice cover thicknesses from km 1.5 to km 4.5 for Runs 3a, 3b, 3c, and 3d were 2.15 m, 2.18 m, 2.33 m, and 2.53 m, respectively (i.e. average thickness increased as v_{crit} decreased).

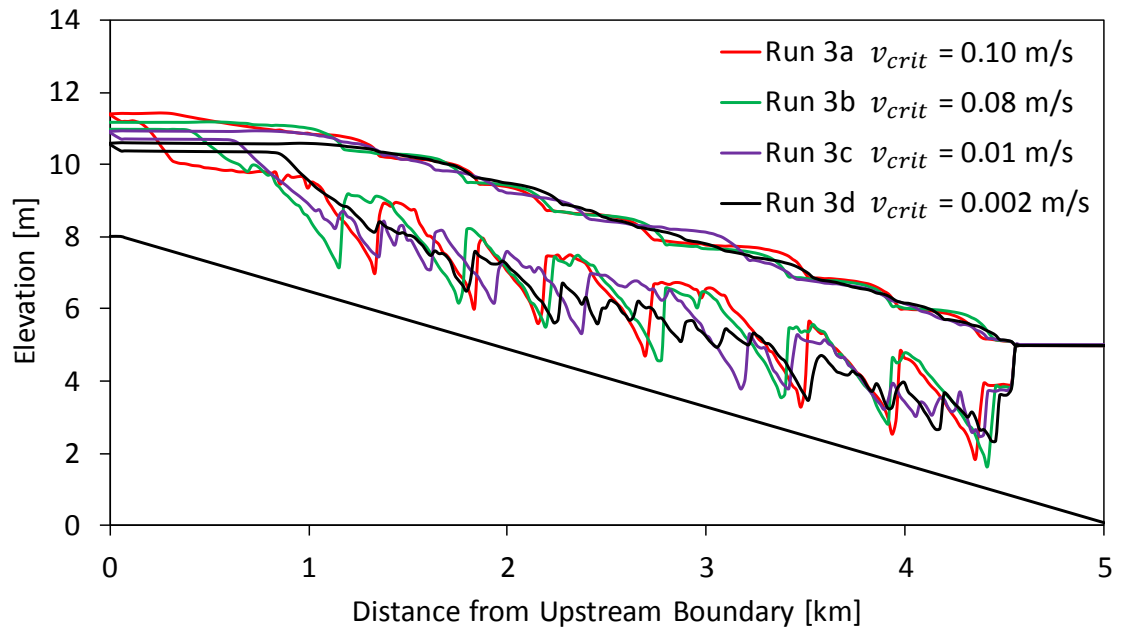


Figure 4.18. Sensitivity of ice cover formation to critical freezing velocity, using parameters of Run 3 as the base case. Ice profile extracted at hour 40 of the simulation.

The effect of varying the critical crust thickness, h_{crit} , is shown by the ice profiles of Runs 3e, 3f, 3g, and 3h, in Figure 4.19. The influence of h_{crit} on the final ice cover thickness is not as straight-forward as v_{crit} . In general, a higher value of h_{crit} will mean that a parcel will be subjected to mechanical thickening for a longer time before becoming frozen. However, once the parcel becomes frozen, it will have more strength (due to the

thicker crust), therefore it will be less susceptible to failure and further mechanical thickening. In Runs 3g and 3h, the values of h_{crit} were low; the thin crust was more susceptible to immediate failure as the stresses increased as the ice cover lengthened. Parcels in these runs went through several cycles of freezing and failure near the leading edge until they attained a thickness and backwater effect that allowed the solid crust to remain intact and allow the ice front to progress upstream. The average ice cover thicknesses from km 2.0 to km 4.5 of Runs 3g and 3h were 2.45 m and 2.55 m, respectively. In Run 3f, the increased value of h_{crit} resulted in parcels near the leading edge that were subjected to mechanical thickening for more time as the crust thickened before exceeding the critical value. However, once the crust thickness exceeded h_{crit} , the thicker crust provided more resistance to collapse in the progressing cover. Thus, the cover was able to progress upstream at an average thickness of 2.21 m (less than Runs 3g and 3h). In Run 3e (highest value of h_{crit}), the parcels underwent even more mechanical thickening before the solid crust thickness exceeded the critical value. Thus, when parcels transitioned to frozen parcels they did not fail (similar to Run 3f), but the cover progressed at a higher average thickness of 2.81 m. At very high values of h_{crit} , the ice cover profile would approach that of Run 1 (see Figure 4.14), where thermal considerations are not considered to influence ice dynamics.

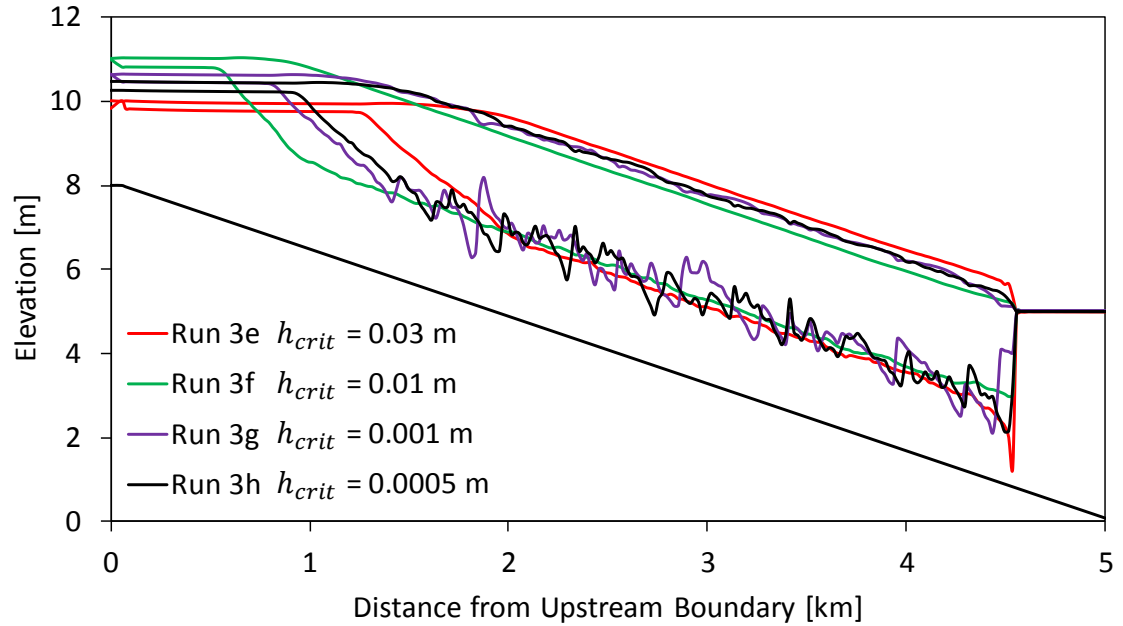


Figure 4.19. Sensitivity of ice cover formation to critical crust thickness, using parameters of Run 3 as the base case. Ice profile extracted at hour 40 of the simulation.

Similar sensitivity runs were performed using Run 2 as the base model (weaker crust). It was found that the variation in parameters affected the final ice cover profiles less than in the runs that used Run 3 as the base model (stronger crust), as shown in Figure 4.20.

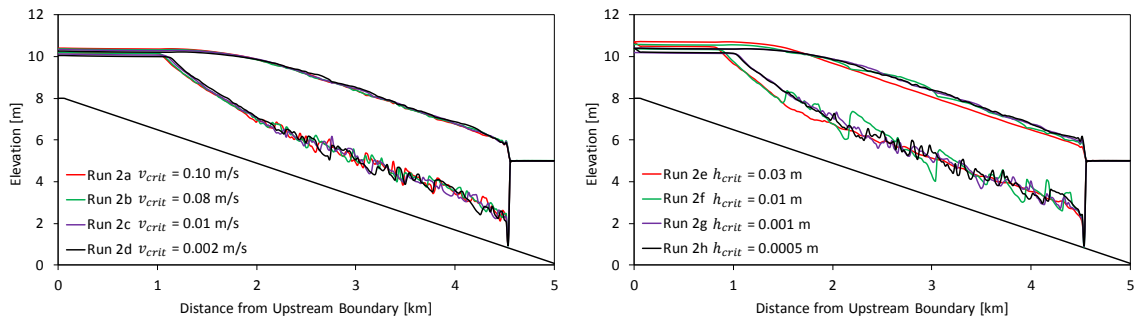


Figure 4.20. Sensitivity of ice cover formation to critical freezing velocity (left) and critical crust thickness (right), using parameters of Run 2 as the base case.

4.4.4 Conclusions and Applicability of the Revised Model

Results from the idealized channel simulations show that the formulations added to CRISSP2D to incorporate the effects of thermal strengthening of a forming ice cover work as intended, and have an impact on the final simulated ice profile. The formulations include the addition of three parameters to control the process, namely v_{crit} , h_{crit} , and σ_{crust} . The value of v_{crit} , the ice parcel velocity at which a solid crust begins to form, is thought to be low, on the order of a few centimetres per second or less. The ice rubble must be moving slow enough to allow any interstitial crust that grows to avoid immediate fracture (and loss of any added strength). In reality, the relative motion of the ice pieces themselves will have an impact on the ability of a surface crust to form. For example, a collection of ice pieces near the leading edge of the cover may be rearranging such that a surface crust cannot initiate. However, when modeled, the same collection of ice pieces (within a parcel) will have a sufficiently low velocity to allow the surface crust to form. The value of the critical crust thickness at which point the strength properties of the ice cover change, h_{crit} , is also unknown. However, results of the laboratory experiments of Chapter 3 suggest that even thin crusts can have a significant impact on the resistance of a floating ice cover to mechanical thickening. At the lab scale, this number was on the order of a few millimetres; at the field scale it is possible that this number is larger. Finally, the failure stress of the solid crust, σ_{crust} , is not exactly known. The compressive strength of crystalline ice, analysis of consolidation events on the Peace River by Andres (1999), and laboratory experiments conducted in this research suggest that values in the range of 0.2-1.5 MPa are plausible for the upper limit of crust strength.

With some uncertainty in the model parameter values, it is possible that different combinations could be used to achieve similar results when calibrating to field observations. Additional research is required to more clearly identify accurate values for each individual parameter. This can be done through laboratory and field studies, as well as the application of numerical models to case studies with good field datasets.

There are several applications where the incorporation of thermal processes on ice dynamics is an important consideration; some examples are provided below.

- (i) In a freeze-up scenario on a steep river prone to consolidation events, thermal strengthening of the ice cover may influence the timing and extent of shove events (e.g. Peace River in Alberta, and Dauphin River in Manitoba). Warm spells may reduce the strength of the ice cover and lead to collapse events, while extreme cold weather may allow a stable cover to more easily form. The revised model formulations could aid the modeler in determining the likelihood of ice cover consolidations given a set of weather and hydraulic conditions during the freeze-up period. A well calibrated model could enhance flood prediction and preparedness.
- (ii) On regulated rivers, the quick formation of a stable ice cover early in winter is often sought after, as it reduces the total volume of ice produced and the associated negative impacts on flow conveyance. For example, the formation of an ice cover on the Nelson River upstream of Jenpeg Generating Station in central Manitoba is critical to maximize the winter discharge capacity and potential hydroelectric production of downstream generating stations. Manitoba Hydro performs a temporary flow cutback at

Jenpeg each winter to reduce water velocities in critical regions that could otherwise form thick ice accumulations. Several of the channels form a stable ice cover through surface jamming of skim and frazil ice pans, and subsequent freezing of the cover in place. As flows are increased again, the cover has gained sufficient strength through thermal processes to resist collapse. The revised model formulations could help simulate these processes and inform the timing and duration of the flow cutback required to form a competent ice cover.

- (iii) Applying a river ice model to an entire winter season, from freeze-up, through mid winter, to breakup, requires consideration of thermal impacts on the strength of the ice cover. The thickness of the solid ice layer would continue to grow throughout the winter even after the cover has become stable. Snowfall would insulate the cover and reduce the growth rate of the solid ice layer. During spring, the balance between the rate of melt of the solid ice layer (and reduction in strength) and increase in applied forces (due to increased flows from snowmelt) determine if a thermal or mechanical breakup will occur. Accounting for the effects of thermal processes on the strength of an ice cover is critical in simulating these processes; the revised model formulations (while simplified) bring the CRISSP2D model one step closer to the ability to simulate this.

4.5 Case Study: Lower Dauphin River, Manitoba

4.5.1 Objective

A CRISSP2D model of the Lower Dauphin River was created and used as a case study for the revised model formulations. The field monitoring program discussed in Chapter 2 provided data such as water levels, visual observations of ice cover characteristics, top-of-ice elevations, and ice thickness estimates to compare with simulation results. The primary goal of this exercise was not to evaluate the ability of the revised model in achieving a perfect match with field measurements. The freeze-up of the Lower Dauphin River involves numerous processes that are still not entirely understood and cannot be captured in the current model (e.g. smoothing of the underside of the ice cover and development of open water leads). Rather, the purpose of the case study was to apply the CRISSP2D model to a real-world scenario, with real combinations of ice supply, air temperatures, channel geometry, and flow, to evaluate the plausible range of values of the crust strength parameter, σ_{crust} that allow the model to differentiate between rapid ice cover progression versus mechanical thickening. The model was first applied to the 2016-2017 freeze-up period, which was the coldest of the three seasons and was expected to have the most potential for highlighting the effects of thermal strengthening of the forming ice cover. The model was then applied to the 2015-2016 and 2017-2018 freeze-up periods to assess the transferability of the calibrated crust strength parameter to seasons with milder air temperatures where thermal strengthening of the cover may not have played as significant of a role.

4.5.2 Site Description and Model Setup

The Dauphin River study site was described in Section 2.2.3.1. The Dauphin River is over 50 km long and flows from Lake St. Martin to Lake Winnipeg, as shown in Figure 4.21. At a distance of approximately 11.2 km from Lake Winnipeg, the average bed slope of the river changes from 0.029% to 0.16%; the upstream reach is referred to as the Upper Dauphin River, while the downstream, steeper reach is referred to as the Lower Dauphin River. Applying a single two-dimensional model to the entire length of the Dauphin River would be impractical due to computational requirements. Therefore, the river was divided into two reaches, and a separate CRISSP2D model was created for each reach.

A CRISSP2D model was set up for the upper 35 km of the Dauphin River (see Figure 4.21). The purpose of this model was to simulate ice generation over the freeze-up period, which is transported downstream to the Lower Dauphin River. Ice leaving the downstream boundary of the model was used as the upstream boundary condition of ice discharge in the downstream model. The finite element mesh consisted of approximately 28,000 elements and 16,000 nodes. When run with steady discharge, the outflow at the downstream boundary was typically within 4% of the upstream discharge, indicating that the mesh was at an adequate resolution to maintain flow continuity. Bathymetry of the Dauphin River was collected in 2011 by KGS Group. This data was used with permission and interpolated to the numerical mesh. Field observations indicated that this reach does not typically exhibit significant dynamic interaction of surface ice floes until the ice front has progressed up the Lower Dauphin River. Therefore, the ice dynamics for this model were treated as “free-drift”, meaning force interactions between ice parcels were not calculated to save computational time. Hereafter, this model will be referred to as the “free-

drift” model. The upstream boundary of the free-drift model domain was the outlet of Lake St. Martin, which is approximately 26 km from Environment Canada gauge 05LM006. As the average water velocity at a discharge of $200 \text{ m}^3/\text{s}$ was calculated to be approximately 0.8 m/s, the travel time is approximately 9 hours for water leaving Lake St. Martin to reach the Environment Canada gauge location. As such, the discharge at gauge 05LM006 was prescribed at the upstream boundary, with a time shift of -9 hours. The simplified routing method was deemed appropriate as the discharge does not typically fluctuate rapidly during the freeze-up period. The water depth obtained from a calibrated open water HEC-RAS model was prescribed as the downstream boundary condition. The upstream water temperature boundary condition was assumed to be 0.01°C , which is typical of water exiting a lake at the time of ice formation.

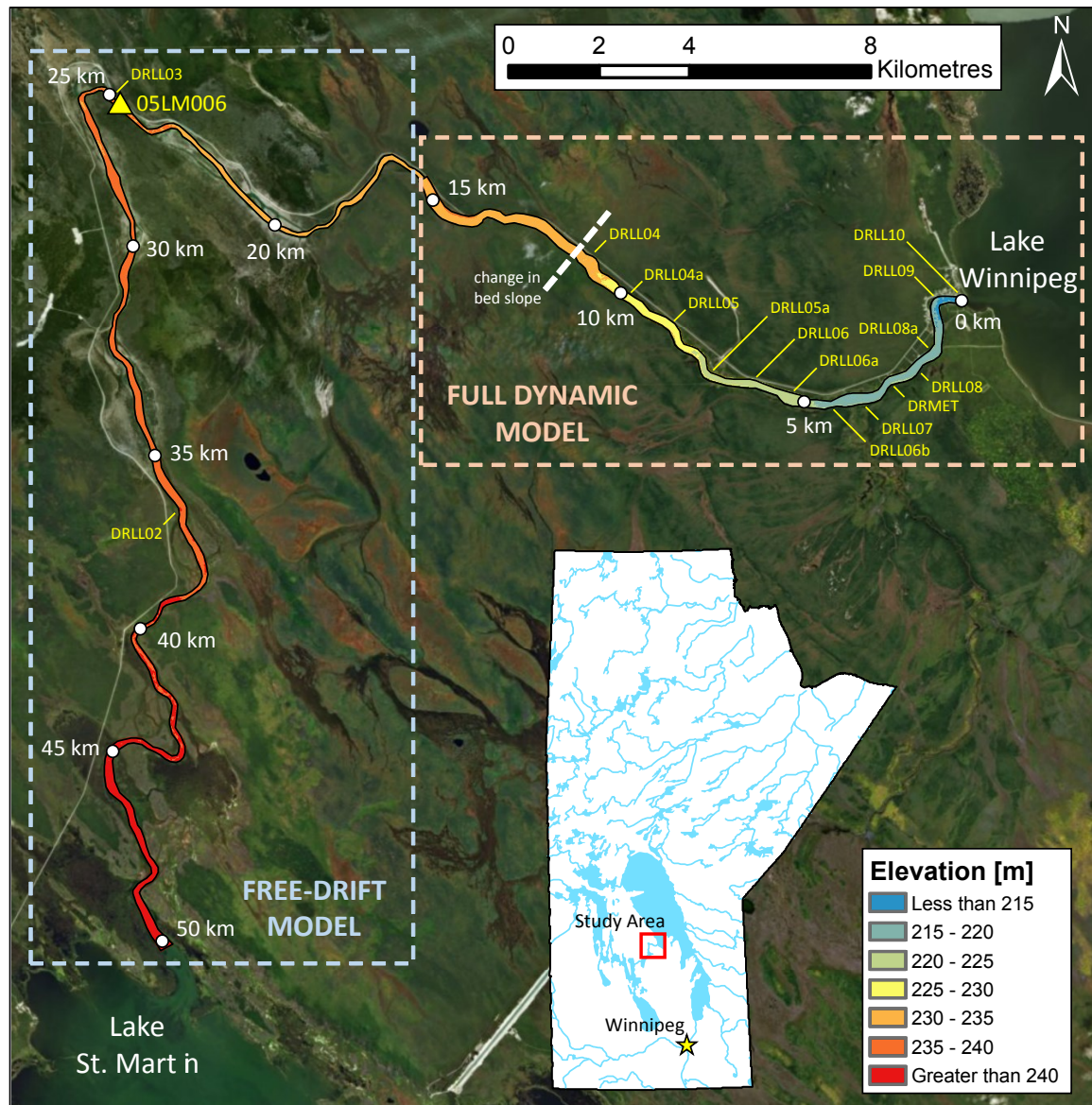


Figure 4.21. Dauphin River study area and boundaries of free-drift and full dynamic CRISSP2D models.

A separate model was set up for the downstream 15 km reach (see Figure 4.21). This model was configured to apply the full ice dynamic equations to account for parcel interactions, which is necessary to simulate the process of ice cover progression and thickening. Hereafter, this model will be referred to as the “full dynamic” model. The main channel bathymetry used in the model was also obtained by KGS Group in 2011.

Due to high water velocities and shallow areas along the reach, the KGS bathymetry did not extend from bank to bank in several areas, as shown in Figure 4.22. A shoreline survey of the left bank was conducted at several locations along the reach in 2016 as part of this research to supplement the main channel bathymetry. The surveyed left bank profiles were mirrored across the river to the right bank (which was not accessible), using satellite imagery to aid in alignment. The surveyed bank sections were then interpolated with the interpolation tool in HEC-RAS, following the shoreline of each bank. The composite scatter set is shown in Figure 4.22. When interpolating to the mesh, the KGS bathymetry was used for the main channel (i.e. wetted portion of the river), while the surveyed banks were used for the dry portions of the river under open water conditions.



Figure 4.22. Bathymetry sources for full dynamic model. Yellow points are from bathymetric survey performed by KGS Group in 2011 and green points are interpolated bank profiles surveyed in 2016.

The numerical mesh of the full dynamic model consisted of approximately 20,000 elements and 10,000 nodes, resulting in 5-7 elements across wetted flow areas in the model.

When run with steady discharge, the outflow at the downstream boundary was typically within 2-5% of the upstream discharge, indicating that the mesh was at an adequate resolution to maintain flow continuity. Due to model run times, the full dynamic model was not formulated to consider creation of new parcels by thermal processes, as this would drastically increase the number of ice parcels in the model and further increase model run times. Thermal processes were only considered to add mass to existing parcels in the model, which were introduced at the upstream boundary.

4.5.3 Meteorological Data and Heat Exchange Formulation

Meteorological data collected at the installed weather station at DRMET during the period of ice cover progression in the three monitoring seasons are shown in Figure 4.23. A similar comparison of air temperatures is shown in Figure 2.6. The wind speed and relative humidity did not differ greatly over the three seasons. The net radiation is consistent as well, but it is likely that snow coverage on the top of the sensor resulted in under-estimation of incoming solar radiation. The diurnal cycle of net radiation in 2016-2017 appears to cease just prior to November 21, around the same time that the Hodgson weather station, located about 90 km southeast of DRMET, recorded a few snowfall events. The 2017-2018 data shows the same pattern (although not shown in the timeframe of the plots); the net radiation varied diurnally until approximately the time of the snowfall event on November 4. Although it is possible that the incoming solar radiation plays a role in the diurnal variation in crust growth, the timing of consolidation events observed in the field did not appear to show a strong correlation with time of day (i.e. the ice cover was just as likely to shove during the night as it was during the day).

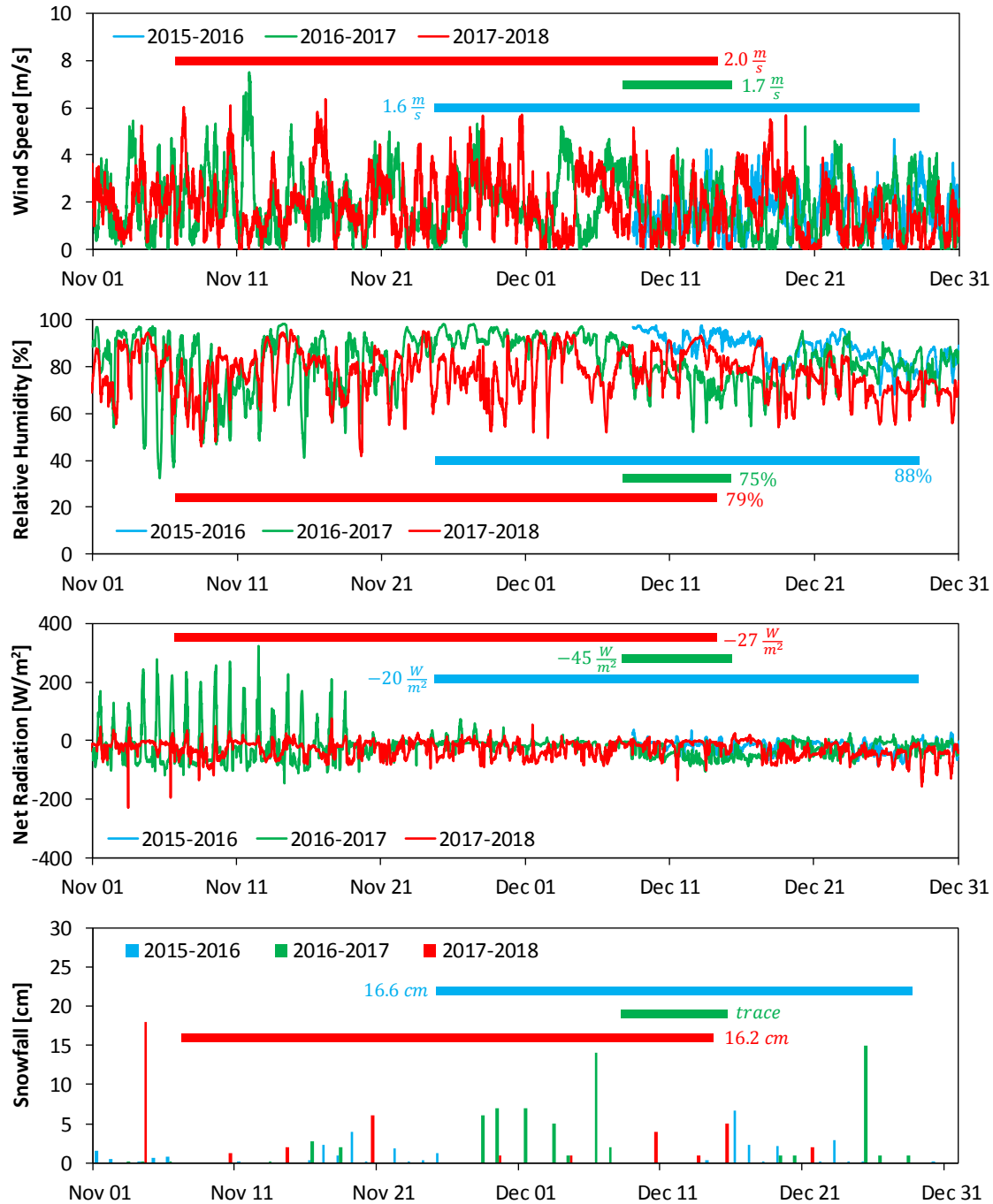


Figure 4.23. Comparison of meteorological data during ice formation periods in 2015-2016, 2016-2017, and 2017-2018. Wind speed, relative humidity, and net radiation collected at DRMET. Snowfall is from Environment Canada stations at Fisher Branch (2015-2016) and Hodgson (2016-2017 and 2017-2018). Bars indicate time of ice cover progression, and values indicate averages (or cumulative in the case of snowfall) over that period.

The linear heat transfer approach was used in conjunction with solar radiation in the CRISSP2D models, as shown in Equation [4.18]. As the net radiometer likely did not measure the actual incoming solar radiation, the contribution of solar radiation to heat exchange processes was incorporated into the models with theoretical equations and a daily cloud cover condition which was estimated from trail camera images. Weather conditions were updated every 15 minutes in the model to capture fluctuations in air temperature and solar radiation. For comparison purposes, the incoming solar radiation can be as high as approximately 200-300 W/m² during a sunny day and 0 W/m² at night. The sensible heat flux at an air temperature of -20°C is approximately 400-600 W/m².

4.5.4 Hydrodynamic Calibration

The free-drift and full dynamic models were calibrated to an open water period of steady flow on November 1, 2016, when the discharge recorded at gauge 05LM006 remained near 210 m³/s for several consecutive days. Water level loggers were deployed during this period, providing water surface elevations at multiple sites along the Dauphin River. The bed roughness in each model was modified until a good match was obtained between measured and simulated water levels. The bed roughness (Manning *n*) in the full dynamic model ranged from 0.025-0.035, while in the free-drift model it was set to a constant value of 0.027. A period of higher flow on July 1, 2017 was used for validation, when discharge at gauge 05LM006 was near 343 m³/s for several days. Table 4.7 shows simulated and measured water surface elevations for the hydrodynamic calibration and validation of the Dauphin River models. As shown, under a flow condition of 210 m³/s, most simulated water levels were within 5 cm of the measured values. The worst match occurred at site DRLL07, where the modeled water level was 9 cm higher than the

measured value. The errors for the higher flow simulation are slightly larger. In the full dynamic model, the simulated water levels were over 10 cm higher than the measured values at sites DRLL04a, DRLL06a, and DRLL07. In the free-drift model, the simulated water level was 10 cm lower than the measured value at DRLL02. These differences in water level are attributed to the limited bathymetric data, which was focused mainly on the centre portion of the channel and had to be extrapolated to the banks. Near DRLL07 and DRLL06a, low-lying overbank areas and islands were not captured in the measured bathymetry, thus it is expected that simulated water levels may have more error in these areas. Still, both models were able to reproduce water levels over a large spatial area and under a wide range in flow conditions to a satisfactory degree.

Table 4.7. Comparison of water levels for hydrodynamic calibration and validation of Dauphin River models.

Model	Site	Water Surface Elevations [m]						
		November 1, 2016 (Q = 210 m ³ /s)			July 1, 2017 (Q = 343 m ³ /s)			Difference
		Measured	Simulated	Difference	Measured	Simulated	Difference	
Free-Drift	DRLL02	240.50	240.46	-0.04	241.21	241.11	-0.10	
	DRLL03	237.58	237.61	+0.03	238.25	238.24	-0.01	
	DRLL04a	229.89	229.93	+0.04	230.21	230.33	+0.12	
Full Dynamic	DRLL05	227.37	227.36	-0.01	no data	227.85		
	DRLL05a	224.44	224.48	+0.04	224.83	224.81	-0.01	
	DRLL06	222.31	222.34	+0.03	222.77	222.70	-0.07	
	DRLL06a	221.32	221.27	-0.05	221.50	221.60	+0.10	
	DRLL06b	220.36	220.29	-0.07	220.62	220.61	-0.01	
	DRLL07	219.49	219.40	-0.09	219.58	219.77	+0.19	
	DRLL08	218.27	218.29	+0.02	218.48	218.57	+0.09	
	DRLL09	218.19	218.17	-0.02	218.31	218.32	+0.02	
	DRLL10	218.14	218.14	+0.00	218.23	218.23	+0.00	

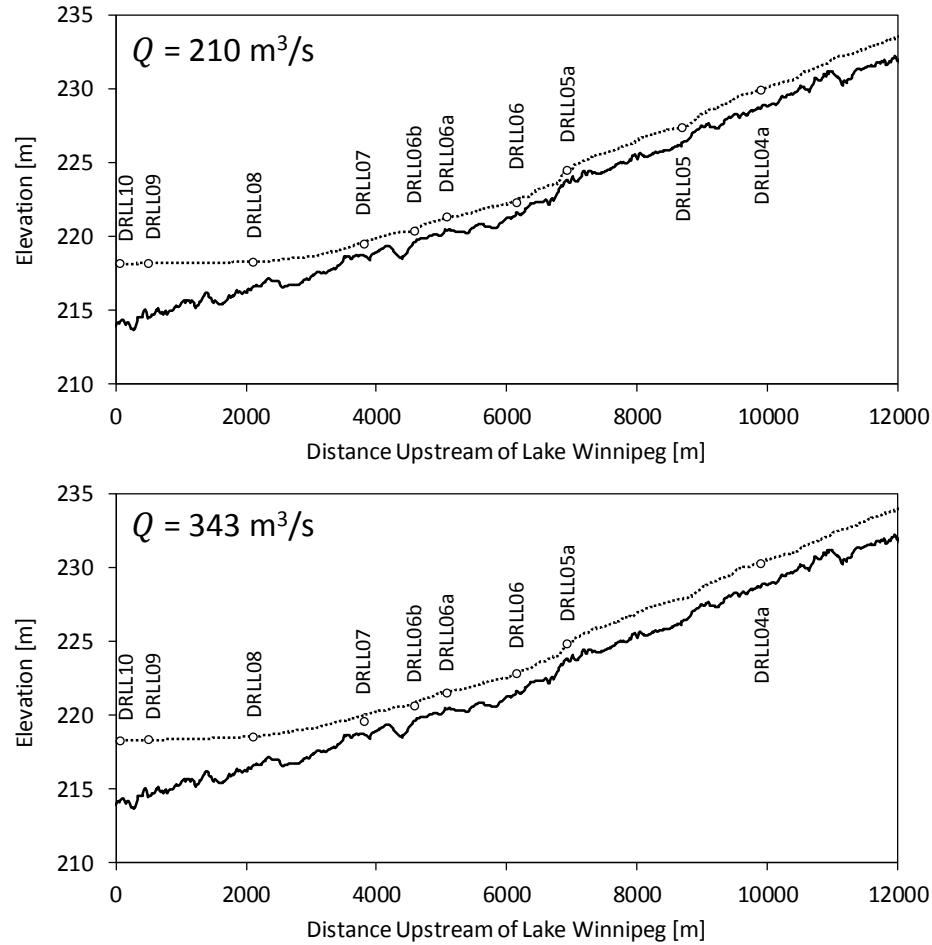


Figure 4.24. Full dynamic model simulation of open water levels at $Q = 210 \text{ m}^3/\text{s}$ (top) and $Q = 343 \text{ m}^3/\text{s}$ (bottom). Markers indicate measured water levels and dashed line indicates simulated water surface profile.

4.5.5 Calculation of Surface Ice Discharge

4.5.5.1 Field Estimation of Ice Discharge

To provide calibration data for the free-drift model, ice discharge was estimated on multiple dates during the 2016-2017 and 2017-2018 seasons. No field estimates of ice discharge were made in 2015-2016. During the site visit on December 14, 2016, the UAV was commissioned to record plan video of the surface ice near DRLL03 (about 25 km upstream of Lake Winnipeg; average river width of approximately 77 m). A reference

object was set up in the video for scale to allow for post-processing of the video footage with a particle tracking software to estimate the velocity of the surface ice floes. Video taken on December 14, 2016 at 10:50 am corresponded to a concurrent air temperature of -21.7°C recorded at DRMET. A snapshot of the video is shown in Figure 4.25.



Figure 4.25. Snapshot of surface ice floes near DRLL03 taken with unmanned aerial vehicle on December 14, 2016 (left). Concurrent air temperature was -21.7°C . Flow is from top to bottom. Average width of river is approximately 77 m. Right image shows the surface ice floes on November 21, 2017 (concurrent air temperature of -11.9°C) for comparison.

Analysis of a series of images during the video indicated that the average surface concentration of the ice floes was approximately 0.56. The particle tracking analysis indicated that the average velocity of the ice floes was approximately 1.1 m/s. The ice discharge was then estimated using Equation [4.24].

$$Q_{ice} = V_{ice}NBt_{ice} \quad [4.24]$$

where Q_{ice} is the ice discharge, V_{ice} is the average surface ice velocity, N is the average surface ice concentration, B is the river width, and t_{ice} is the average thickness of a surface ice floe. The major unknown in the estimation of ice discharge with this method was the ice thickness. Ice floes were not extracted from the river to measure the thickness directly on this date, but visual observations of the surface ice floes were made on a different date

(November 21, 2017, concurrent air temperature of -11.9°C), shown in Figure 4.25 and Figure 4.26. The floes consisted of a solid surface layer with some porous frazil slush underneath. Smaller floes (less than about 6 m^2) were estimated to have a solid layer thickness of 2 cm with a 10 cm layer of frazil slush (porosity of about 0.6), while larger floes (greater than 6 m^2) were estimated to have a solid layer thickness of 8-10 cm with about 25 cm of frazil slush underneath. These observations suggest that the equivalent thickness of the ice floes could range from 6-20 cm on November 21, 2017.



Figure 4.26. Photograph of surface ice floes on November 21, 2017. Concurrent air temperature was -11.9°C .

As shown in Figure 4.25, the ice floes on December 14, 2016 had a similar appearance to those on November 21, 2017, but were visually whiter in colour, indicating that they may be slightly thicker. Using an average equivalent ice floe thickness of 0.10-0.20 m (on the higher end of the range estimated on November 21, 2017), the ice discharge on December 14, 2016 was estimated to be approximately $7.0 \text{ m}^3/\text{s} \pm 2.4 \text{ m}^3/\text{s}$ at site

DRLL03. A study conducted by KGS Group in 2014 noted that the estimated ice discharge at the outlet of the Lower Dauphin River could reach 5 m³/s when the air temperature decreased below -20°C for several days, and could reach 10 m³/s with more extreme winter conditions and strong winds (KGS Group and North/South Consultants, 2014). This is roughly in agreement with the field estimated ice discharge on December 14, 2016.

Following the same approach, ice discharge was estimated on four different dates at site DRLL03 in 2017 (November 21-23, and December 14). The calculated ice discharges based on these videos are shown in Table 4.8. The high and low estimates assumed an equivalent ice floe thickness of 0.15 m and 0.05 m, respectively. Note the low estimate of ice discharge on November 23 despite the similar air temperature.

Table 4.8. Ice discharges calculated from field observations in 2017.

Time of Observation	Air Temperature [°C]	Ice Discharge [m ³ /s]		
		Low Estimate ($t_{ice} = 0.05$ m)	Mid Estimate ($t_{ice} = 0.10$ m)	High Estimate ($t_{ice} = 0.15$ m)
Nov. 21 2017 8:30 am	-11.9	1.2	2.5	3.7
Nov. 21 2017 8:30 am	-11.9	1.5	3.1	4.6
Nov. 22 2017 8:30 am	-14.8	1.1	2.2	3.3
Nov. 23 2017 8:30 am	-12.0	0.4	0.9	1.3
Dec. 14 2017 9:45 am	-11.9	1.3	2.6	3.9

4.5.5.2 Simulation of Ice Discharge with Free-Drift Models

The free-drift model was used to generate a time series of ice discharge over each of the three freeze-up periods. The model generated skim ice and frazil ice, which was transferred to the water surface from suspension in the water column. A small portion of the reach near the downstream boundary was prescribed a high frazil buoyant velocity to ensure that all suspended ice became surface ice before leaving the model domain (for

computation of ice discharge). For each season, the air-water heat transfer coefficient, H_{wa} , was modified until the simulated ice discharge matched the field estimates. The models were started on the date at which ice first appeared in trail camera images, which corresponded to the time at which water temperatures decreased to 0°C along the reach.

The 2016-2017 free-drift model started on December 4, 2016 and ran through the full period of ice cover progression up the Lower Dauphin River. As shown in Figure 4.27, the discharge fluctuated between approximately 200 m³/s and 240 m³/s over the simulation period. The air temperature remained above -5°C until December 7, when it suddenly dropped, reaching -20°C on December 9. From December 9 to 18, the average air temperature was -23.1°C; however the air temperature fluctuated diurnally by as much as 10°C. Temperatures rose after December 18, reaching -3.5°C on December 20. The surface ice discharge was extracted from the downstream boundary, shown in panel (c) of Figure 4.27. It was found that a heat transfer coefficient of 30 W/m²/°C at the air-water interface produced a reasonable match between the simulated and observed ice discharge.

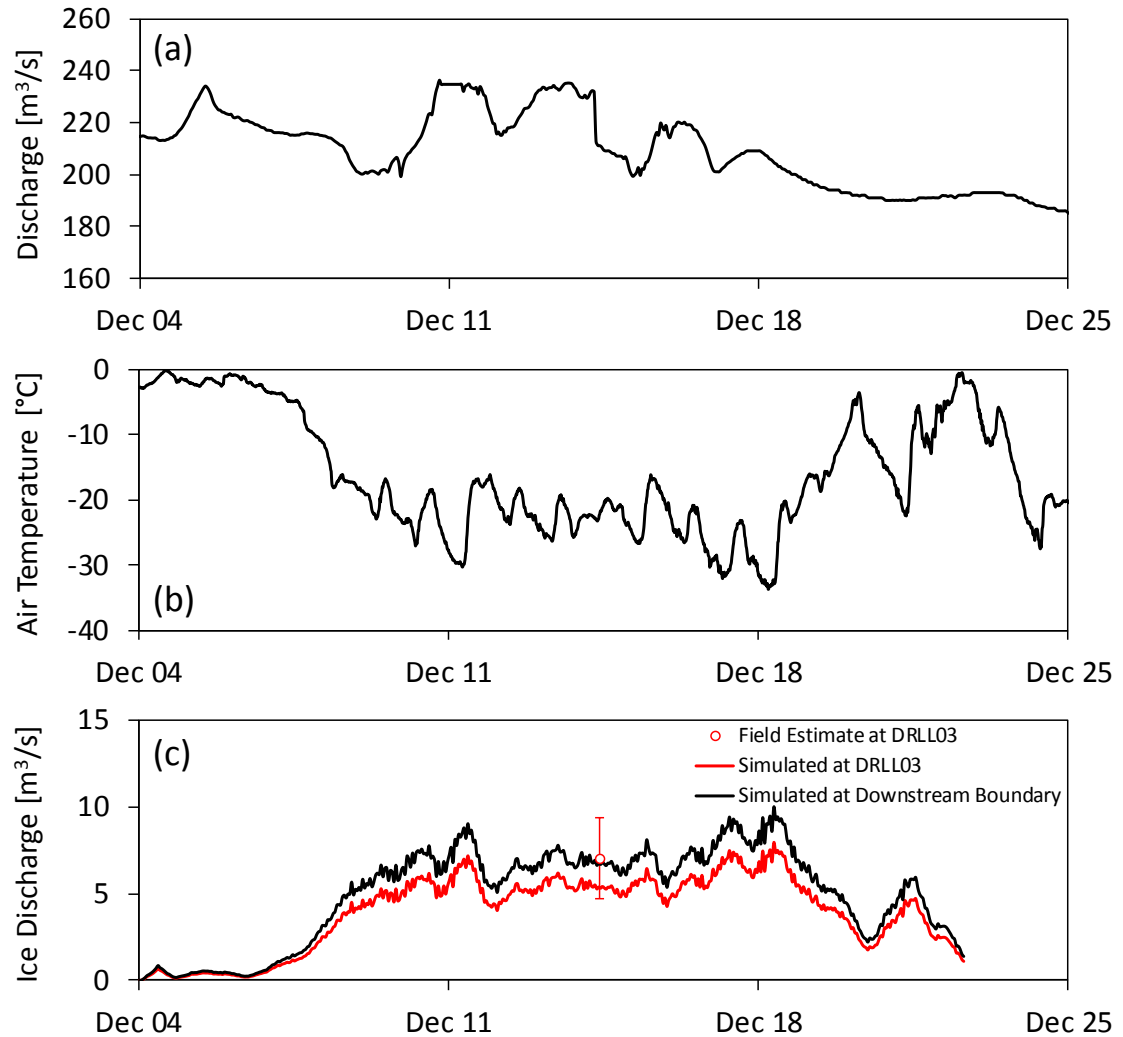


Figure 4.27. Conditions during freeze-up 2016-2017; (a) discharge at gauge 05LM006, (b) air temperature recorded at installed meteorological site at DRMET, and (c) simulated ice generation in free-drift CRISSP2D model. Red marker shows field estimated ice discharge and whiskers show the approximate range of uncertainty in the estimate.

The 2017-2018 free-drift model started on November 3, 2017 and was run until December 15 when the ice cover progressed past site DRLL04. As shown in Figure 4.28, the discharge slowly decreased from approximately 230 m^3/s to 170 m^3/s over the simulation period. Following a brief cold spell on November 10 where air temperatures dropped as low as -17°C , the temperature rose and conditions were relatively mild for the next several weeks; the average air temperature from November 11 to December 4 was -

7.3°C. The air temperature briefly rose above 0°C on several occasions during this time. After December 4, temperatures generally decreased again, and reached -26°C on December 15. A linear heat transfer coefficient, H_{wa} , of 20 W/m²/°C was found to produce the best match between simulated and field estimated ice discharges in 2017-2018.

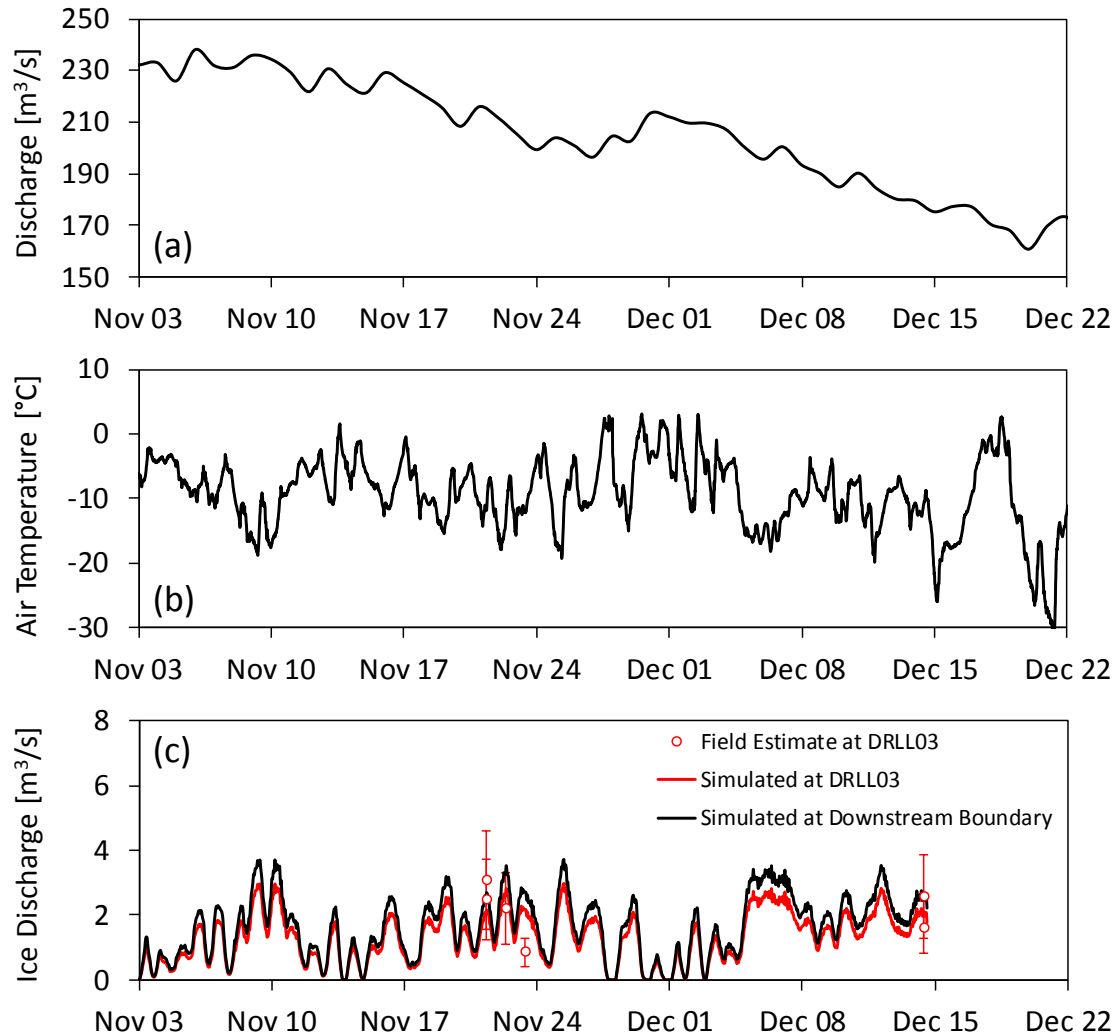


Figure 4.28. Conditions during freeze-up 2017-2018; (a) discharge at gauge 05LM006, (b) air temperature, and (c) simulated ice generation in free-drift CRISSP2D model. Red marker shows field estimated ice discharge and whiskers show the approximate range of uncertainty in the estimate.

The discrepancy in the value H_{wa} between the 2016-2017 and 2017-2018 is a curious result. One possible explanation for a variation in heat transfer coefficient between seasons is a change in wind speed, but Figure 4.23 shows that the average wind speed did not differ greatly between the seasons. In this case, the discrepancy may be partially explained by effects of cloud cover on the net longwave radiation heat flux that is lumped into the linear heat transfer coefficient. Both the surface of the Earth and atmosphere emit longwave radiation according to the Stefan-Boltzmann law, and the relative magnitudes of the radiation emitted by each will determine the net energy exchange at the water surface. In the case of a river near the freezing point, the net longwave radiation will always result in an energy loss from the water. The net longwave radiation from a water body at 0°C, ϕ_L , can be calculated with Equation [4.25] (Ashton, 1986)

$$\phi_L = \xi_c \sigma (273.2 + T_a)^4 - \xi_s \sigma (273.2)^4 \quad [4.25]$$

where ξ_c is the emissivity of the sky, T_a is the air temperature in °C, σ is the Stefan-Boltzmann constant (equal to $5.67 \times 10^{-8} \text{ W/m}^2/\text{K}^4$), and ξ_s is the emissivity of the water surface, taken to be 0.97. The first and second terms on the right side of Equation [4.25] constitute the longwave radiation emitted from the sky and water, respectively. Water at 0°C will emit approximately 300 W/m^2 of longwave radiation. The longwave radiation emitted from the sky depends on the emissivity of the sky and cloud cover conditions. The clear sky atmospheric emissivity, ξ_a , can be calculated with Equation [4.26].

$$\xi_a = c + d\sqrt{e} \quad [4.26]$$

where c and d are empirical coefficients, and e is the vapour pressure in millibars. The values of the empirical coefficients c and d show some scatter, but are centred around 0.52

and 0.065, respectively (Brunt, 1932). The sky emissivity is adjusted for cloudy conditions with Equation [4.27] (Ashton, 2013; Jacobs, 1978).

$$\xi_c = \xi_a(1 + 0.026C) \quad [4.27]$$

where C is the cloudiness in tenths. Using the above equations, the net longwave radiation out of the river was calculated for a range of air temperatures, shown in Table 4.9. As shown, the net longwave radiation is greatest for clear, cold periods, and smallest for warm, cloudy periods.

Table 4.9. Computed net longwave radiation heat flux over open water at the freezing point.

Air Temperature [°C]	Net Longwave Radiation Heat Flux Out [W/m ²]		
	Cloudy Sky	Clear Sky	Difference
-5	70	119	49
-10	94	138	44
-15	116	155	39
-20	135	170	35

The approximate daily cloud cover during the three freeze-up periods was estimated with photographs from deployed trail cameras, and the results are shown in Figure 4.29. In general, there were more cloudy days during the freeze-up periods in 2015-2016 and 2017-2018 than there were in 2016-2017. From Table 4.9, it is plausible that the heat transfer coefficient calibrated to the 2016-2017 season with colder air temperatures and (on average) clearer skies may over predict the heat exchange in seasons with warmer air temperatures and cloudier skies.



Figure 4.29. Estimated cloud cover from trail camera imagery for three freeze-up periods.

The 2015-2016 free-drift model was run from November 20 to December 28, 2015. As shown in Figure 4.37, discharge increased to approximately $190 \text{ m}^3/\text{s}$ on November 24 early in the simulation, then slowly decreased to $120 \text{ m}^3/\text{s}$ on December 28. The average air temperature from November 20 to December 3 was -7°C . A period of warmer weather followed, where the average air temperature was -2.6°C between December 3 and December 14. The air temperature generally decreased after this, and reached a low of -27.6°C on December 26. The absence of field estimated ice discharge prevented calibration of H_{wa} . However, since the 2015-2016 freeze-up period was similar to 2017-2018 in terms of average air temperature and cloud conditions, the value of $20 \text{ W/m}^2/^\circ\text{C}$ was applied to the 2015-2016 free-drift model.

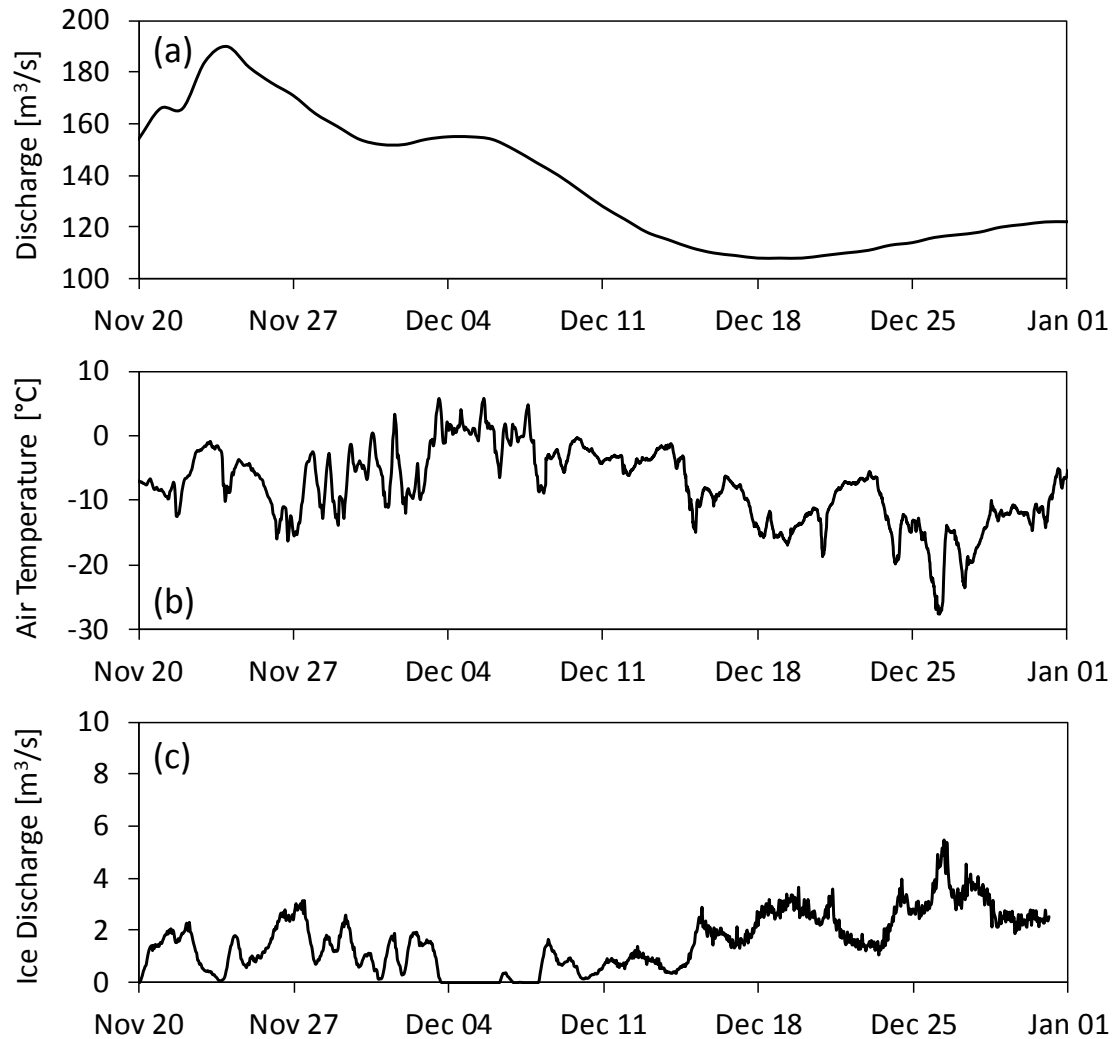


Figure 4.30. Conditions during freeze-up 2015-2016; (a) discharge at gauge 05LM006, (b) air temperature, and (c) simulated ice generation in free-drift CRISSP2D model.

There is evidently some uncertainty associated with imposing the simulated time series of ice discharge at the boundary of the full dynamic model. The time series were calibrated to few estimates of ice discharge in the field. The field estimates were inexact, and a fairly wide range of values were plausible due to uncertainties in ice floe thickness. However, ice discharge is rarely measured in the field. More commonly, surface ice concentration and velocity are estimated from shore to get a rough estimate of the ice discharge. The method used in this research to estimate ice discharge removes several

common sources of uncertainty in field measurements, such as river width, surface ice concentration, and ice velocity. Thus, the time series of ice discharge, calibrated to these field measurements, should provide a reasonable estimate of the rate at which ice is generated and added to the advancing ice front.

In some situations, surface ice can submerge to be transported underneath the ice cover at the leading edge. Field observations indicated that, during cold periods, the surface ice was mostly in the form of competent frazil pans. The high supply surface ice was observed to converge in the backwatered areas upstream of the stationary front, creating a moving surface ice layer with nearly 100% surface concentration, as shown in Figure 4.31.



Figure 4.31. Photograph of moving surface ice at site DRLL05 taken on December 12, 2016. Flow is from top to bottom. The top portion of the image shows surface ice that is moving at a higher velocity than the ice at the bottom of the image. The convergence of the surface ice creates a moving surface layer with nearly 100% surface concentration, shown at the bottom of the image.

During periods of warmer weather, surface ice on the Lower Dauphin River was observed at a lower concentration, and frazil pans did not appear to have the same competency. Figure 4.32 and Figure 4.33 show photographs from shore and with the unmanned aerial vehicle of these observations.



Figure 4.32. Appearance of surface ice at site DRLL05a on November 21, 2017. Note the loosely solidified frazil pans.



Figure 4.33. Appearance of surface ice at site DRLL06 on November 22, 2017. Note the loosely solidified frazil pans.

Observations from shore and video footage taken with the UAV suggested that the incoming ice mostly remained at the surface and packed against the stationary front, and did not overturn or submerge to be transported under the ice cover. This observation was also made by Lindenschmidt et al. (2012) who noted that the drawdown of surface ice fragments is reduced due to protection provided by the continuous, congested supply of surface ice. Therefore, all incoming ice in the full dynamic model was assumed to contribute to the advancement of the ice cover (i.e. no submergence criteria was used). On only a few occasions, during periods of low ice supply in the form of frazil slush (concurrent with warm air temperatures), was ice observed to be entrained at the leading edge. It is unclear if the ice would be immediately deposited on the underside of the ice cover, or if it would be transported a long distance downstream. Thus, the assumption that all incoming ice remained at the surface and was accumulated at the leading edge may lead to an over prediction of ice cover progression rate during periods of mild weather when the air temperature is only slightly below 0°C.

4.5.6 Full Dynamic Model Application to 2016-2017 Freeze-up Period

4.5.6.1 Simulation Setup and Selection of Parameters

The full dynamic model was run for the freeze-up period starting on December 8, 2016. Trail camera images indicated that the ice front began progressing upstream from Lake Winnipeg at approximately 4:30 pm on December 8. An ice boom was inserted near the downstream boundary of the model at this time to simulate the lake ice cover that obstructed surface ice transport. Two versions of the model were run. The first used the traditional approach to simulate ice dynamics (hereafter called the “base” run); the ice cover was only able to gain strength from frictional interlocking of ice floes which followed

a Mohr-Coulomb failure criterion with a constant friction angle. The second used the revised formulations presented in this research that incorporate thermal strengthening of the ice cover due to the growth of the interstitial crust (hereafter called the “thermal” run). The full dynamics models used a hydrodynamic time step of 0.1 seconds, an ice dynamic time step of 0.1 seconds, and a coupling time interval of 60 seconds.

The models were calibrated to water levels measured by the deployed instruments over the freeze-up period, and surveyed final top-of-ice elevations. Table 4.10 lists the calibrated parameter sets for each model. Several parameters were assigned values reported in the literature based on previous applications of numerical river ice models (see Table 1.1). These include the roughness of the underside of the ice cover, the porosity of the rubble accumulation, the heat transfer coefficient at the ice-air interface, the albedo of the ice cover, and the seepage flow coefficient. The main calibration parameter for the base model was the internal friction angle of the ice rubble, which essentially affects the final ice cover thickness and staging. The value of 40° was found to produce a good match between simulated and measured water levels and ice elevations when used in conjunction with ice roughness for an accumulation of ice floes proposed by Nezhikhovskiy (1964) (see Figure 2.33). This value is in agreement with the range of values documented by White (1999) (see Table 1.1). The thermal model was prescribed the same friction angle of 40° , and the calibration focussed on the critical freezing velocity, critical crust thickness, and crust failure stress. The latter was found to have the most influence on the simulation result; a value of 0.3 MPa produced the best match to field measurements in 2016-2017.

Table 4.10. Parameters used in full dynamic models.

Parameter	Description	Base Model	Thermal Model
ϕ	internal friction angle of ice rubble	40°	40°
v_{crit}	critical freezing velocity	N/A	0.05 m/s
h_{crit}	critical crust thickness	N/A	0.001 m
σ_{crust}	crust failure stress	N/A	0.3 MPa
n_i	Manning roughness of ice cover	Nezhikhovsky method	Nezhikhovsky method
p_j	porosity of ice rubble	0.4	0.4
α_i	albedo of ice cover	0.3	0.3
λ	seepage flow coefficient	0.2 m/s	0.2 m/s
H_{ia}	linearized heat transfer coefficient at ice-air interface	12.2 W/m ² /°C	12.2 W/m ² /°C

4.5.6.2 Simulated Ice Profiles and Water Levels

The final simulated ice profiles along the channel centreline and surveyed top-of-ice elevations are shown in Figure 4.34. Estimated bottom-of-ice elevations are also shown at sites DRLL05, DRLL05a, and DRLL06; these were calculated by subtracting the estimated ice thickness (see Chapter 2) from the peak ice elevation. The peak and end-of-season water levels at each gauge location are also shown. In general, the ice profile of the thermal model approaches that of the base model in areas where the rate of strength addition was insufficient to prevent full mechanical thickening from occurring (i.e. final ice thickness based on rubble strength alone). The most notable difference between the simulated ice profiles is within 5 km of Lake Winnipeg, and upstream of site DRLL04a (10 km from Lake Winnipeg). In both of these areas, the thermal model top-of-ice profile is lower than the base model, and more closely matches the surveyed top-of-ice profile. This is because the rate at which strength was added to the ice cover from the growth of the surface crust was sufficient to allow the ice cover to progress upstream as a thinner accumulation.

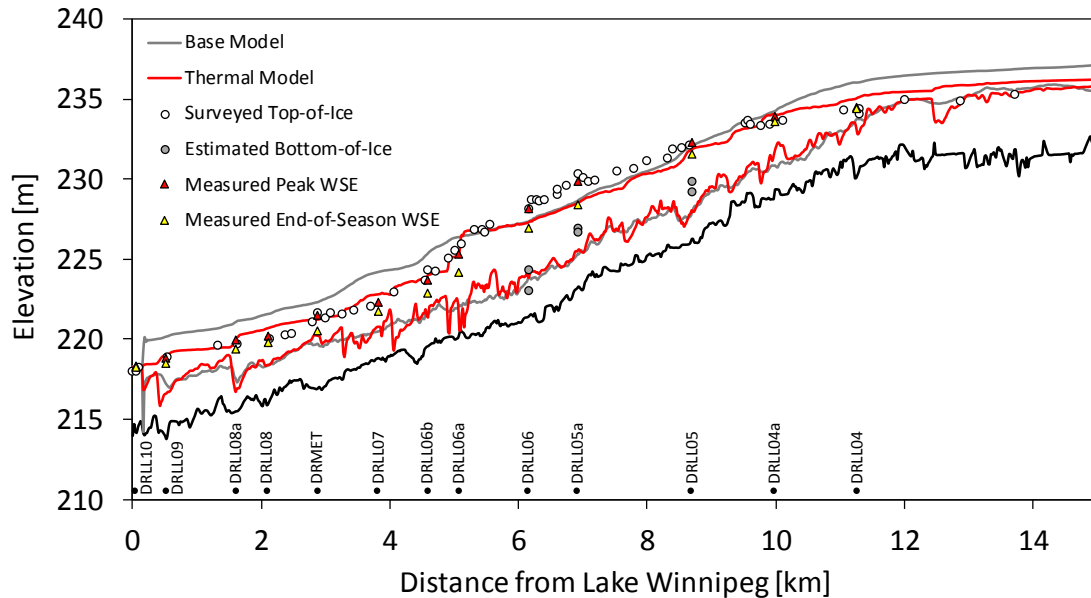


Figure 4.34. Simulated ice profiles for 2016-2017 freeze-up period.

Approximately 4.5 km from Lake Winnipeg, the thermal model shows a distinct toe region where the ice cover becomes much thicker in the upstream direction. The combination of crust strength and frictional effects of the bank and bed prevented the cover downstream from thickening. The surveyed top-of-ice profile also shows a distinct rise near the same location. From km 6 to km 10, the ice profile simulated with the thermal model closely resembles that of the base model. The rate of strength addition in the thermal model was insufficient to prevent mechanical thickening in this region. This has less to do with temperature fluctuations which changed the rate of growth of the surface crust (the air temperature was consistently below -15°C), and instead was likely caused by the higher rate at which the applied stresses increased as the cover lengthened due to the higher bed slope. The thermal model simulated small, frequent consolidations as incoming ice was continually added to the cover. The occurrence of major consolidations was not captured well by the model. In other words, the model predicted that the rate of strength gained by

the ice cover was not sufficient to prevent mechanical thickening, but the progression of a thin cover upstream with intermittent dramatic collapses was not captured. Both models produced a top-of-ice profile that was slightly lower than the surveyed top-of-ice profile from km 5 to km 10.

The final ice thickness simulated by the thermal and base models are shown in Figure 4.35. Several toe regions are visible in the thermal model result, while the ice thickness varies more gradually in the base model result. Both models were able to simulate the field estimated ice thicknesses reasonably well, as shown by the bottom-of-ice estimations in Figure 4.34 and the comparison with field deduced values in Table 4.11. As there is some variation in simulated ice thickness across the width of the river (see Figure 4.35), the values reported in Table 4.11 were calculated by taking an average ice thickness across the main channel at the site location. At DRLL06, the ice thicknesses simulated by the thermal and base models were 4.2 m and 4.1 m, respectively, both of which were within the range estimated in the field. At site DRLL05a, the ice thickness simulated by the thermal model was within 0.1 m of the field estimated value, while the base model was within 0.2 m. At DRLL05, the thermal model predicted a small toe region with increased thickness, and was 0.5 m higher than the field estimated range. The base model simulated an ice thickness of 4.0 m at site DRLL05, which is 0.8 m higher than the range estimated in the field.

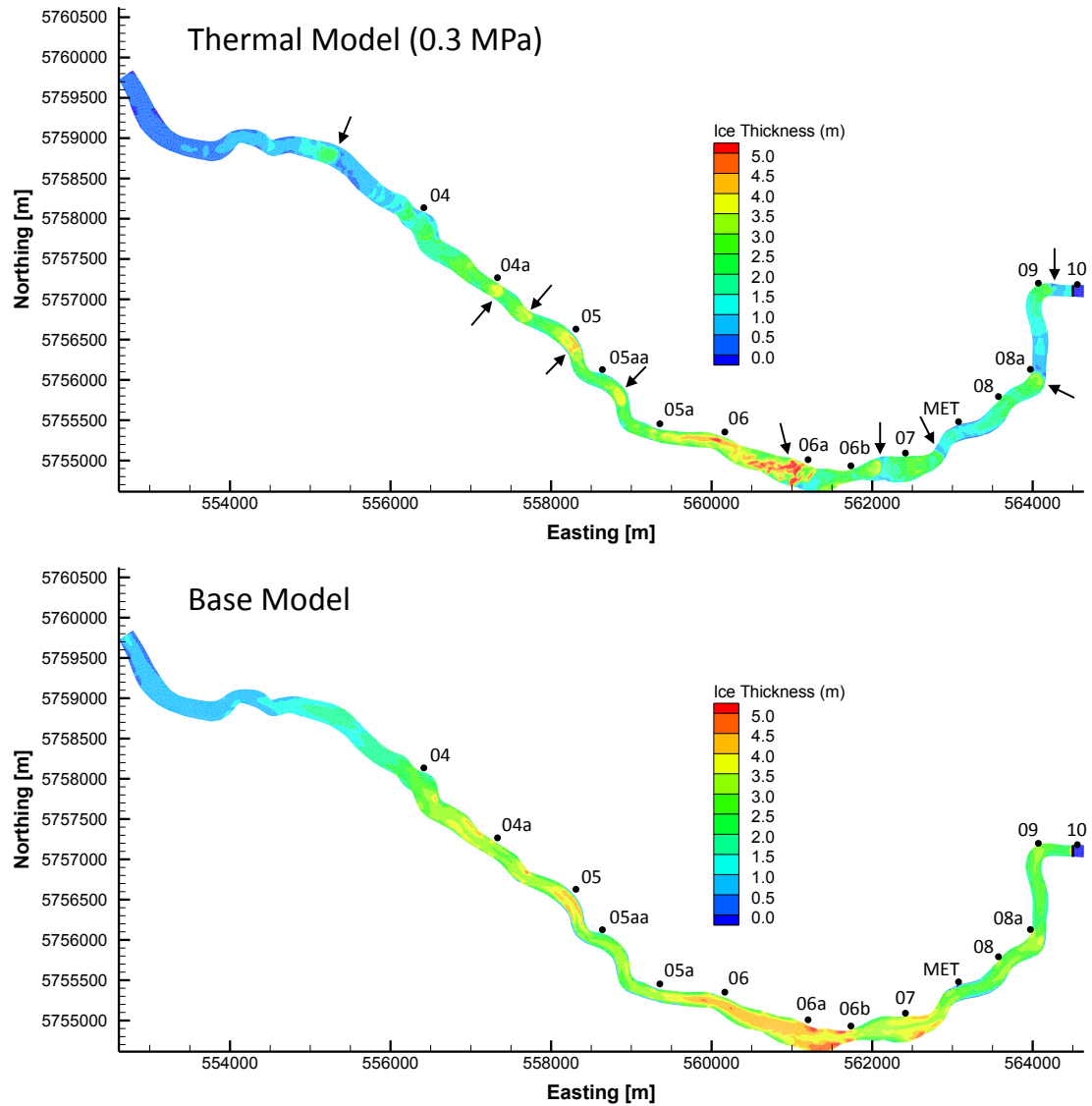


Figure 4.35. Simulated final ice thickness in 2016-2017. Arrows indicate toe regions of consolidated ice covers in the thermal model.

Table 4.11. Simulated and observed ice thicknesses in 2016-2017 season.

Location	Measured Range [m]	Thermal Model		Base Model	
		Simulated [m]	Difference [m]	Simulated [m]	Difference [m]
DRLL05	2.5-3.2	3.7	+0.5	4.0	+0.8
DRLL05a	3.6-3.9	3.5	-0.1	3.4	-0.2
DRLL06	4.0-5.3	4.2	in range	4.1	in range

The variation in solid crust thickness simulated by the thermal model at the time the ice cover progressed to the upstream boundary (approximately hour 144 of the simulation) is shown in Figure 4.36. The crust thickness is related to the length of time the cover has been stable at a given location; thus, the crust is thickest at the downstream end (approximately 0.5 m thick), and gets thinner in the upstream direction.

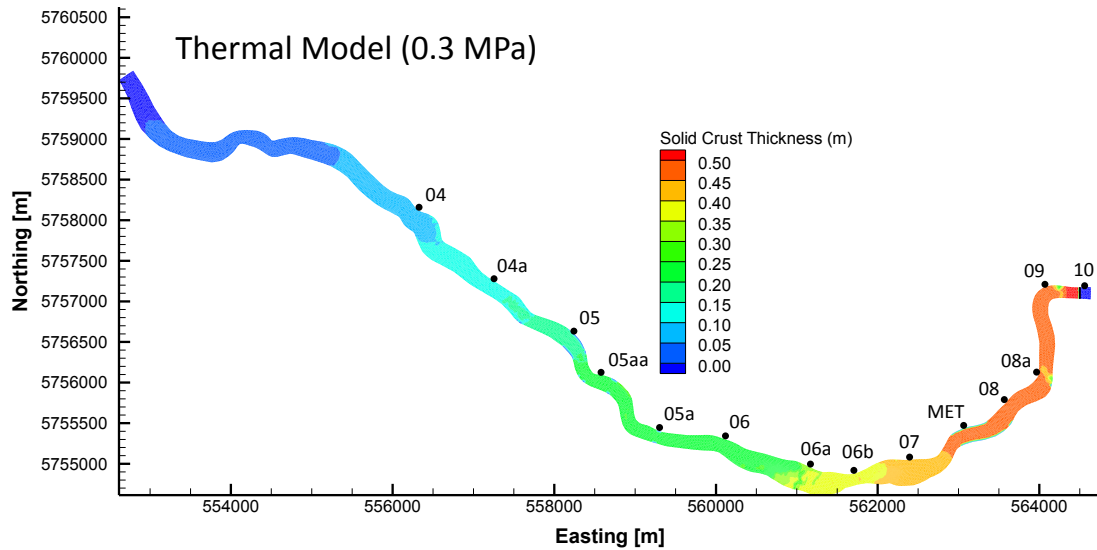


Figure 4.36. Solid crust thickness at hour 144 simulated by the thermal model for 2016-2017 freeze-up period.

Simulated and observed water levels for the thermal and base models are shown in Figure 4.37 and Figure 4.38, respectively. The timing of ice cover progression at each site, in terms of first occurrence of water level rise to the time when the cover is stable and the water level plateaus, is better simulated by the thermal model. For example, the water level at DRLL08 increases within a few hours of the start of the simulation for both models. In the thermal model, the water level plateaus near hour 25 of the simulation when the crust grows sufficiently to resist further mechanical thickening of the ice cover. The measured water level at DRLL08 stabilizes even earlier, approximately 20 hours from the start of the

simulation. In the base model, the water level at DRLL08 continues to rise gradually (as the ice cover continues to slowly thicken as the cover lengthens upstream) until approximately hour 60 when it plateaus. One potential solution to allow the cover to progress faster in the base model would be to increase the friction angle of the ice rubble. However, this would cause the ice cover to progress at a reduced thickness not only in this region, but in the upstream reach as well. This would result in a lower ice profile which is even lower than the observed values near km 7, for example. The rate of advancement of the ice cover from Lake Winnipeg to site DRLL04 is simulated reasonably well by both models, with the thermal model simulating a higher rate of advancement than the base model.

The observed water levels show several fluctuations as the ice cover progressed and shoved during the freeze-up period. The thermal model was able to capture this process reasonably well. Video of the ice cover progression in the numerical simulation indicated that the cover would experience distinct small shoves near the leading edge, and less frequent larger shoves that caused mechanical thickening over a larger reach. The water levels simulated by the thermal model show fluctuations caused by these shove events. For example, the water level at DRLL04a shows several sudden drops as the ice front progresses past the site. The water level subsequently shows an upward step which was caused by a consolidation that thickened the cover just downstream of the site. Although the magnitudes of water level fluctuations measured in the field were not exactly captured (i.e. the sharp rise of 2.5 m was not simulated at DRLL06), the thermal model was able to simulate the general process of frontal progression and shoving reasonably well. The base model did not show any fluctuations related to shoving events; as mentioned previously

the treatment of ice dynamics results in the gradual thickening of the ice cover and corresponding changes to water levels.

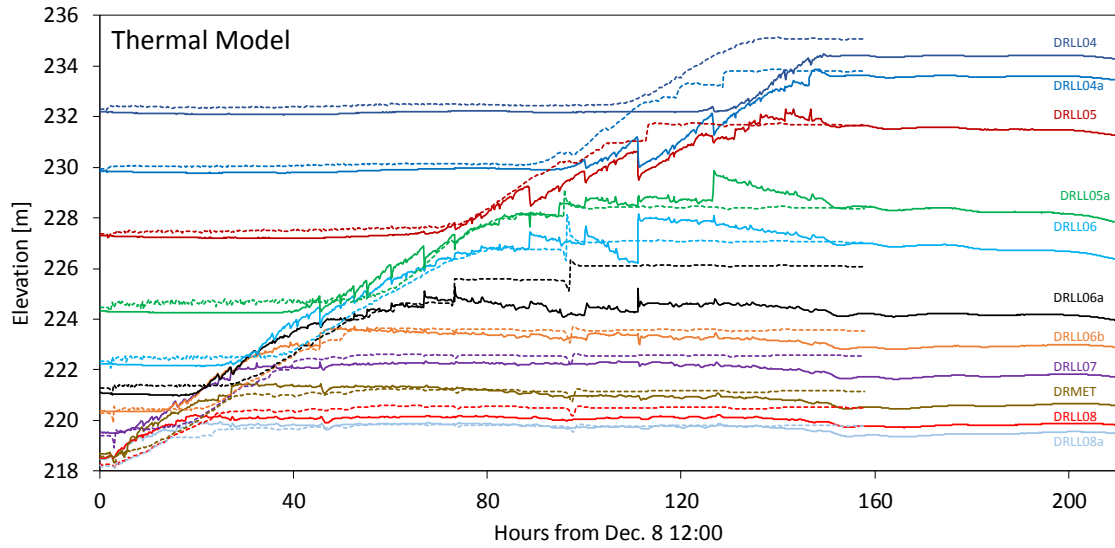


Figure 4.37. Water levels measured (solid lines) and simulated with thermal model (dashed lines) for freeze-up 2016-2017.

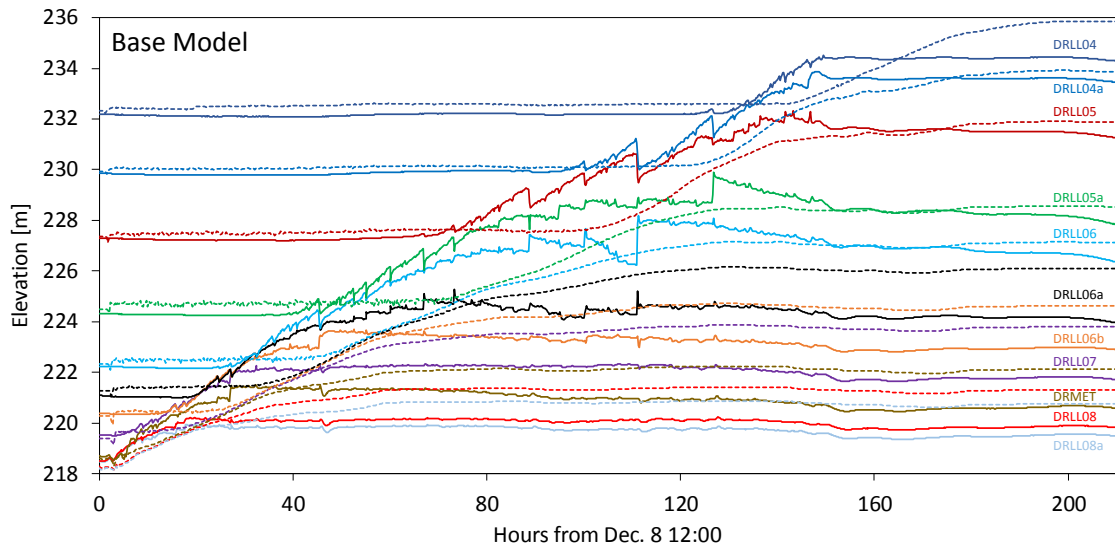


Figure 4.38. Water levels measured (solid lines) and simulated with base model (dashed lines) for freeze-up 2016-2017.

Simulated and measured peak water levels are summarized in Table 4.12. As shown, the thermal model outperforms the base model, and is able to more accurately simulate the range of peak water levels over the entire reach. The worst match of the thermal model occurred at DRLL06a, where the simulated peak water level was 1.08 m higher than measured. The peak water levels simulated by the base model were over 1.0 m different than measured at several sites. The mean absolute errors of the peak water levels simulated by the thermal and base models were 0.33 m and 0.89 m, respectively.

Table 4.12. Measured and simulated peak water levels for 2016-2017 freeze-up.

Location	Measured Peak WSE [m]	Thermal Model		Base Model	
		Simulated Peak WSE [m]	Difference [m]	Simulated Peak WSE [m]	Difference [m]
DRLL04	234.51	235.14	+0.64	235.88	+1.37
DRLL04a	233.87	233.88	0.00	233.92	+0.05
DRLL05	232.31	231.75	-0.56	231.91	-0.40
DRLL05a	229.89	229.13	-0.76	228.57	-1.31
DRLL06	228.17	228.14	-0.04	227.16	-1.02
DRLL06a	225.27	226.35	+1.08	226.17	+0.89
DRLL06b	223.71	223.72	+0.01	224.71	+1.00
DRLL07	222.34	222.66	+0.32	223.87	+1.53
DRMET	221.52	221.26	-0.26	222.24	+0.72
DRLL08	220.23	220.58	+0.35	221.42	+1.19
DRLL08a	219.95	219.86	-0.09	220.87	+0.92
DRLL09	218.80	218.96	+0.15	219.99	+1.19
DRLL10	218.37	218.34	-0.03	218.34	-0.03
mean absolute error [m]			0.33		0.89

Simulated and measured end-of-season water levels are summarized in Table 4.13. The measured end-of-season water level was taken at December 15, 2016 4:00 am (160 hours from start of simulation), when the ice cover stabilized at DRLL04. Note that the measured end-of-season water levels are as much as 1.5 m lower than the peak levels (e.g. at DRLL05a); this is likely caused by the reduction ice cover roughness with time. Shortly

after consolidation events, the thickened ice cover is extremely rough with large ice pieces protruding into the flow. Over time, these protrusions are smoothed out, causing water levels to decrease. This process is not included in the current CRISSP2D formulation. Near Lake Winnipeg, the end-of-season water levels simulated by the thermal model are lower than the base model, and more closely match the observed water levels. Both models perform satisfactorily at sites DRLL06, DRLL05a, DRLL05, and DRLL04a; the simulated end-of-season water levels were within 0.30 m of the field measurements. Both models over predict the end-of-season water level at DRLL04, although the thermal model achieved a better match than the base model. The mean absolute errors for the simulated end-of-season water levels were 0.50 m and 1.07 m for the thermal and base models, respectively.

Table 4.13. Measured and simulated end-of-season water levels for 2016-2017 freeze-up.

Location	Measured WSE [m]	Thermal Model		Base Model	
		Simulated WSE [m]	Difference [m]	Simulated WSE [m]	Difference [m]
DRLL04	234.38	235.08	0.70	235.88	1.50
DRLL04a	233.59	233.80	0.21	233.89	0.30
DRLL05	231.58	231.67	0.08	231.87	0.28
DRLL05a	228.42	228.38	-0.04	228.54	0.12
DRLL06	226.97	227.04	0.07	227.11	0.14
DRLL06a	224.20	226.09	1.88	226.09	1.89
DRLL06b	222.88	223.54	0.66	224.62	1.74
DRLL07	221.72	222.55	0.82	223.79	2.06
DRMET	220.59	221.13	0.60	222.13	1.55
DRLL08	219.78	220.49	0.71	221.32	1.54
DRLL08a	219.42	219.77	0.35	220.77	1.35
DRLL09	218.49	218.81	0.32	219.91	1.42
DRLL10	218.24	218.21	-0.03	218.28	0.04
mean absolute error [m]			0.50		1.07

4.5.6.3 Discussion of Model Results

It is clear from the model results that the base model could not accurately replicate the ice profile and water levels along the entire reach. The base model performed best in the areas that were observed to experience consolidations that mechanically thickened the ice cover during the freeze-up period. However, the top-of-ice elevations and water levels simulated by the base model were much higher than observed in the more gently sloped portions of the river near Lake Winnipeg and upstream of DRLL04. A single value of the internal friction angle would not allow the base model to match observed ice elevations along the entire Lower Dauphin River; increasing the friction angle would give a better match near the lake and upstream of DRLL04, but the ice elevations would also be lower from km 6 to km 10. Conversely, if the friction angle were decreased, the model would achieve a better match from km 6 to km 10, but would further over-estimate the ice and water elevations near the lake and upstream of DRLL04. That said, the base model still performed reasonably well, and could be useful in providing a conservative estimate for peak ice elevations plausible over the reach. Numerical models are often calibrated to much fewer field data. For example, a calibration dataset may include a dozen ice elevations and/or water surface elevations at the end of the freeze-up period in some key areas that may be prone to flooding. It is then possible for the modeler to obtain a useful result by calibrating the model (changing friction angle and ice roughness) to match the few observations. The key takeaway, though, is that there is a fundamental process missing from the base model that is likely, in part, why the model cannot achieve a good match to field data over the entire reach.

Incorporating the effects of thermal processes on the resistance of a forming ice cover to mechanical thickening, albeit with a simplified method, allowed the thermal model to achieve a good match to field observations over the entire length of the Lower Dauphin River. The thermal model was able to predict the more rapid advancement of the ice cover near Lake Winnipeg and the slower advancement, involving more mechanical thickening, from site DRLL06b to DRLL04a. The final ice profile shows several toe regions indicating the downstream extent of consolidations. The effects of ice cover shoves are also seen in the water level time series, although not to the same magnitude as in the field data. In general, the thermal model was able to simulate the time variation in water levels and final ice elevations and thicknesses better than the base model. Additionally, the simulated ice cover exhibited the processes of progression and shoving that were observed in the field, that were not captured by the base model. The revised model incorporates a process that is known to be occurring and undoubtedly has an impact on the formation of the ice cover at freeze-up. It is necessary to consider these processes for a variety of numerical modeling applications, such as the simulation of peak water levels at freeze-up, or predicting the safe range of a hydro-peaking operation to avoid consolidation of a downstream ice cover.

4.5.6.4 Uncertainty in Model Parameters

There are a number of parameters that are used in CRISSP2D (in addition to those added to model the thermal strengthening of the ice cover) that are not known to a high level of certainty. These will be discussed in this section, along with how they may influence the simulated ice cover. Due to the long run times of the full dynamics model, a full sensitivity analysis was not performed.

The Manning roughness of the underside of the ice cover is linked to a number of processes. A rougher cover will be subjected to higher shear forces from water flowing beneath the floating ice. This will also increase resistance to flow, causing greater staging of upstream water levels. In the current application of the CRISSP2D model, the ice roughness is assumed to vary with thickness based on the measurements conducted by Nezhikhovskiy (1964). Thinner ice covers at freeze-up are typically formed from juxtaposition and minor packing of ice floes at the water surface, with the floes oriented more or less with their long axis parallel to the flow of water. Therefore, the roughness of the cover is relatively small. As the ice cover grows a thermal crust and later consolidates, the composition of the cover changes to large, competent, irregularly shaped blocks of ice that are arranged in a disorderly fashion and can protrude out into the flow, as shown in Figure 4.39.



Figure 4.39. Rough, consolidated ice cover at site DRL05a on December 14, 2016.

The form drag induced by these protrusions increases the effective roughness of the upper boundary on the flow, and therefore a larger Manning roughness would be expected. However, as time passes, the processes of preferential melting and freezing, erosion of rubble protruding into the flow, and deposition of suspended ice in the void spaces smooth the undersurface and reduce the effective Manning roughness. The latter process of ice smoothening is not considered in the CRISSP2D model. It is not known whether the roughness variation proposed by Nezhikhovskiy (1964), which plateaus at $n_i = 0.10$ at thicknesses near 3.2 m (see Figure 2.33), is representative of the roughness variation in ice covers that undergo consolidations as observed on the Dauphin River. In fact, increasing the ice roughness in the reach that obtained the thickest ice cover (from km 5 to km 10) would have increased the simulated ice elevations to more closely match surveyed elevations. Thus, it is possible that the treatment of ice roughness should be modified for consolidated ice covers. In an application of the one dimensional model, RIVICE, to the Lower Dauphin River, Lindenschmidt et al. (2012) used the data of Nezhikhovskiy (1964) for dense slush, which ranges from 0.04 at a thickness of 1.0 m to 0.08 at a thickness of 3.0 m. Lindenschmidt et al. (2012) noted that the roughness could be substantially more for a freeze-up ice jam composed of competent ice floes. Gerard and Andres (1982) performed under-ice velocity measurements on freeze-up ice accumulations, and concluded that the surface appearance and state of the ice cover (whether consolidated or not) provides a better indicator of ice roughness, and that the roughness may not be related to cover thickness as suggested by other investigators. Additional research is required to investigate this further and to reduce the uncertainty in this model input parameter.

Seepage flow through the void spaces of the rubble ice cover is not completely understood in the context of the Dauphin River. A porous accumulation of ice with higher seepage velocities would reduce the amount of flow that moves under the ice cover. This results in lower under-ice water velocities and reduced staging. Experiments and analysis by Beltaos and Wong (1986) indicated that the seepage flow is related to square root of the water surface gradient, the wetted area of the ice cover, and a seepage flow coefficient with dimensions of velocity (Beltaos, 1993). The seepage flow coefficient is a user input parameter in CRISSP2D. Extrapolation of lab experiment data by Beltaos and Wong (1986) gives a value of 0.7 m/s (for a porosity of 0.4), while field applications of a one-dimensional numerical model to breakup jams on the Restigouche River and Rushoon River indicated a range of 1.0-2.5 m/s (Beltaos, 1993). Another study by Beltaos (1999) on the Credit River indicated a range of 1.5-2.1 m/s. These studies involved breakup jams formed of competent ice blocks. It is plausible that the permeability of a freeze-up jam would be decreased due to frazil blockages between the rubble ice, thus lowering the seepage coefficient. It is also likely that the value of the seepage coefficient should change with time, as water erodes a path through grounded portions of the ice cover. The seepage parameter value of 0.2 m/s was found to give a good match of water levels and ice elevations in the calibrated model. The parameter has the most influence on the very thick portions of the ice cover, where the water surface gradient is large. Additional research on flow through freeze-up ice accumulations would decrease the uncertainty in this input parameter.

The porosity of the ice accumulation in the CRISSP2D model is assumed to be constant at 0.4, which is in agreement with other river ice numerical model applications.

Field observations roughly agree with this value, although no direct measurements were taken. There has not been a study on the accuracy of this commonly used value, or on how the porosity may change as the ice cover consolidates. Future research on this topic may reduce the uncertainty in this parameter.

The value of 40° for the internal friction angle of the ice rubble, ϕ , was found to produce satisfactory matches of simulated ice thickness to field measurements. This value is within the range reported in the literature, but is lower than the default value of 46° in CRISSP2D. One possible reason for this is the over thickening of the ice cover that occurs during consolidation events on this reach; a lower value of ϕ is required for the model to generate thicker ice accumulations as observed in the field.

Bank and bed friction coefficients are also unknown to some degree. The friction coefficient is the ratio of shear stress to normal stress applied at the surface. Bank and bed friction play an important role in stabilizing the ice cover; higher friction coefficients will reduce the amount of streamwise stress transferred to the ice cover downstream, effectively decreasing the equilibrium thickness, which is the maximum potential thickness of the ice cover. Both the bank and bed friction coefficients were given the value of 0.84, which is equal to $\tan \phi$, where $\phi = 40^\circ$, the friction angle of ice rubble. Field observations indicated that shear lines are often present at the banks, such that the central portion of the ice cover would shove while a portion near the bank would remain stationary for smaller shove events. Thus, the shear force at the bank is a result of ice-ice friction, so the selected value of $\tan \phi$ should be appropriate. However, during larger consolidation events, the entire cover may mobilize, and some ice rubble would be pushed up against the tree line which could possibly increase the friction coefficient. Less is known about the bed friction

coefficient. The simulated ice cover often grounded near channel banks, but only grounded in the middle portion of the channel in a few locations, the most significant being near site DRLL06a, where the cover grounded at a higher island area (which gets flooded out during the ice covered period). Further research could reduce the uncertainty in these parameters and allow the model to more accurately simulate the effects of bank and bed friction on ice cover formation.

The surface albedo of the ice cover was prescribed a value of 0.3, which is the default value in CRISSP2D. The surface albedo of bare ice is typically near 0.52 (Perovich, 1996), while a fragmented ice cover has a surface albedo of approximately 0.08 (Prowse and Marsh, 1989). Albedo can also vary depending on snow coverage or decay of the ice cover (candling). A higher surface albedo will reflect more incoming solar radiation back to the atmosphere, thus allowing for higher rates of surface crust growth during sunny periods (compared to a surface with a low albedo). Shoving events were not strongly correlated with time of day on the Lower Dauphin River, thus the importance of this parameter is low and using the default value of 0.3 is likely sufficient. In cases where sunny days can trigger consolidation events, the surface albedo parameter may require more attention.

The value of $12.2 \text{ W/m}^2/\text{°C}$ was used for H_{ia} , which is the default value in CRISSP2D. Note that this is similar to the value calculated in the laboratory experiments of Chapter 3. This parameter will influence the rate at which heat is transferred from the ice cover to the cold air, thus affecting the rate of growth of the surface crust. Jasek (2006) calibrated a simple thermal ice growth Excel model to field measurements of crust thickness on the Peace River, and found that a heat transfer coefficient of $15 \text{ W/m}^2/\text{°C}$

produced good results. To obtain a better estimate of the site-specific value, ice thickness measurements are required at multiple times over the winter. However, these measurements were not taken on the rubble ice cover on the Lower Dauphin River due to safety concerns. Although the true value of H_{ia} is not known, the calibrated value of σ_{crust} would encompass some of the error in the prescribed value of H_{ia} .

4.5.7 Full Dynamic Model Application to 2015-2016 and 2017-2018 Freeze-up

Periods

4.5.7.1 Simulation Setup

The base and thermal models were applied to the 2015-2016 and 2017-2018 freeze-up periods to evaluate their transferability to seasons with different combinations of hydraulic and meteorological conditions. The major difference between these two seasons and the 2016-2017 season was the average air temperature and the length of time required for the ice cover to progress up the Lower Dauphin River (see Figure 2.6). An ice cover formed in under seven days in 2016-2017, while in 2015-2016 and 2017-2018 it took more than 30 days for the ice cover to advance approximately 12 km up the steeper lower reach. The average discharges during the periods of ice cover progression in 2015-2016 and 2017-2018 were 130 m³/s and 220 m³/s, respectively. The 2015-2016 simulation was started on November 25, 2015, and the 2017-2018 simulation was started on November 9, 2017. The start times coincided with the time at which the ice cover began progressing upstream from Lake Winnipeg. It was found that applying the crust strength of 0.3 MPa in the thermal models, which worked well for the 2016-2017 freeze-up period, resulted in the simulated ice cover advancing too quickly and attaining final ice elevations that were lower than observed for the 2015-2016 and 2017-2018 seasons. Thus, an additional thermal run was

performed for each season with a reduced crust strength of 0.1 MPa to assess the impact on the final ice profiles and water levels.

4.5.7.2 2015-2016: Simulated Ice Profiles and Water Levels

The final simulated ice profiles along the channel centreline and surveyed top-of-ice elevations for the 2015-2016 freeze-up period are shown in Figure 4.40. The peak and end-of-season water surface elevations at each working gauge are also shown. All three models simulated top-of-ice elevations that were lower than measured between DRLL08a and DRLL05. The thermal model with 0.3 MPa crust strength simulated the thinnest ice cover, with several thicker toe regions that produced a step-like profile. The model severely under predicted the ice elevations along most of the reach, but achieved a satisfactory match near Lake Winnipeg and upstream of DRLL04. The thermal model with 0.1 MPa crust strength simulated an ice profile that more closely resembled the base model along most of the reach. The base model simulated the thickest ice cover of the three models, but still under predicted the top-of-ice elevations at several locations, especially near site DRLL06. The high ice elevations surveyed in the field were likely a result of the consolidation event that occurred late in the freeze-up period, on December 20, 2015. This event was not captured in any of the models.

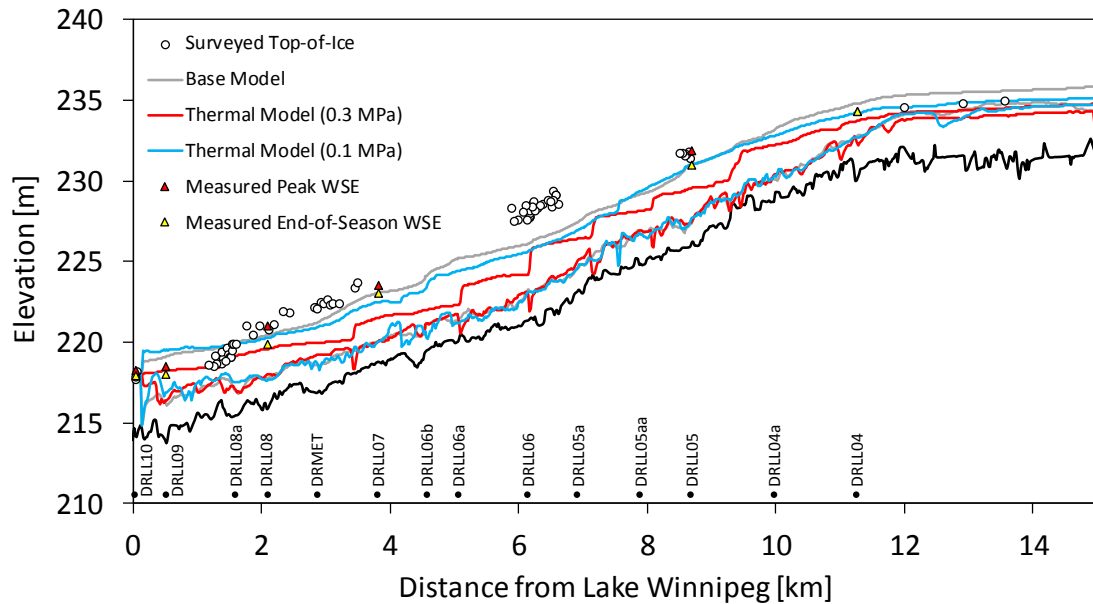


Figure 4.40. Simulated ice profiles for 2015-2016 freeze-up period.

The final ice cover thicknesses simulated for the 2015-2016 freeze-up period are shown in Figure 4.41. The thermal model with a crust strength of 0.3 MPa simulated the thinnest ice cover, with several toe regions where the cover mechanically thickened. Decreasing the crust strength to 0.1 MPa resulted in the simulation of a thicker ice cover, with fewer toe regions. The base model simulated the thickest cover, and the thickness changed most gradually of the three models. There were no field estimates of final ice cover thickness in 2015-2016 to compare to the simulation results.

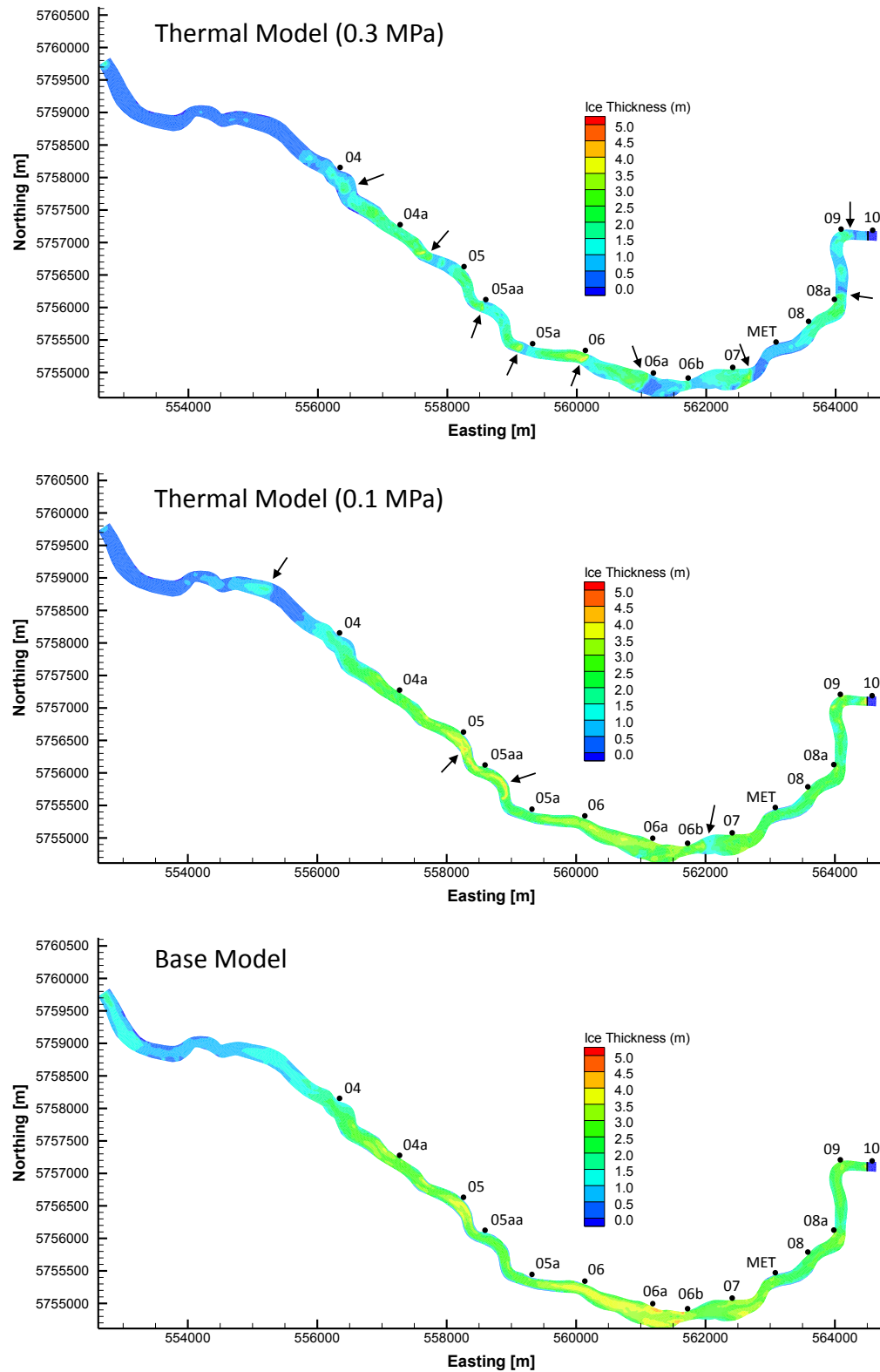


Figure 4.41. Simulated final ice thickness in 2015-2016. Arrows indicate toe regions of consolidated ice covers in the thermal models.

Simulated and observed water levels for the 2015-2016 freeze-up period are shown in Figure 4.42, Figure 4.43, and Figure 4.44 for the thermal models with 0.3 MPa and 0.1 MPa crust strengths, and base model, respectively. Model results were extracted at an hourly time step. The simulated water level lines end when the ice cover progressed to the upstream boundary of the model. As shown, the thermal model with 0.3 MPa crust strength simulated the most rapid advancement of the ice cover, indicated by the earlier rise in water levels at sites DRLL04 and DRLL05. One shove event at DRLL07 near hour 100 caused the water level to rise abruptly. In general, the water levels simulated by the thermal model with 0.3 MPa crust strength were lower than field measurements. The thermal model with 0.1 MPa crust strength achieved a better match of the timing of ice cover progression to DRLL05, but did not simulate the severe shove event at approximately hour 600. However, smaller shove events are evident in the simulated water level records at DRLL08 (near hour 250) and DRLL05 (near hour 620). Water levels simulated by the base model are the smoothest of the three models, and do not show any rapid steps from shove events. The timing of cover progression was simulated reasonably well. Both the thermal model with 0.1 MPa crust strength and the base model simulated final water levels that matched reasonably well with field measurements. Measured water levels decreased during the warm spell from hour 100 to 500. This was, in part, due to the decreasing flow, but also from the development of open water leads and smoothening of the ice cover which increased flow. Details of this process were not well captured by any of the models.

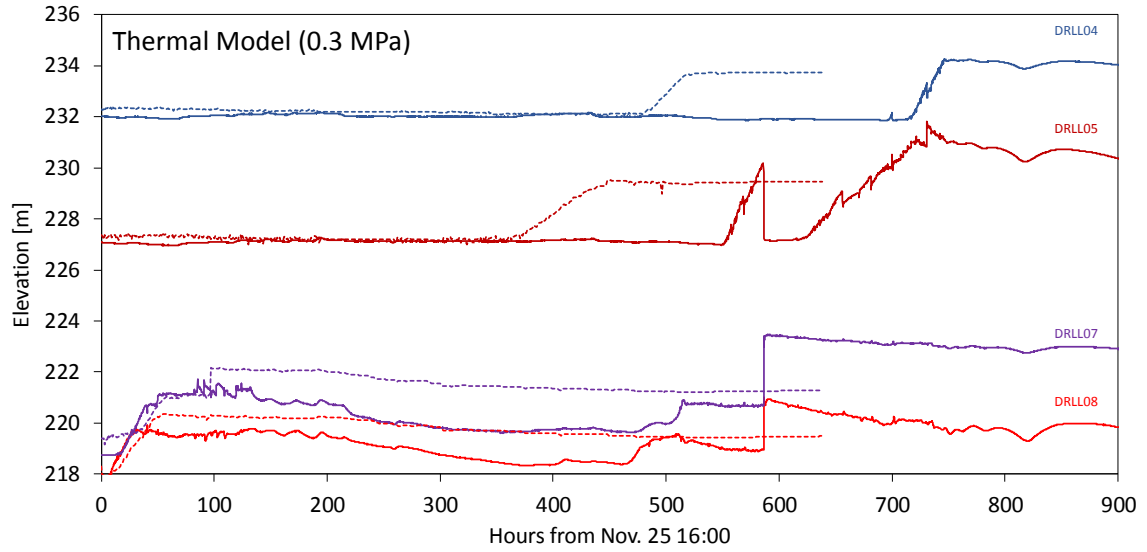


Figure 4.42. Water levels measured (solid lines) and simulated with thermal model with 0.3 MPa crust strength (dashed lines) for freeze-up 2015-2016.

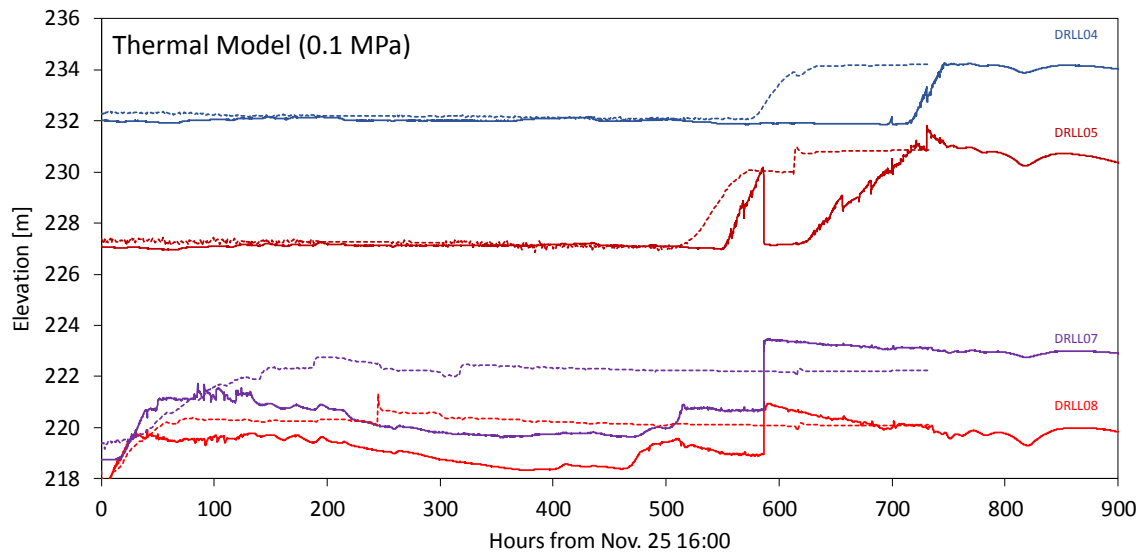


Figure 4.43. Water levels measured (solid lines) and simulated with thermal model with 0.1 MPa crust strength (dashed lines) for freeze-up 2015-2016.

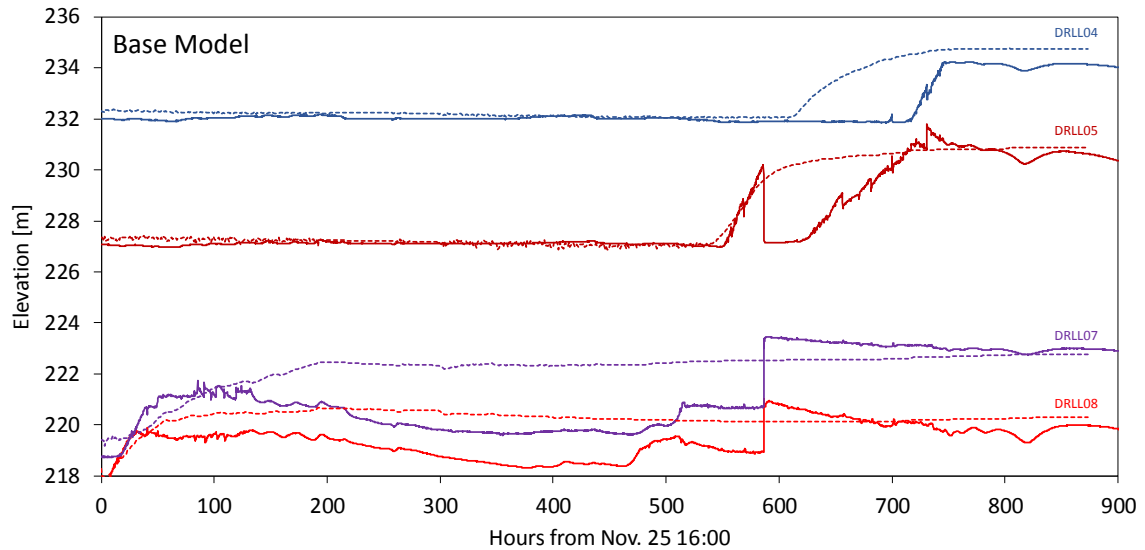


Figure 4.44. Water levels measured (solid lines) and simulated with base model (dashed lines) for freeze-up 2015-2016.

Simulated and measured peak water levels for the 2015-2016 freeze-up season are summarized in Table 4.14. As shown, the base and thermal model with 0.1 MPa crust strength achieved similar mean absolute errors of 0.55 m and 0.57 m, respectively. Both of these models achieved the worst match at DRLL09, which is near the initiation of the cover at the lake. The thermal model with 0.3 MPa crust strength achieved an mean absolute error of 0.79 m, but under predicted the water level at all gauges upstream of DRLL09. The worst match occurred at DRLL05, where the model under predicted the peak water level by over two metres.

Table 4.14. Measured and simulated peak water levels for 2015-2016 freeze-up.

Location	Meas. [m]	Thermal Model (0.3 MPa)		Thermal Model (0.1 MPa)		Base Model	
		Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]
DRLL04	234.24	233.74	-0.50	234.21	-0.03	234.77	+0.53
DRLL05	231.81	229.53	-2.28	230.97	-0.84	230.90	-0.91
DRLL07	223.46	222.16	-1.30	222.77	-0.69	222.77	-0.69
DRLL08	221.01	220.37	-0.64	221.32	+0.31	220.68	-0.33
DRLL09	218.48	218.50	+0.02	220.03	+1.55	219.32	+0.84
DRLL10	218.18	218.17	-0.01	218.18	0.00	218.17	-0.01
mean absolute error [m]			0.79		0.57		0.55

Simulated and measured end-of-season water levels for the 2015-2016 freeze-up season are summarized in Table 4.15. The measured end-of-season water level was taken at December 27, 2015 18:00 (770 hours from start of simulation), when the ice cover stabilized at DRLL04. The simulated end-of-season water levels were extracted when the ice cover progressed to the upstream boundary of the model, which occurred at hour 734, 661, and 540 for the base model, and thermal models with 0.1 MPa and 0.3 MPa crust strength, respectively. The base and thermal model with 0.1 MPa crust strength achieved mean absolute errors of 0.38 m and 0.45 m, respectively, while the thermal model with 0.3 MPa crust strength achieved an mean absolute error of 0.78 m.

Table 4.15. Measured and simulated end-of-season water levels for 2015-2016 freeze-up.

Location	Meas. [m]	Thermal Model (0.3 MPa)		Thermal Model (0.1 MPa)		Base Model	
		Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]
DRLL04	234.24	233.73	-0.51	234.17	-0.07	234.67	+0.43
DRLL05	230.96	229.43	-1.53	230.82	-0.14	230.78	-0.18
DRLL07	223.01	221.23	-1.78	222.20	-0.81	222.67	-0.34
DRLL08	219.87	219.44	-0.43	220.09	+0.22	220.21	+0.34
DRLL09	217.94	218.19	+0.25	219.30	+1.36	218.86	+0.92
DRLL10	217.90	218.09	+0.19	217.99	+0.09	217.95	+0.05
mean absolute error [m]			0.78		0.45		0.38

4.5.7.3 2017-2018: Simulated Ice Profiles and Water Levels

Figure 4.40 shows the simulated ice profiles and surveyed ice elevations for the 2017-2018 freeze-up period. The peak and end-of-season water surface elevations at each working gauge are also shown. Once again, the thermal model with 0.3 MPa crust strength under predicted the ice elevations along most of the reach, but achieved a satisfactory match upstream of DRLL04a. The ice profile simulated by the thermal model with 0.1 MPa crust strength closely matches the ice profile of the base model. The ice cover simulated by the base model was the thickest of all the models. A good match to surveyed ice elevations was achieved from Lake Winnipeg to DRLL06. At this location, surveyed ice elevations show a steep rise. Field observations at DRLL06 indicated that a shove event late in the season pushed the ice accumulation out toward the banks and created an open water lead in the centre of the channel (see Figure 2.28 and Figure 2.29). The ice cover likely grounded in several locations while the open lead conveyed the majority of the flow. The complexities of this process were not captured by the model, which is likely why the models under predict the ice elevations between DRLL06 and DRLL05.

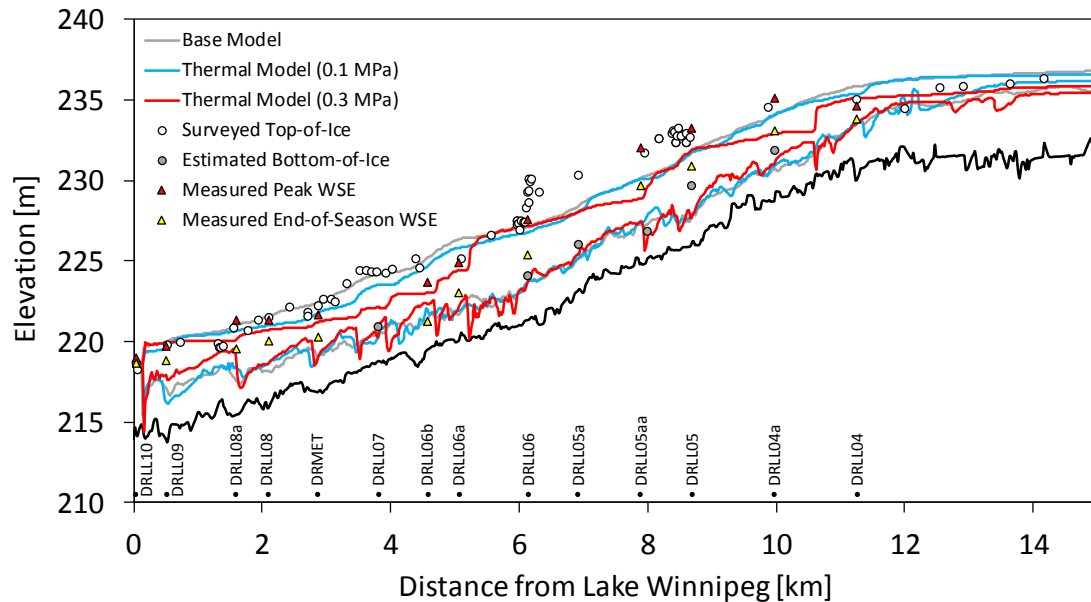


Figure 4.45. Simulated ice profiles for 2017-2018 freeze-up period.

The final ice cover thicknesses simulated for the 2017-2018 freeze-up period are shown in Figure 4.46. The thermal model with a crust strength of 0.3 MPa produced the thinnest ice cover over the majority of the domain, with the exception of the area near DRLL06a where the cover became very thick. There is a small island near this location toward the right bank of the river, which may have helped prevent the cover from consolidating further downstream. The thermal model with a crust strength of 0.1 MPa does not show as many distinct toe locations as the run with 0.3 MPa crust strength. The ice covers simulated by all three models are thickest in the region between DRLL06 and DRLL06b. Comparison of simulated ice thicknesses to field estimated values are summarized in Table 4.16. Simulated ice thicknesses were calculated by averaging the ice thickness across the main section of the channel at each monitoring site. The base model and thermal model with 0.1 MPa crust strength were able to simulate the ice cover thickness reasonably well over the entire length of the Lower Dauphin River, with the exception of

site DRLL05aa. The base model and thermal model with 0.1 MPa crust strength simulated ice thicknesses of 2.9 m and 2.2 m, respectively, while the field estimated range was 4.6-5.1 m. The thermal model with 0.3 MPa crust strength consistently under predicted ice thicknesses over the model domain, typically by 1.0-1.5 m, with the exception of sites DRLL05 and DRLL06, where the model simulated a toe region of a consolidation and more closely matched the field measurements.

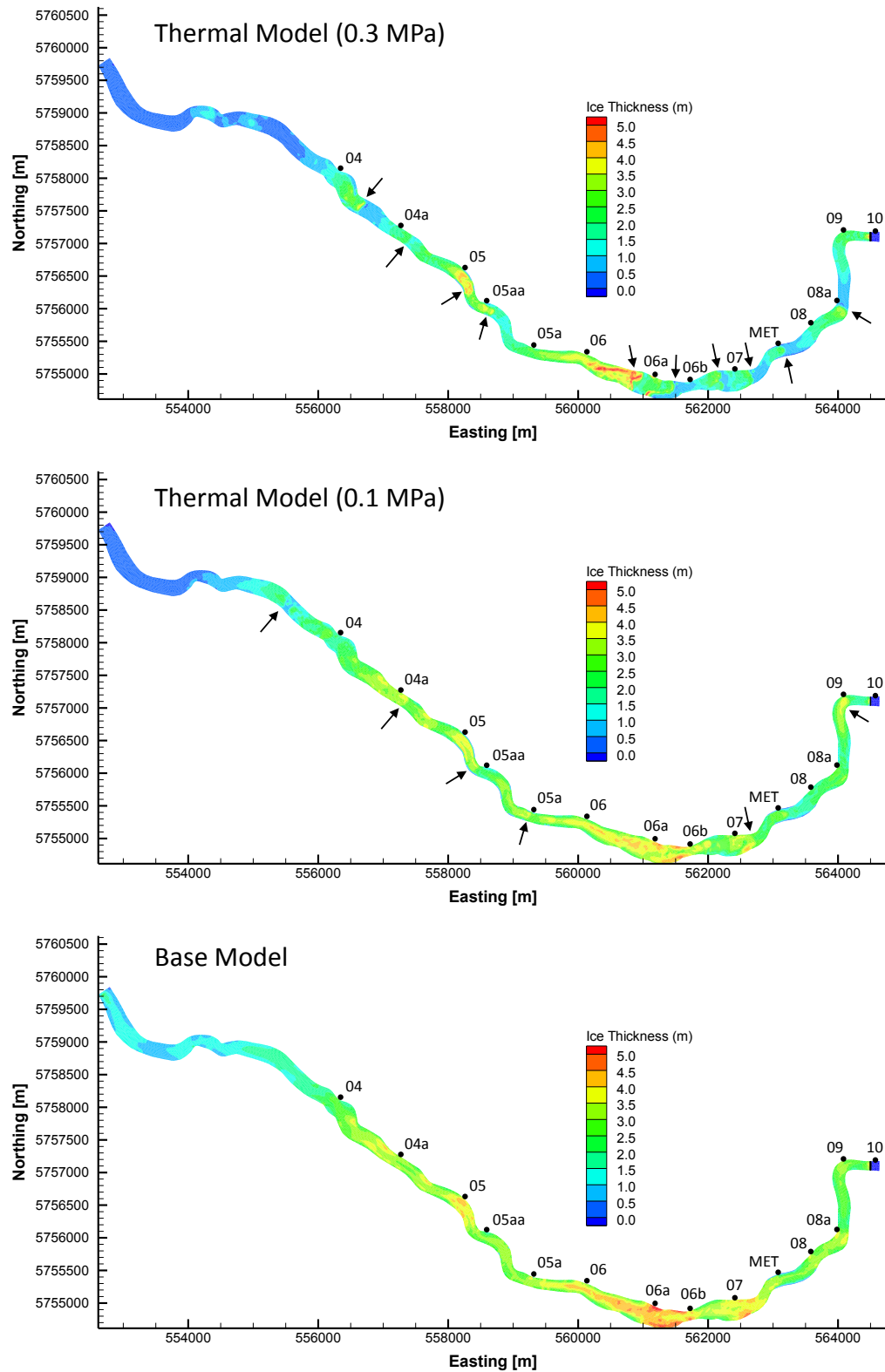


Figure 4.46. Simulated final ice thickness in 2017-2018. Arrows indicate toe regions of consolidated ice covers in the thermal models.

Table 4.16. Simulated and measured ice thicknesses in 2017-2018 season.

Location	Meas. [m]	Thermal Model (0.3 MPa)		Thermal Model (0.1 MPa)		Base Model	
		Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]
DRLL04	1.8-2.9	1.3	-0.5	2.7	in range	2.5	in range
DRLL04a	2.9-3.3	1.8	-1.1	3.2	in range	3.2	in range
DRLL05	2.7-3.4	3.9	+0.5	3.7	+0.3	4.1	+0.7
DRLL05aa	4.6-5.1	3.1	-1.5	2.2	-2.4	2.9	-1.7
DRLL05a	3.7-4.9	2.5	-1.2	2.8	-0.9	3.3	-0.4
DRLL06	3.8-5.5	3.5	-0.3	3.5	-0.3	3.9	in range
DRLL07	2.9-3.9	1.6	-1.3	3.4	in range	3.6	in range

Simulated and observed water levels for the 2017-2018 freeze-up period are shown in Figure 4.47, Figure 4.48, and Figure 4.49 for the thermal models with 0.3 MPa and 0.1 MPa crust strengths and base model, respectively. Model results were extracted at an hourly time step. The simulated water level lines end when the ice cover progressed to the upstream boundary of the model. In general, the thermal model with 0.3 MPa crust strength simulated faster progression of the ice cover and lower staging than field measurements. Shove events are evident in the abrupt changes in simulated water levels at DRLL05. The thermal model with 0.1 MPa crust strength simulated the timing of cover progression from DRLL10 to DRLL06 reasonably well (first 300 hours of simulation). However, the warm weather that followed halted cover progression in the field, while the model simulated continued cover advancement upstream. Trail camera images indicate that the surface ice concentration at DRLL03 was low during this period, with some periods of completely open water. The ice supplied at the upstream boundary from the free-drift model did not completely capture this reduction in surface ice, leading to the inaccurate simulation of cover progression in the full dynamic model. The base model simulated the timing of cover

progression well until hour 400, but also erroneously simulated continued progression during the warm spell from hour 400 to hour 600.

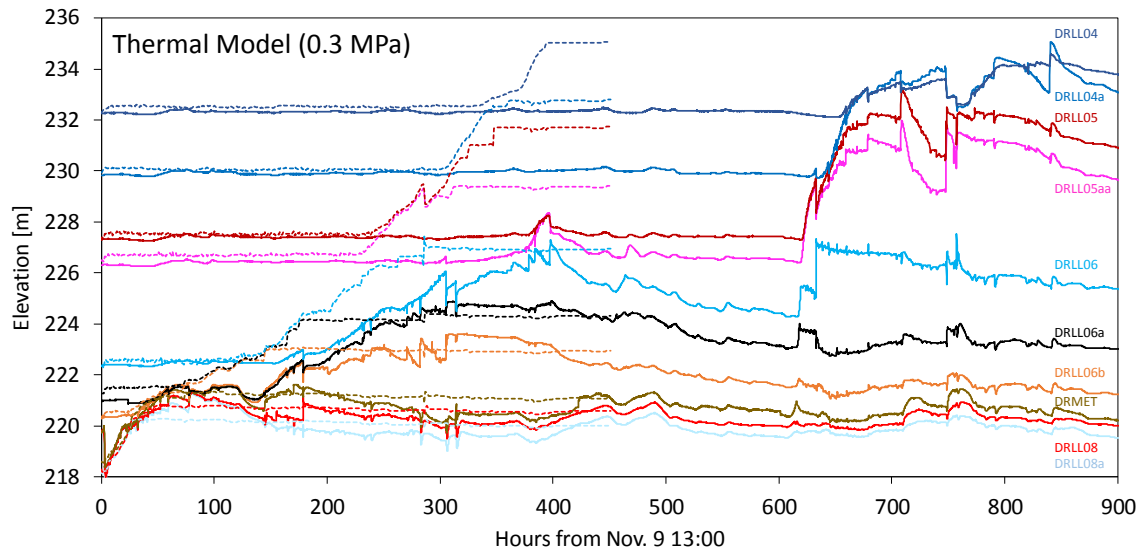


Figure 4.47. Water levels measured (solid lines) and simulated with thermal model with 0.3 MPa crust strength (dashed lines) for freeze-up 2017-2018.

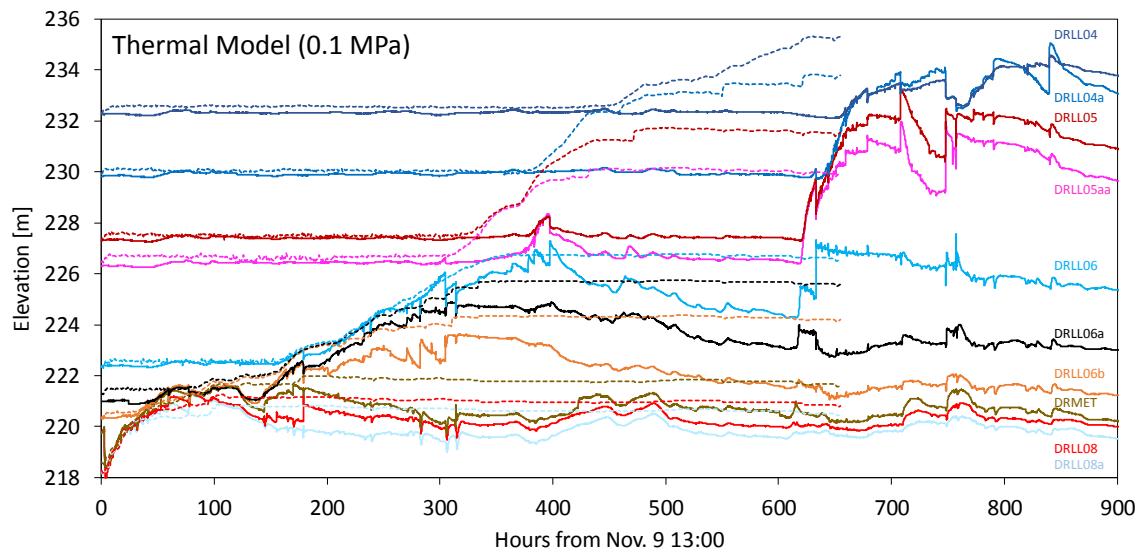


Figure 4.48. Water levels measured (solid lines) and simulated with thermal model with 0.1 MPa crust strength (dashed lines) for freeze-up 2017-2018.

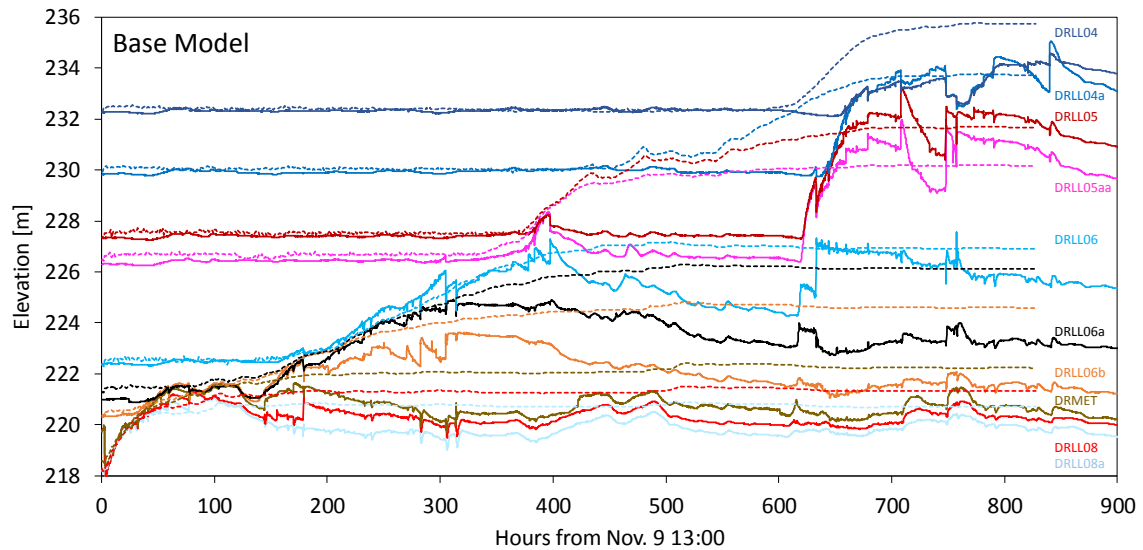


Figure 4.49. Water levels measured (solid lines) and simulated with base model (dashed lines) for freeze-up 2017-2018.

Simulated and measured peak water levels for the 2017-2018 freeze-up season are summarized in Table 4.17 (see also Figure 4.45). None of the models performed well over the entire length of the Lower Dauphin River. In general, the base model over predicted peak water levels from DRLL10 to DRLL06a, and under predicted levels from DRLL06a to DRLL04a. The thermal model with 0.1 MPa crust strength showed a similar pattern. The thermal model with crust strength of 0.3 MPa consistently under predicted peak water levels over the majority of the reach, with the exception of site DRLL04. The mean absolute errors in peak water levels simulated by the base model, and thermal models with 0.1 MPa and 0.3 MPa crust strengths were 0.89 m, 0.72 m, and 0.83 m, respectively.

Table 4.17. Measured and simulated peak water levels for 2017-2018 freeze-up.

Location	Meas. [m]	Thermal Model (0.3 MPa)		Thermal Model (0.1 MPa)		Base Model	
		Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]
DRLL04	234.57	235.07	+0.50	235.32	+0.75	235.77	+1.20
DRLL04a	235.07	232.79	-2.28	233.82	-1.25	233.77	-1.30
DRLL05	233.20	231.74	-1.46	231.74	-1.46	231.71	-1.49
DRLL05aa	231.97	229.43	-2.54	230.16	-1.81	230.20	-1.77
DRLL06	227.55	227.42	-0.13	226.79	-0.76	227.19	-0.36
DRLL06a	224.90	224.53	-0.37	225.75	+0.85	226.30	+1.40
DRLL06b	223.62	223.19	-0.43	224.37	+0.75	224.82	+1.20
DRMET	221.63	221.34	-0.29	221.99	+0.36	222.41	+0.78
DRLL08	221.30	220.83	-0.47	221.25	-0.05	221.52	+0.22
DRLL08a	221.26	220.28	-0.98	220.89	-0.37	220.93	-0.33
DRLL09	219.65	220.02	+0.37	219.80	+0.15	220.27	+0.62
DRLL10	218.95	218.86	-0.09	218.91	-0.04	218.92	-0.03
mean absolute error [m]			0.83		0.72		0.89

Simulated and measured end-of-season water levels for the 2017-2018 freeze-up season are summarized in Table 4.18. The measured end-of-season water level was taken on December 17, 2017 at 1:00 am (900 hours from start of simulation), when the ice cover stabilized at DRLL04. Figure 4.45 provides a good visual of how drastically the end-of-season water level differs from the peak water level at each site. In general, all three models over predicted the end-of-season levels along the river, owing to the absence of the simulation of open water leads and smoothening that occurred to the ice cover in reality. These processes may also affect the forces on the ice cover as the water surface slope gradually changes over time. The base model and thermal models with 0.1 MPa and 0.3 MPa crust strengths achieved mean absolute errors in end-of-season water levels of 1.45 m, 1.15 m, and 0.85 m, respectively.

Table 4.18. Measured and simulated end-of-season water levels for 2017-2018 freeze-up.

Location	Meas. [m]	Thermal Model (0.3 MPa)		Thermal Model (0.1 MPa)		Base Model	
		Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]	Sim. [m]	Diff. [m]
DRLL04	233.78	235.03	+1.25	235.32	+1.54	235.73	+1.95
DRLL04a	233.09	232.76	-0.33	233.80	+0.71	233.73	+0.64
DRLL05	230.91	231.71	+0.80	231.55	+0.64	231.66	+0.75
DRLL05aa	229.66	229.39	-0.27	229.96	+0.30	230.16	+0.50
DRLL06	225.37	226.93	+1.56	226.62	+1.25	226.90	+1.53
DRLL06a	223.01	224.30	+1.29	225.59	+2.58	226.11	+3.10
DRLL06b	221.23	222.94	+1.71	224.19	+2.96	224.59	+3.36
DRMET	220.22	221.08	+0.86	221.63	+1.41	222.24	+2.02
DRLL08	220.00	220.57	+0.57	220.82	+0.82	221.33	+1.33
DRLL08a	219.53	220.02	+0.49	220.47	+0.94	220.70	+1.17
DRLL09	218.80	219.77	+0.97	219.45	+0.65	219.77	+0.97
DRLL10	218.60	218.64	+0.04	218.59	-0.01	218.72	+0.12
mean absolute error [m]			0.85		1.15		1.45

4.5.7.4 Discussion of Model Results

Results show that the revised model formulations with a crust strength parameter of 0.3 MPa did not work well for the 2015-2016 and 2017-2018 freeze-up periods, which were milder than the 2016-2017 freeze-up period. The crust added too much resistance to the ice cover, causing it to progress upstream too quickly and not attain the same thickness as estimated in the field. Reducing the crust strength to 0.1 MPa improved the result, causing the simulated ice cover to mechanically thicken in more areas along the river. Good results were also obtained by the base model, which does not account for any thermal strengthening of the cover.

The simplistic criteria for the initiation (and persistence) of crust growth likely contributes to the poor transferability of the crust strength parameter between seasons in this case study. The velocity of an ice parcel will always decrease as the parcel approaches

the leading edge of the cover. Thus, parcels accumulating at the leading edge will always meet this criteria and begin growing a surface crust. The ability for the cover to progress upstream versus mechanically thicken will then depend on the rate of increase of the applied forces due to cover lengthening and the rate of crust growth. However, there are several additional factors that may affect the ability of a forming ice cover to grow a surface crust in the field that are not captured by the present model. For example, the initial formation of the crust would likely depend on relative movement of ice pieces that are added to the cover. If the ice floes are continuously rearranging, the solid crust may not as easily grow and remain intact. It is possible that a certain rate of heat transfer is required to initiate this process, and if the rate of heat transfer is too low, the crust will not form and increase the resistance of the ice cover. The seepage of water through the ice accumulation may also hinder the ability of the solid crust to form. In cases where the ice cover is not moving (or is moving very slowly), but the water surface gradient and seepage velocities are high, the solid crust may not form as easily. In terms of the direction of heat transfer, the 2016-2017 freeze-up season was simpler, as the cold air temperatures always resulted in energy transfer out of the ice cover. Thus, freezing was always occurring and the effects of surface melting or deterioration of the crust were not relevant. In 2015-2016 and 2017-2018, periods of mild air temperatures caused the direction of heat transfer to switch such that the ice cover was gaining energy from the atmosphere. The model calculates the associated reduction in crust thickness, but the melt is assumed to occur evenly and the reduction in strength would be related only to the decrease in crust thickness. It is possible that the crust would, in reality, melt unevenly and lose strength faster than what is predicted by the model. During periods of warmer weather, the seepage flow may deteriorate the

crust both thermally and mechanically, and reduce the overall resistance of the cover to consolidation. These processes were likely not as relevant in 2016-2017, but may have played a role in the crust formation in 2015-2016 and 2017-2018.

4.6 Conclusions and Contributions of this Work

In this chapter, an overview of the CRISSP2D model was presented, along with a detailed summary of the current treatment of ice dynamics. A new formulation was presented, which incorporates the effects of thermal processes, via growth of a surface crust, on the resistance of an ice cover to mechanical thickening. The proposed treatment considers ice velocity as a limiting factor that determines when the ice cover can begin to grow the surface crust. The crust is assigned a user-specified failure stress, that will likely require calibration to different sites due to the assumptions and simplifications regarding the crust growth. As the surface crust grows, the resistance of the ice cover to collapse and mechanical thickening increases. Similarly, warm spells can theoretically cause the surface crust to melt and trigger an ice cover collapse.

The revised formulations were shown to work at a conceptual level when applied to an idealized channel scenario. A forming river ice cover may be more prone to consolidation events or frontal progression, depending on the rate of cover lengthening, heat transfer from the ice cover to the atmosphere, channel properties such as bed slope, and the crust strength specified by the user. The revised model was applied to the 2016-2017 freeze-up period of the Lower Dauphin River, which exhibited very cold air temperatures. The revised model was able to better simulate the fast cover progression and low ice elevations in slower moving areas of the river, while still predicting mechanical

thickening in faster moving areas. A crust failure stress of 0.3 MPa was found to give a good match between simulation results and field measurements of water levels and ice thicknesses. The revised model also captured the shoving process as observed in the field, although not to the same magnitude. The base version of the model over predicted the ice thickness and water levels in the slower moving areas, while achieving a good result in the faster moving areas.

The models were then applied to the 2015-2016 and 2017-2018 freeze-up periods, which exhibited milder air temperatures than the 2016-2017 season and more complex ice processes resulting from the fact that heat was not continually lost from the river to the atmosphere. For example, open water leads were observed to develop at various sites in the field after the ice cover had formed and progressed beyond that site. The thermal model did not achieve as good of a match to field measurements in these seasons, which highlighted areas of future work and model development.

Modeling the time dependent change in strength of the ice cover causes some ancillary parameters to become more important; one example is the ice roughness. When simulating the formation and progression of an ice cover at freeze-up, the ice roughness will not only affect the final staging caused by an ice cover, but it will also affect the transient shear stresses acting on the cover due to flowing water and the ability of the cover to progress upstream or consolidate. As mentioned in Section 1.2, there is uncertainty associated with several modeling parameters (see Table 1.1). Parameters such as rubble friction angle, rubble porosity, and bank friction coefficient, among others, are all tied to the forces acting on the ice cover. In order to expect a model to replicate field

measurements from season to season (and site to site), these ancillary parameters must be known to a higher level of certainty.

Although the revised model was not able to achieve a perfect match with all three seasons of field measurements, the simulations show that incorporating the process of surface crust growth does, in fact, influence ice dynamics during the ice cover formation period, and can be useful in explaining several of the observations and measurements obtained in the field monitoring program. Additional field investigations and model applications will allow the current treatment to be further developed, but the framework is in place for the CRISSP2D model to incorporate the important, and previously missing, link between thermal and dynamic ice processes.

Chapter 5 Conclusions and Future Work

5.1 Summary and Conclusions

The formation of a river ice cover at freeze-up has several societal and engineering implications. In fast-moving rivers, the progression of an ice cover upstream is interrupted by consolidation events: the sudden collapse and mechanical thickening of the ice cover that occurs when the driving forces exceed the resistance of the ice cover. Mechanically thickened ice covers have a greater effect on water levels, and can lead to rapid flooding and threats to human safety. Thick, rough ice covers constrict the flow of water and limit the energy production of hydroelectric generating stations through the winter season. The resistance of a rubble ice cover to mechanical thickening is provided by frictional interlocking of the rubble. The ice cover will thicken such that the rubble accumulation can withstand the stresses in the streamwise direction, and transfer these stresses laterally to the channel banks. At freeze-up, cold air temperatures permit the growth of a surface ice crust that solidifies the rubble ice together and adds non-frictional resistance to mechanical thickening. If the rate at which the surface crust grows and adds strength to the cover is greater than the rate at which forces increase as the cover lengthens, the ice cover will advance upstream faster, and at a reduced thickness. However, if the resistance of the cover is at any point exceeded by the applied forces, the surface crust will break and

the cover will collapse. The attendant recession of the ice front downstream and mechanical thickening of the ice rubble causes rapid water level changes; water levels increase suddenly in the consolidated reach while levels decrease far upstream with the removal of the ice cover. It is important for engineers to be able to predict ice cover thicknesses and peak ice-affected water levels to design infrastructure and issue flood warnings. Numerical models continue to be the most efficient and cost effective methods to study the impacts of river ice. The link between thermal and dynamic processes is a critical consideration when simulating the formation of an ice cover at freeze-up.

Chapter 1 of this thesis presented a literature review, including theory on ice strength and formation processes, laboratory investigations on ice cover strength, field studies on ice cover consolidation events, and an overview of several state-of-the-art numerical river ice models.

Chapter 2 presented two papers related to the field monitoring program established on the Dauphin River in central Manitoba. The first paper outlined the methods of the monitoring program, and presented quantitative and qualitative data collected over three freeze-up seasons spanning from 2015 to 2018. Collected field data indicated that the ice cover at several locations in the fast-moving Lower Dauphin River (average slope of 0.16%) attained final thicknesses of 3-5 m. First hand observations revealed that the ice cover would undergo small, frequent consolidations near the stationary front (period of approximately 20-30 minutes) as the cover lengthened. The area near the stationary front was termed the “actively consolidating zone.” The ice cover mechanically thickened more aggressively and abruptly during discrete “secondary consolidation” events that occurred less frequently (period of approximately 8 hours or more). Water level records captured

the dynamic nature of these events; levels within the consolidated reach could rise as much as 2.5 m over a span of only 8 minutes (0.32 m/min). Upstream water levels dropped by as much as 2.9 m in approximately 40 minutes, while gauges downstream captured the passage of the surge waves released during the consolidation events. The high spatial and temporal resolution of measured water levels highlighted the link between dynamic ice events and unsteady flow hydraulics during the freeze-up period. Peak ice-affected staging was typically over 3 m at most monitoring sites along the Lower Dauphin River, and could be as high as 5.7 m at locations that formed a rough, thick ice cover.

All together, the field research was extremely successful, and is a major contribution to the field of river ice engineering. Field data related to river ice processes is relatively sparse, but is critically important in advancing the understanding of the complexities of river ice. Datasets and observations such as those produced by this research test the limits of the current formulations of river ice processes, many of which are based upon seminal research conducted many decades ago. Inabilities of the current approaches to emulate field measurements highlight areas for improvement, and push the science forward. The data and observations in this research constitute one of the most detailed and complete freeze-up monitoring programs to date. Arduous efforts in installing equipment in a natural river environment and working in freezing conditions during the period of ice cover formation proved fruitful, resulting in a better understanding of the processes of ice cover progression and consolidation on fast flowing rivers such as the Lower Dauphin River.

The second paper of Chapter 2 focussed on the analysis of surge waves recorded by water level gauges during the 2016-2017 freeze-up period, which originated from the

release of water from storage during the collapse and consolidation of the ice cover. Collected field data facilitated the application of the Rising Limb Analysis Method to determine unsteady flow hydraulics during the passage of the surge waves. Near the toe of the consolidated reach, the water level was measured to rise over 0.4 m on several occasions, and as much as 0.93 m during one event. The wave attenuated as it moved downstream; the average celerity of the leading edge typically ranged from 2-4 m/s, while the peak celerity ranged from 1-2 m/s. On two occasions, the discharge computed with the RLAM exceeded 400 m³/s near the consolidation toe, more than twice the steady unperturbed discharge. Shear stresses on the ice cover were found to increase by as much as 54 Pa while the total consolidation stress increased by as much as 99 Pa.

This work was a novel application of the RLAM to water levels recorded at freeze-up, using gauges spaced at a relatively high resolution. The analysis highlights that consolidation events create unsteady flow conditions that could have important implications on the formation of the ice cover. The attendant increase in the driving stresses on the ice cover, in addition to the separation of rubble and lifting of the cover from the channel banks, poses a threat to the stability of the cover and can lead to further consolidation. Showcasing this concept through the presentation of field data and application of the RLAM emphasizes a potential shortcoming in current methods to predict the ultimate thickness of the ice cover, which often assume steady flow conditions.

Chapter 3 presented a laboratory investigation of the influence of thermal processes on ice dynamics. The experiments constitute the first laboratory study of the consolidation resistance of a floating rubble ice cover with a thermally grown surface ice crust. An apparatus was designed and constructed to test the ability of a floating ice cover to resist a

uniaxial force after various freezing indices. With exposure to a freezing air temperature of -10.5°C , the interstitial water between the rubble pieces at the surface froze, creating a solid crust layer that grew downwards through the ice cover. Freeze-bonds were also observed at contact points between the rubble pieces near the surface. It was found that at low freezing indices (less than 315°C-min), the surface crust was very thin and did not provide much rigidity to the ice cover; the consolidation resistance of the ice cover increased by approximately $0.6\text{ N/}^{\circ}\text{C-min}$. At freezing indices greater than 630°C-min , the surface crust added more rigidity to the cover and the consolidation resistance increased by approximately $5.0\text{ N/}^{\circ}\text{C-min}$. The presence of the rubble layer was found to add stability to the surface crust layer; the peak resistance of a rubble cover with a thermally grown surface crust was greater than the summation of peak strengths of the rubble cover and surface crust layer tested individually.

The laboratory experiments constitute a novel contribution to the field of river ice research. The apparatus and test facility permitted the careful control of test conditions, and ability to precisely measure the important variables of interest. The work builds off past research on the strength of rubble ice accumulations by introducing the effect of additional freezing, which is a factor that plays an important role in the real world. The study highlighted the strong influence of thermal processes on river ice dynamics, and provided an approximate range of values of the crust strength that may be applicable to field studies.

In Chapter 4, a summary of the CRISP2D model formulations was presented, with a focus on the treatment of ice dynamics. In essence, the model considers thermal processes in the generation of ice volume, but there is no direct effect of thermal processes

on the ability of ice parcels to resist mechanical thickening. A new formulation was proposed, which incorporates the non-frictional resistance added by the thermally grown surface crust. The formulations were shown to work, in concept, on an idealized, prismatic channel. This is a significant contribution to the CRISSP2D model, and also to the general field of numerical modeling of river ice. The model now includes a process that can have significant implications on the formation of an ice cover at freeze-up, as noted by other researchers and supplemented by the field and laboratory work in this research. The implementation of the treatment that links thermal and dynamic processes in the CRISSP2D code provides the framework with which the model can be further tested and developed as additional research is performed and new theories are proposed.

The CRISSP2D model was applied to three freeze-up seasons of the Lower Dauphin River. The detailed field datasets allowed for thorough comparisons between model results and field measurements. The revised model formulations with a crust strength of 0.3 MPa worked well for the 2016-2017 freeze-up period, which exhibited consistently cold air temperatures. Peak ice elevations and water levels were more accurately modelled over the entire reach of the Lower Dauphin River compared to the base model; the mean absolute error in peak water level at 13 gauge locations decreased from 0.89 m to 0.33 m. The 2015-2016 and 2017-2018 freeze-up seasons involved periods of warmer weather and additional processes, some of which CRISSP2D is not currently capable of simulating, which highlighted some potential areas of future work to allow the revised model to be more robust and applicable to a wider range of freeze-up conditions. In these seasons, the base model and thermal model with a crust strength of 0.1 MPa

performed comparably well, although the peak ice elevations were under predicted by both models in some locations.

The application of the two-dimensional numerical model to a field case study is another contribution of this research. Since the model was intended to simulate the time dependent formation of the ice cover, the model itself took significant effort to “build.” For example, the surface ice discharge had to be calibrated to field observations to ensure that the model was correctly simulating the rate of heat transfer, which has implications on the rate of increase of stresses on the ice cover (via cover lengthening), as well as the rate of increase of resistance to consolidation (via interstitial crust growth). The attempt to incorporate the surface crust growth and effect on mechanical thickening in a two-dimensional model was also a novel endeavour, and provides the basis for future research in the area.

5.2 Future Work

The work presented in this thesis has been successful, and has highlighted some areas for future field and laboratory work, as well as some areas of model development. Extended from the summary and conclusions above, the following topics of future work have been identified:

Field Monitoring

1. The field monitoring program on the Lower Dauphin River should be continued.

The current spatial resolution of deployed instruments is adequate to capture consolidation events and associated changes in water levels along the reach. The

practice of surveying the final ice profile and estimating ice thicknesses based on ice and ground elevations provides a very useful calibration set. Monitoring sites could be added upstream of DRLL04 (in the Upper Dauphin River) to investigate the processes related to ice cover progression in this area. The additional data from new sites and future freeze-up periods will be useful in further assessment and development of the numerical model.

2. Ice discharge should be estimated at more times during the freeze-up period using UAV video and particle tracking to allow for better calibration of the free-drift model (or to obtain a better understanding of the uncertainty of this model). Discrete measurements of ice discharge may allow for rectification of trail camera images to produce a continuous time series of ice discharge over the freeze-up period. If possible, ice floes should be extracted from the river and measurements taken to have a better estimate of the average ice thickness of the surface floes in the UAV video.
3. The dominant terms in the energy budget should be quantified on the Dauphin River to allow for a more reliable simulation of ice generation and crust growth in the model. Special attention should be given to keeping the net radiation sensor clear of snow to obtain more reliable measurements of the diurnal variation in solar radiation. Measurements of thermally grown ice, for example on Lake St. Martin, could be useful in calibrating the linear heat transfer coefficient in future seasons.

4. The value of the Manning roughness of the ice cover and its variation through various stages of consolidation requires attention. It is unclear if the variation in Manning roughness proposed by Nezhikhovskiy (1964) is representative of newly consolidated ice covers with large fragments protruding into the flow. Analysis of ice surface elevations as measured with a terrestrial laser scanner or with UAV photogrammetry may be useful to try to quantify the surface roughness, which may provide better estimates of the roughness of the underside of the ice cover.

Laboratory Studies

5. Additional experiments should be conducted with the same apparatus that was used for the test series in this thesis. Variables such as air temperature, ice cover length, and ice cover composition (e.g. finer ice pieces to represent frazil slush) could be varied to evaluate their effect on the resistance of the ice cover to consolidation. Additionally, the effect of above 0°C air temperatures (i.e. deterioration and melting of ice) following the growth of a surface crust on the peak resistance of the ice cover could be investigated.
6. The effect of small water seepage velocities, and relative motion of the ice pieces in the ice cover, on the ability of the ice cover to grow a surface crust should be investigated. These factors occur at the field scale and were not considered in the laboratory test series in this thesis.

Numerical Modeling

7. The revised model should be applied to additional field datasets as they are collected on the Dauphin River, but also to other case studies where the link

between thermal and dynamic ice processes is important. For example, the formation and stability of an ice cover upstream of the Jenpeg Generating Station in Manitoba during the Ice Stabilization Program might be better simulated with the revised model formulations.

8. Additional subroutines should be added to the CRISSP2D model to capture the apparent smoothening of the underside of the ice cover with time, as well as the development of narrow open water leads. While complex, these processes were observed in the field and undoubtedly affect ice-affected staging and under ice water velocity.
9. The speed of the CRISSP2D model could be increased by code parallelization and/or improvements to the efficiency of the search algorithm for neighboring parcels in the SPH method in the ice dynamics calculations. A faster model could aid in further development (e.g. faster calibration and determination of sensitivity to parameters), and make the model more attractive as an operational tool.

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Appendix A Fitting of Measured Waveforms using Rising Limb Analysis Method

The following figures show the computed water levels using the RLAM for each surge wave identified in Figure 2.34. The calibration parameters of C_p (peak celerity), C_l (leading edge celerity), and m are shown in each sub-plot.

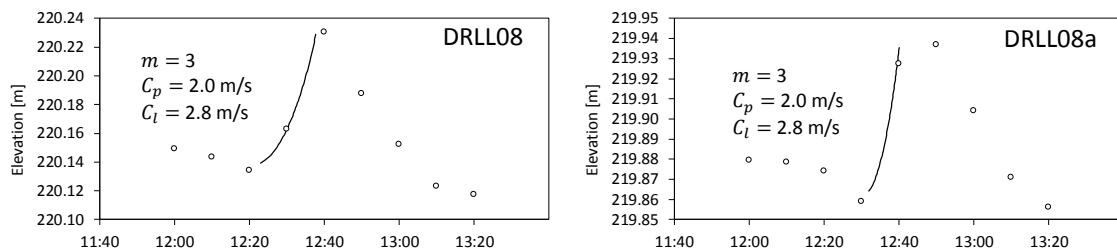


Figure A.1. Computed waveform and fitted parameters for Surge 1.

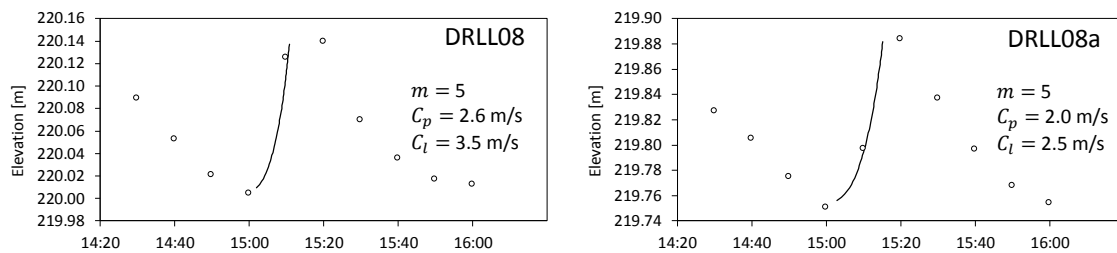


Figure A.2. Computed waveform and fitted parameters for Surge 2.

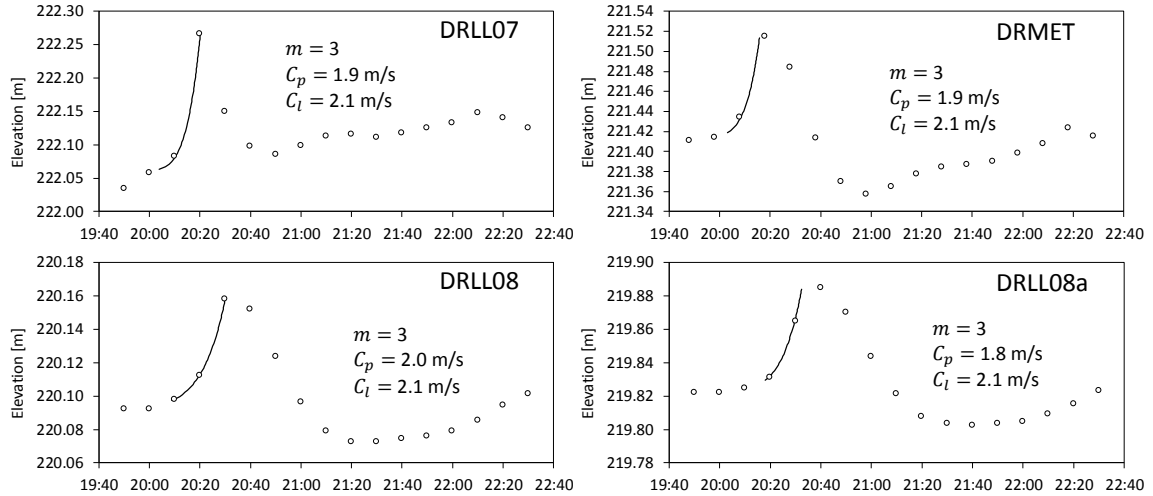


Figure A.3. Computed waveform and fitted parameters for Surge 3.

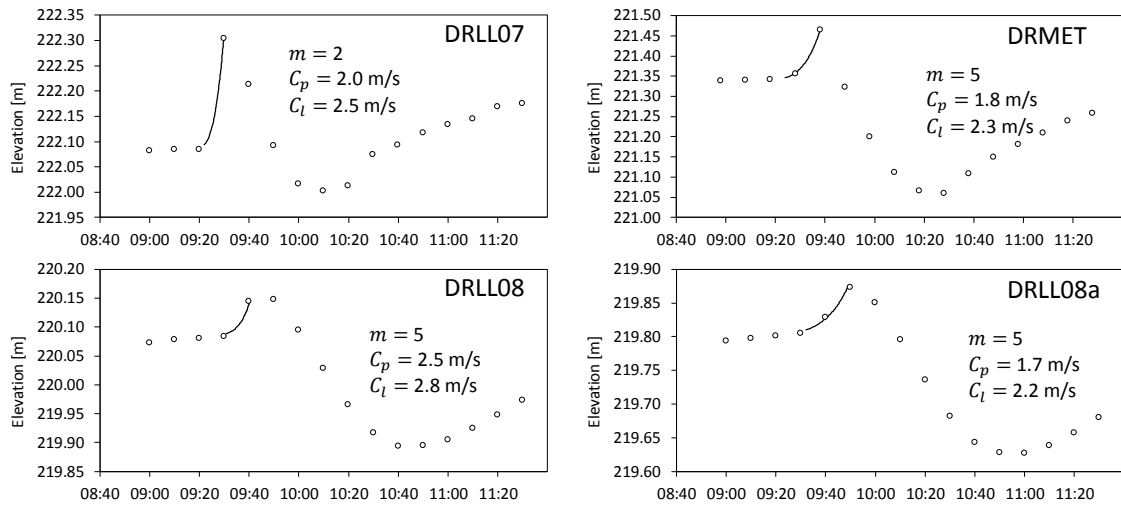


Figure A.4. Computed waveform and fitted parameters for Surge 4.

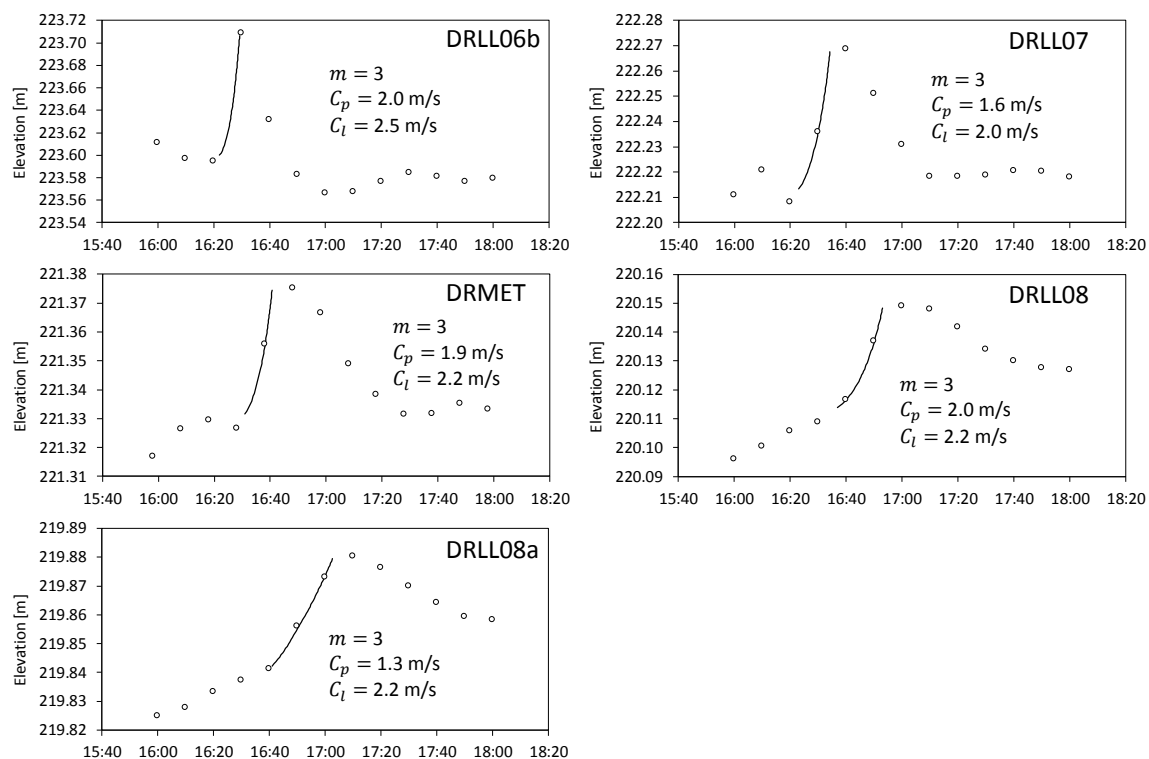


Figure A.5. Computed waveform and fitted parameters for Surge 5.

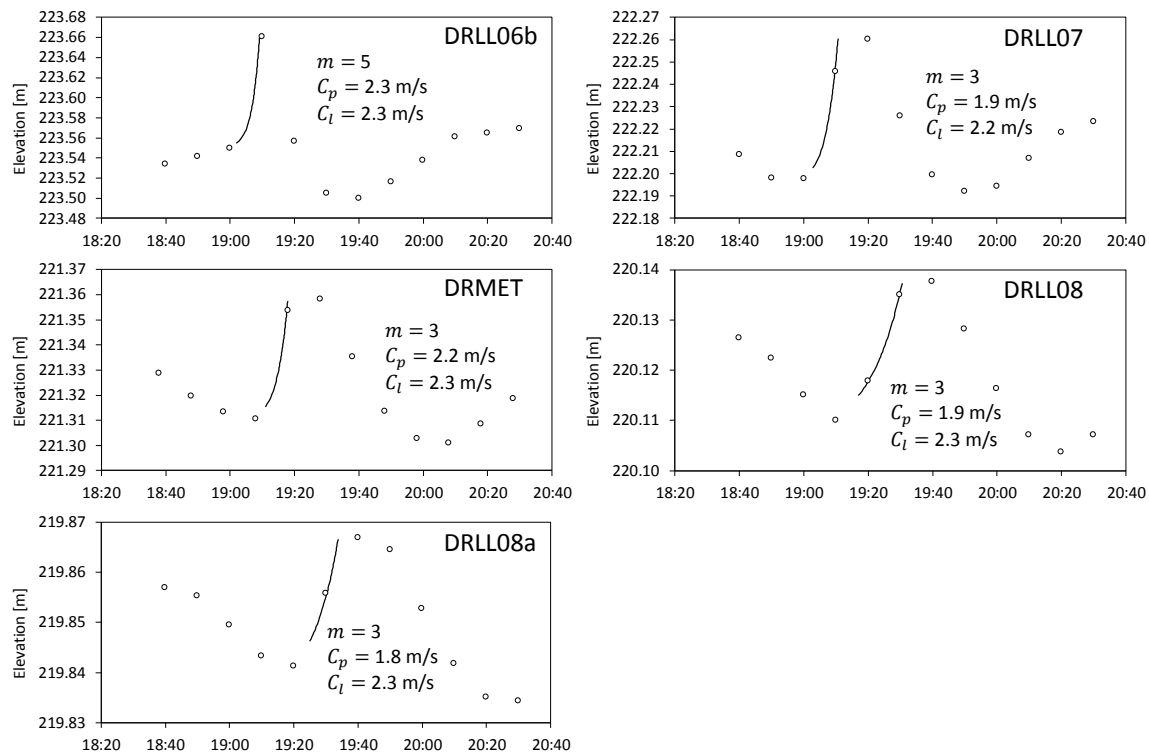


Figure A.6. Computed waveform and fitted parameters for Surge 6.

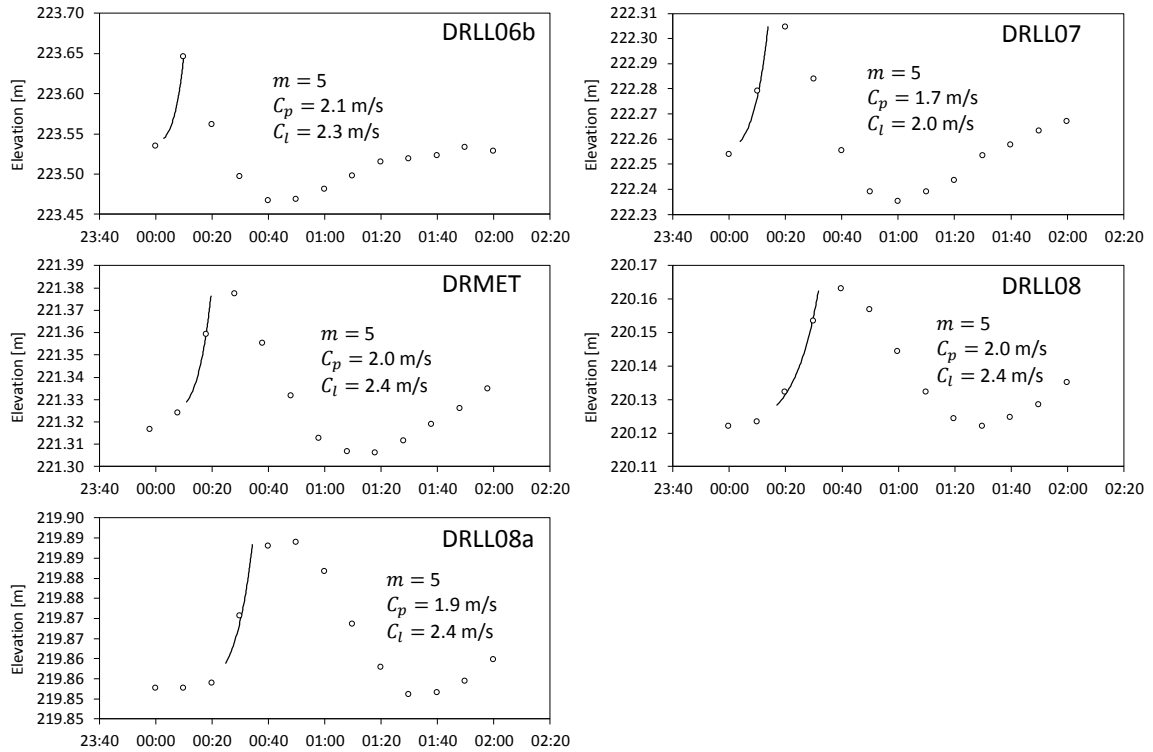


Figure A.7. Computed waveform and fitted parameters for Surge 7.

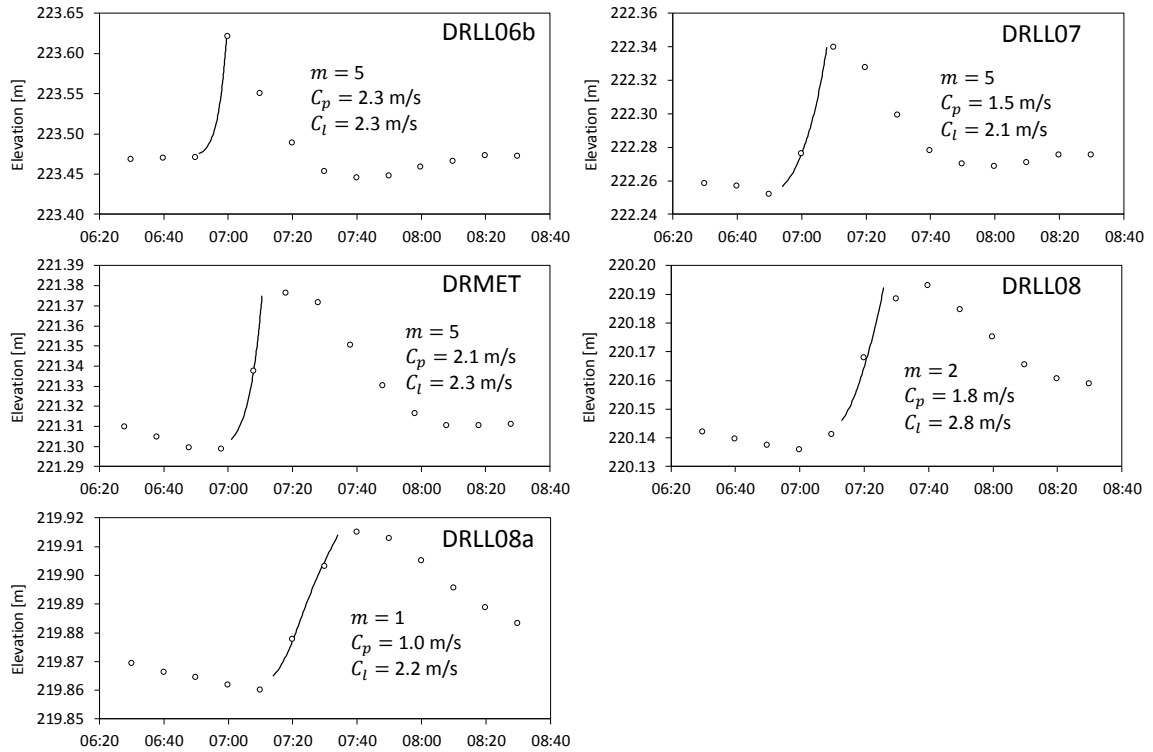


Figure A.8. Computed waveform and fitted parameters for Surge 8.

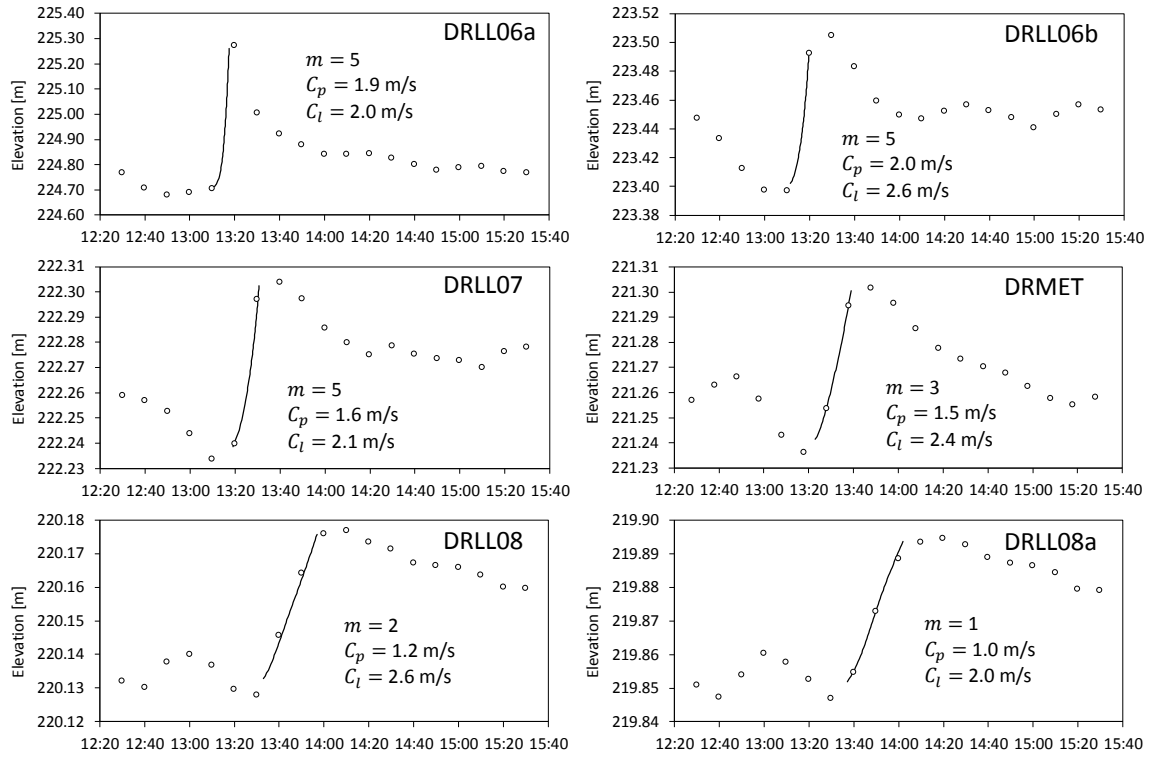


Figure A.9. Computed waveform and fitted parameters for Surge 9.

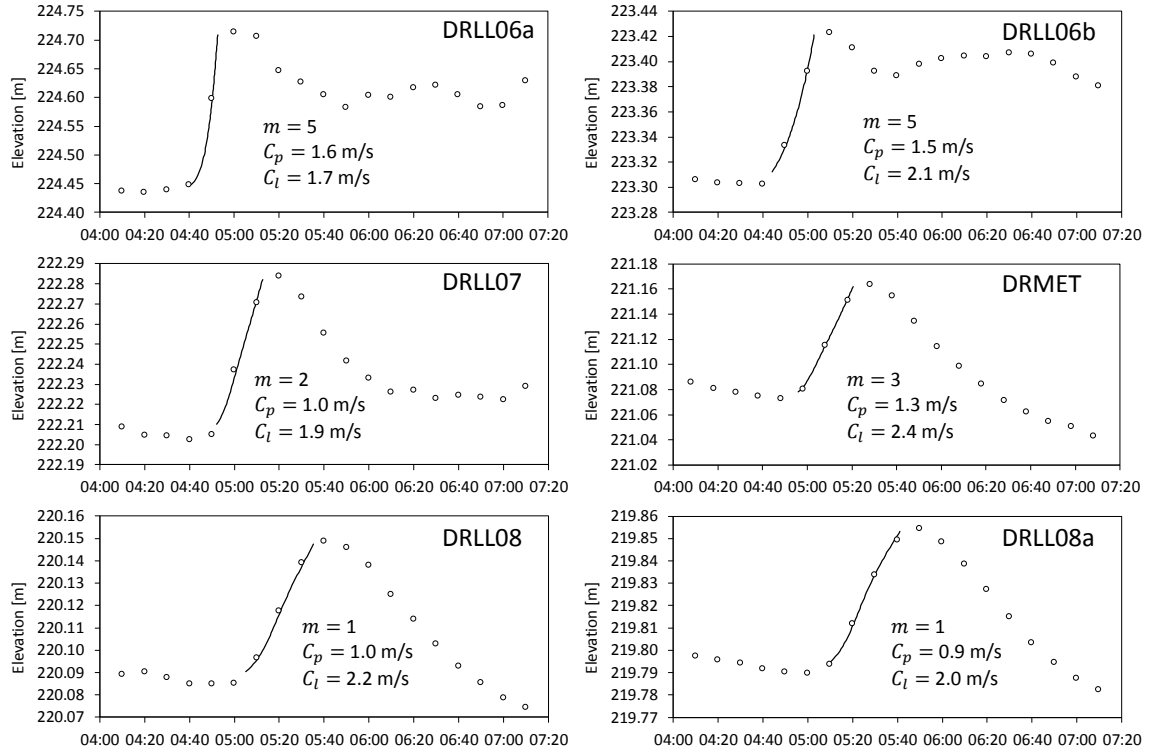


Figure A.10. Computed waveform and fitted parameters for Surge 10.

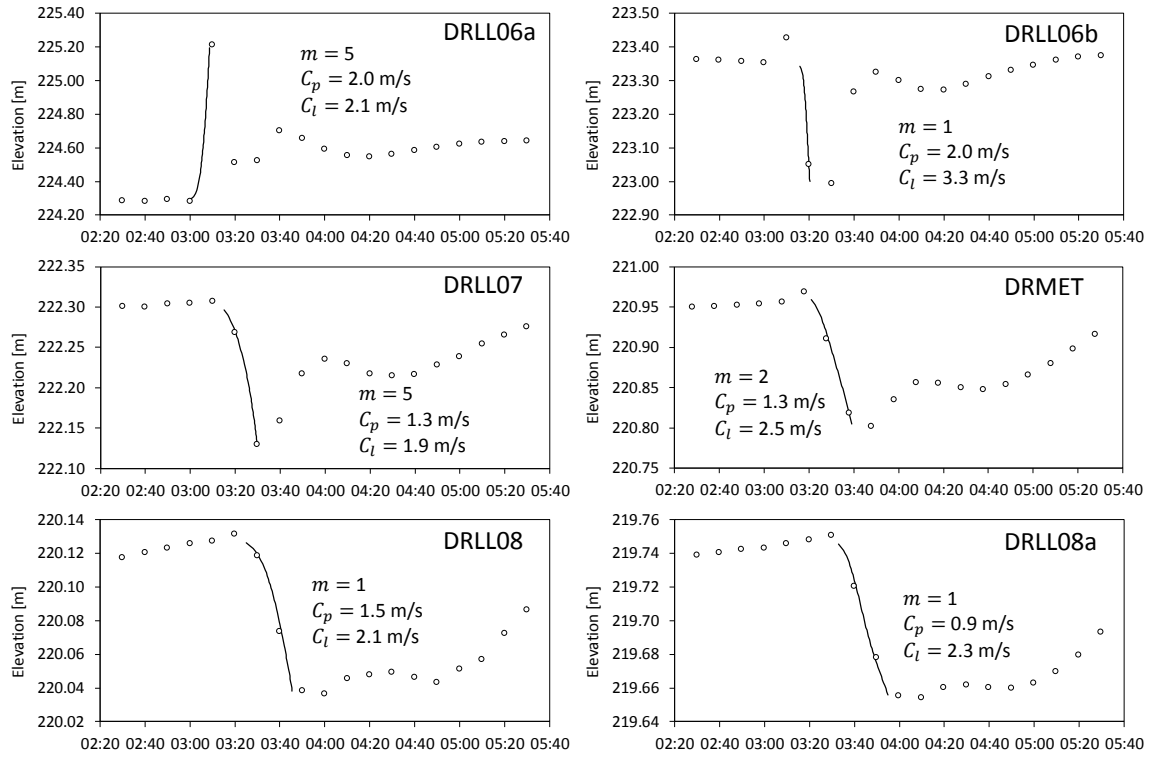


Figure A.11. Computed waveform and fitted parameters for Surge 11.

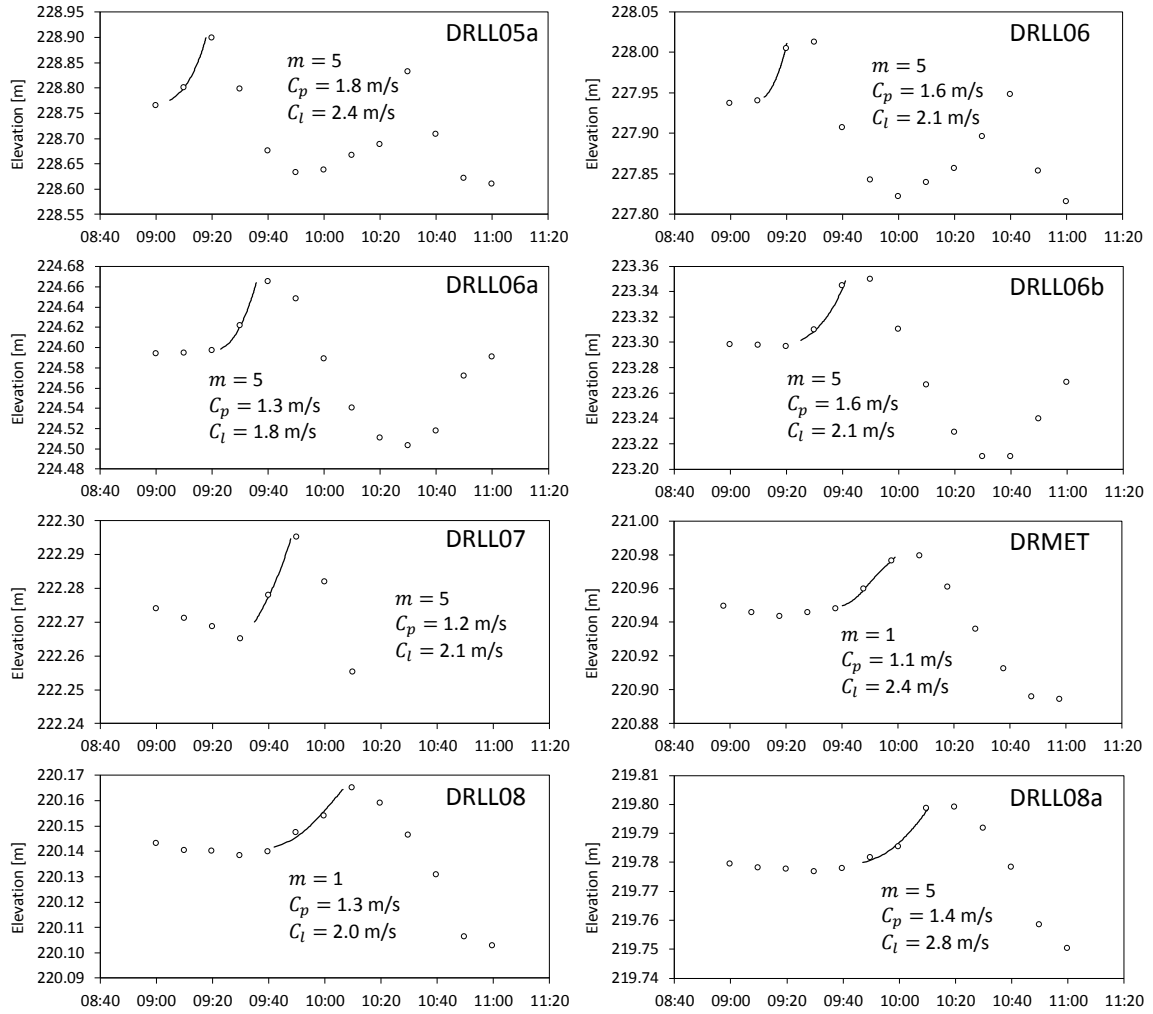


Figure A.12. Computed waveform and fitted parameters for Surge 12.

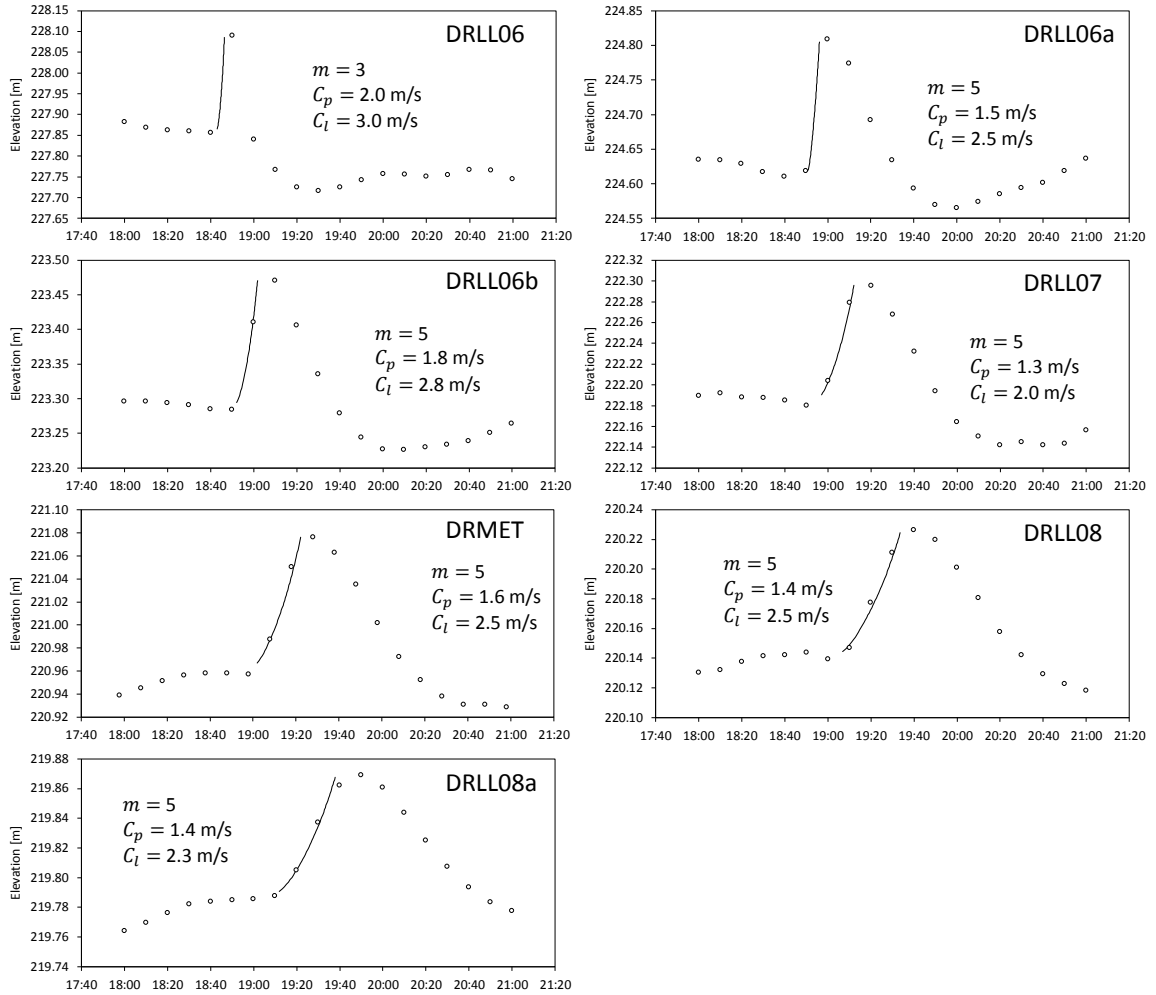


Figure A.13. Computed waveform and fitted parameters for Surge 13.