Proceedings
The 19th IAHR International Symposium on Ice
“USING NEW TECHNOLOGY TO UNDERSTAND WATER-ICE INTERACTION”
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BC Hydro

VOLUME ONE

International Association of Hydraulic Engineering and Research
Ice Research & Engineering
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Preface

The International Symposium on Ice is sponsored by the International Association of Hydraulic Engineering and Research (IAHR). It has been a regular biennial forum since 1970 for scientists, engineers and researchers to exchange information on the many cross-disciplinary topics of ice engineering.

This is the 19th IAHR International Symposium on Ice and the general theme this year was “Using New Technology to understand Water-Ice Interaction”. The new technology falls into three basic groups, which are: Measurements and Instrumentation, Remote Sensing, and Numerical Simulation. A large portion of the papers used some aspect of these three investigative strategies to solve problems and to give us a better understanding of water-ice interaction.

Special sessions were held on Oil Spills in Ice, Ice on Compliant Structures, Remote Sensing, Numerical Simulation and the Ice Crushing Process. A unique special session of 9 papers on ice process considerations for the remediation options to address PCB contamination in the river bed sediments of the Grasse River was also held. Climate change and environmental considerations were considered in papers throughout the sessions. Other topic areas were Freeze-up Processes in Rivers and Oceans, River Ice break-up and Ice Jam Formation, Ice Measurement Technologies, Ice Effects on Hydropower Generation, the Effect on the Ice Regime due to Dam Decommissioning, Sea Ice Ridging, Ice Properties, Testing and Physical Modelling, Ice – Shore and/or Structure Interaction, Icebergs, Ice and Navigation, and Ice Mechanics.

On behalf of the organizing committee I wish to thank the many people who traveled long distances and at considerable expense to contribute technically, as well as socially, to this event. I know that this event has been an experience that we will all treasure, and that the knowledge exchanged and friendships formed and solidified will help us solve ice problems in the future.

I would like to acknowledge the University of Alberta, the Canadian Hydraulics Centre and Alberta Environment for providing the backbone of support and experience to organize and run the symposium. The sponsors were also of great help to keep registration costs down and we are all indebted to them. Finally I would like to thank BC Hydro for providing staff and resources to organize this event; without this the Symposium would not have been possible.

Best Wishes

Martin Jasek, M.Sc., P.Eng.,
Chair of the Local Organizing Committee
19th IAHR International Symposium on Ice
Senior Engineer, British Columbia Hydro and Power Authority (BC Hydro)

July 2008
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Front Cover: Smoky River breaking up into the Peace River, Alberta, April, 2007
Back Cover: Frazil ice pans on the Peace River near Alces River, British Columbia, January 2005
by Martin Jasek, BC Hydro

Cover Photos Volume Two
Front Cover: Edge of the pack ice region in the Beaufort Sea in April, 2007
Back Cover: Ice pile-up in Beaufort Sea
by Garry Timco and used in his keynote lecture entitled "Why Does Ice Fail the Way it Does?", Canadian Hydraulics Centre
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1992 Banff, Canada
1994 Trondheim, Norway
1996 Beijing, China
1998 Potsdam, USA
2000 Gdansk, Poland
2002 Dunedin, New Zealand
2004 St. Petersburg, Russia
2006 Sapporo, Japan
Challenges and opportunities in the study of river ice processes

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Abstract

Many rivers of cold, and even temperate, regions of the globe are covered with ice for a part of the year. Dynamic phenomena during the transitory periods of freezeup and breakup and the more stable condition of a complete ice cover during the winter, all have major and varied impacts on stream ecology, safety of people and infrastructure, transportation, and hydro-power generation. In the past fifty years or so, remarkable progress has been made in understanding and quantifying many of the complex thermodynamic, hydraulic, and structural processes that govern the ice regime of rivers. Yet, many problems posed by river ice remain unsolved or partly addressed, while some are just as intractable now as in the past. The practical answer to this situation has been to rely on empirical relationships, developed from historical data at specific sites. Implicit in such empiricism is the assumption of a “stationary” climate, that is, the statistical properties of local climatic variables do not change over time. This assumption is becoming increasingly untenable in view of already-experienced and anticipated climatic changes, which also tend to be more pronounced in a northerly direction and during the winter months. Numerous changes to the ice regimes of the world’s rivers have already been recorded, and some general projections can be made for the future. However, it is not possible to make specific predictions because our physical understanding remains incomplete. Thus, the main challenge is how to accelerate the pace of discovery and bridge the major knowledge gaps. Discussion of current opportunities indicates that river ice researchers can now obtain plentiful field data and perform sophisticated analyses by exploiting recent technological developments. These tools include new remote-sensing instrumentation, GPR systems, satellite imagery and processing algorithms, as well as advanced numerical models, whose scope is continuously expanding as computing power increases.
1. Introduction

The duration of the ice season in northern rivers typically ranges from a few weeks to several months. On some occasions, ice may also occur at latitudes that would normally be considered southern. For instance, the US Army Corps of Engineers ice jam database (White and Eames, 1997; https://rsgis.crrel.usace.army.mil/icejam) indicates that ice jams in the United States have occurred as far south as Texas, at a latitude of 32°, though the vast majority of occurrences lie north of 37°. Ice is also common in rivers of Europe and Asia, while no ice-related problems have been reported for rivers located in the southern hemisphere.

River ice plays a major role in various socio-economic impacts of rivers, such as flooding and low flow extremes, damage to infrastructure, clogging of water intakes, constraints to hydropower generation, and interference with navigation and overland transportation (Ashton, 1986; Beltaos, 1995). Moreover, river ice is intimately linked to a variety of ecological processes and can be either detrimental or beneficial to nearby ecosystems, depending on the situation (Prowse, 2000).

Despite the many effects that ice has on river regimes, most hydrologists and river engineers have only been trained in open-channel processes. Few universities offer full or partial courses on the hydraulics of ice covered channels. In part, this is the result of the relative youth of river ice science (~ 100 years old). An additional factor is the complexity of the subject, which involves several major areas of physics, e.g. thermodynamics, fluid mechanics, geomorphology, as well as the mechanical properties of solids and granular materials.

Various reviews and state-of-the-art reports have been produced in the past few decades, summarizing knowledge in particular areas of river ice research and engineering, but there are only three textbooks that provide comprehensive syntheses of their subject matter:

- River and Lake Ice Engineering (1986)
- River Ice Jams (1995)
- River Ice Breakup (2008)

The objective of this article is not to provide another review, but to highlight the gaps in knowledge and the associated challenges to hydraulic research and engineering, and to identify research opportunities that recent technological advances have created. A brief overview of the main river ice processes is presented first, with emphasis on related problems and beneficial effects. This leads to a subjective assessment of the state of the art and identification of challenges, including those specifically arising from the issue of climate change and variability. Technologies that have recently been used in, or shown to have potential for, river ice studies are discussed next, and shown to offer new opportunities for a wide spectrum of research and engineering activities. These comprise new instrumentation capabilities, sophisticated numerical models, innovative mitigation techniques, as well as improved climate modelling capabilities.
2. Overview of river ice processes and related problems

Detailed descriptions of river ice processes and the evolution of the ice season are beyond the scope of this article, but can be found in several publications, such as Ashton (1986) and Beltaos (1995; 2008). Instead, the aim of this section is to provide a brief overview in order to highlight major challenges posed by river ice.

2.1 Freezeup

As fall advances towards winter, air temperatures and incoming solar radiation decrease to the point that the net heat flux through the open river surface becomes negative. River temperatures drop in response, and eventually ice begins to form. Though border and skim ice are common, it is frazil ice that is most characteristic of early freezeup in natural streams (Figure 1). Frazil appears where the water temperature drops to a few one-hundredths of a degree C below the freezing point (supercooling). By contrast to lakes and tranquil flow near the shores, turbulent mixing of river flow prevents thermal stratification and results in near-uniform cooling of the water column.

![Figure 1](image.png)

**Figure 1.** Frazil ice in suspension, primarily comprising needle-like and discoid crystals. Discoid diameter size is in millimetres. Photo: G. Tsang.

Frazil ice is extremely adhesive in a supercooled environment, readily attaching and accumulating on submerged objects. It can thus seriously clog water intakes used for municipal or industrial purposes. Frazil accretion on the riverbed is a porous accumulation known as anchor ice. However, anchor ice can also form by in situ thermal growth in the shape of large crystals (see Kempema et al., 2008, for an extreme example). Anchor ice can degrade stream habitat and raise local water levels, thus interfering with hydropower generation.

In supercooled water, frazil crystals also attach to each other, forming flocs that tend to rise to the water surface where they successively agglomerate into amorphous clumps of slush and later
into rounded ice floes called “ice pans”. Eventually, ice pans are congested between border ice strips and initiate a stationary but porous ice cover. Freezing of interstitial water results in a solid-ice layer, which continues to grow by freezing into the underlying flow layer (sheet ice cover). Once an ice cover has formed, supercooling and frazil production cease, except for a short distance from the upstream edge of the cover.

Under certain hydraulic conditions, the initial porous cover may collapse and re-form at a greater aggregate thickness, which is just capable of resisting the applied external forces. Such thickened covers are known as narrow- and wide-channel jams (Pariset et al, 1966). They can be several metres thick and their rough undersides greatly augment hydraulic resistance to flow. The water level has to rise considerably in order to pass the river discharge and to accommodate the keel of the jam. Freezeup jams of this type can inhibit hydropower generation and, in extreme cases, cause flooding.

However, another type of freezeup jam can be more dangerous. This is the so-called hanging dam, which forms by transport of frazil ice and eventual deposition under a stationary sheet ice cover that may have already formed over a relatively calm reach. Hanging dams often attain thicknesses measured in the tens of metres (Figure 2), and may continue to grow throughout the winter if the supply of frazil is maintained (Figure 3). This situation occurs where a relatively steep reach above the hanging dam remains open due to rapid flow, and generates large frazil volumes. The increasing thickness of the hanging dam causes a sustained rise in local water levels and could lead to worse flooding than that associated with collapse-type freezeup jams.

![Figure 2](image-url)  
**Figure 2.** Longitudinal profiles of two hanging dams in the LaGrande River, February, 1973. From Ashton (1986); data by Michel and Drouin. Reproduced with permission of Water Resources Publications.
While flooding is a major concern, low-winter flow is another hydrologic extreme that can occur during freezeup. Low flows can compromise the water quality of rivers receiving industrial/municipal effluents and constrain the quantity of water that may be available for industrial uses, such as for tar sands exploitation. The primary cause of low winter flows is the abstraction generated by hydraulic storage of water when an ice cover is forming upstream. The freezing of river water and constraints to groundwater inflows via freezing to the riverbed are additional, but secondary, factors. The abstraction occurs at a time when the river flow is already low, and often leads to seasonal or even annual flow minima (e.g. see Gerard, 1981; Prowse and Carter, 2002) but no form of regional or return-period analysis has ever been conducted on this phenomenon despite the need for such in hydrologic and ecological evaluations.

Figure 3. Growth of a hanging dam in the LaGrande River (Transect B in Fig. 2). From Ashton (1986); data by Michel and Drouin. Reproduced with permission of Water Resources Publications.
2.2. Stable ice cover

Once a stable ice cover has been established, it grows vertically by downward freezing into the underlying water column or into any porous accumulation that may be present as a result of freezeup jamming. In the former case, the ice cover can be highly transparent. The rate of growth decreases over time because the growing ice thickness provides increasing insulation. Maximum winter thickness varies with local hydro-climatic conditions, commonly ranging from 0.3 to 1.5 m. The winter ice cover creates a stable and relatively secure environment for aquatic life. It is also a net asset from the socio-economic point of view: though it inhibits navigation, the ice cover facilitates overland transportation, particularly to and from remote northern communities, and is also used as a construction platform. A key consideration in such activities is the bearing capacity of floating ice covers, which is primarily determined by ice thickness and strength.

2.3. Breakup

On many northern rivers, the terms “breakup” and “spring breakup” are almost interchangeable. Indeed, as the consistently subfreezing winter weather breaks with the arrival of spring, runoff starts to increase and a progressive weakening of the ice cover begins. Rising flows and water levels fracture the cover and reduce its attachment to the river banks and various islands, until a point is reached at which entire ice sheets from the original cover are mobilized. Once in motion, such ice sheets quickly break down into small blocks which may be arrested at places where the ice cover has remained intact, creating breakup jams (Figure 4). Where the winters are relatively moderate, they include occasional brief thaws which are often accompanied by rainfall. Rain-on-snow events cause rapid runoff and often lead to mid-winter breakup events.

Figure 4. Breakup jam in Smoky River, Alberta, upstream of its confluence with Peace River, whose ice cover is still intact, April 1976. Photo: S. Beltaos.
Jamming can take place almost anywhere in a river, though certain morphological features are more jam-prone than others. These include, for example, sharp bends, constrictions, shallows, or abrupt reductions in slope and velocity as often occurs at reservoirs and river mouths. Two such features are exhibited in Figure 5, which depicts the general area of the toe, or downstream end, of the jam.

![Figure 5](image)

**Figure 5.** Ice jam toe located at a sharp bend and constriction in the Smoky River below Watino, Alberta, April 1977. Stability of intact ice sheet is probably enhanced by a set of bridge piers, barely visible near the upper border of the picture. Photo: S. Beltaos.

Breakup jams are almost always of the wide-channel type. Their thickness and roughness depend on local bathymetry as well as on the prevailing river discharge. Because the latter is usually much larger than freezeup flow, breakup jams can cause extreme flood events that are more destructive than freezeup ones, and even than rare open-water floods (Figure 6). Mid-winter jamming is often harsher than spring jamming because residents of flooded communities have to cope with wide-spread and sustained freezing upon resumption of the cold weather.

One might hope that the release of an ice jam would bring about final relief and restore benign conditions along the river but this is not the case. Breakup jams often release in abrupt fashion, generating a steep water wave akin to, but much smaller than, a dambreak. Even so, jam release waves (or “javes” for short) propagate downstream at celerities of 10 m/s or more, while concurrent water speeds can easily exceed 3 m/s (Beltaos, 2008b). Water levels can rise as fast as several metres within minutes (Beltaos and Burrell, 2005; Kowalczyk-Hutchison and Hicks, 2007). There is little time for emergency action, and javes have claimed many lives. The potential for flooding, for bed and bank scour, as well as for damage to river structures by moving ice floes, is very high, especially near the point of release where jave attenuation is still minimal. There are many witness accounts of the destructive power of the jave and the hazards it entails for
local residents (Beltaos, 1995). Upstream of where the toe of the released jam had been, water levels drop precipitously, creating temporary but sharp imbalances in the pore-water pressures within a riverbank. This phenomenon could account for occasional bank failures, such as the one shown in Figure 6. In cases where the jam had been causing floodplain inundation, the quick stage drop to pre-flood levels can cause extensive fish mortality by stranding on the floodplain. Javas can also damage riverbed habitat via large-scale erosion or by eventual deposition of fine sediment that is initially carried by the flow in high-concentration pulses (Beltaos and Burrell, 2000).

![Figure 6](image)

**Figure 6.** Aftermath of a major ice jam and flood event in the Saint John River near Dickey, Maine, April 1991. Note grounded remnants of the jam along the far bank, “orphaned” bridge piers, and riverbank slide with loss of highway segment. Photo: S. Beltaos. [A shear wall is the edge of grounded rubble that remains in place when a jam releases; its height is a rough indication of the jam thickness].

Though ice jams pose extreme hazards to people, they can be vital to the maintenance of floodplain habitat and associated ecosystems. This is exemplified by the large freshwater deltas of Canada’s north, i.e. the Peace-Athabasca Delta, the Slave Delta and the Mackenzie Delta. Ice jams in the delta channels generate much higher water levels than open-water floods and divert much needed water, sediment, and nutrients toward the myriad lakes and ponds of the delta landscape (Figures 7 and 8). Frequent ice jamming is a necessary factor in the replenishment of the higher-elevation, or perched, basins of the delta. Where the frequency of such jamming is diminished by regulation or by climatic changes, the perched basins tend to dry up (Beltaos et al., 2006b, 2006c).
Figure 7. Ice jam in the Middle Channel of the Mackenzie Delta, May 2008, raises water levels and forces water and rubble into various tributary and distributary channels. Looking upstream. Local channel width ~ 6 km. The Mackenzie River is seen entering the Delta in the upper central portion of the image. Photo: S. Beltaos.

Figure 8. Ice jam in the East Channel of the Mackenzie Delta, May 2008, diverts water towards nearby lakes and ponds via low points along the banks. Flow is from right to left. Photo: S. Beltaos.
3. Main Challenges

The overall challenge posed by river ice problems is twofold, that is, how to avoid or mitigate their adverse impacts and how to ensure that their beneficial effects are not diminished by human activities. To meet this challenge, a good qualitative understanding of river ice processes is essential, though rarely sufficient. For example, the height of a bridge superstructure can be determined on the basis of historical data, once it has been established that ice jams govern local high water levels. However, such questions as potential scour and undermining of bridge piers, or whether the obstruction created by the piers can cause ice jamming, can only be answered by means of detailed calculations and numerical model applications (see also Beltaos et al., 2006a 2007b).

Physics-based numerical simulation is thus a key capability for dealing with such practical problems as:
- flood forecasting and warning
- impacts of ice on river structures and vice-versa
- environmental impact assessments of proposed or existing structures and human activities
- assessment of design criteria for, and efficacy of, contemplated mitigation measures

Though there has been considerable progress in the quantification of river ice processes, serious gaps remain. For example, it is not yet possible to forecast major ice jamming events via physics-based equations and models. Nevertheless, useful empirical algorithms can be developed by relating known occurrences to historical hydrologic and meteorological data (e.g. see White, 2008). This approach is site-specific, i.e. it gives good results for a particular site but cannot be transferred to other sites. Not only do quantitative relationships among the selected hydro-climatic variables change from site to site, the selection of the relevant variables may also change.

The writer’s assessment of how well major river ice phenomena are presently understood is summarized in the following table. It is beyond the scope of this article to present a comprehensive list of the extensive literature that has shaped this assessment. A few recent citations, some of which can be found in the Proceedings of this Symposium, are listed in the 3rd column of Table 1 for the convenience of those readers who may want to learn more details and access references to additional information.

The issue of climate change, which has been a growing global concern in the last twenty years or so, underscores the need for improved river ice science. River ice processes are highly sensitive to climatic conditions, not only through changes in atmospheric parameters, but also through climate-induced changes to the hydrologic regimes of river basins. At the same time, ice laden rivers are subject to more severe climatic changes than what is indicated by global means: climate models and known trends show that mean annual warming becomes more pronounced in a northerly direction, while winter warming exceeds mean annual values. Significant changes to the ice regimes of the world’s rivers have already occurred over the past 50-100 years (e.g. Vuglinsky, 2006; White et al., 2007; Beltaos and Prowse, 2008). As climate change is expected
Table 1. Subjective assessment of the state of knowledge on key aspects of the river ice regime

<table>
<thead>
<tr>
<th>Subject</th>
<th>State of knowledge</th>
<th>Recent references</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frazil ice</td>
<td>Fair; advanced modelling capability; difficult to measure concentration of suspended frazil particles and flocs.</td>
<td>Daly (1994*, 2008*); Morse et al. (2003b); Clark and Doering (2008); Jasek and Marko (2008); Richard and Morse (2008)</td>
</tr>
<tr>
<td>Anchor ice</td>
<td>Rudimentary, but good progress in last decade via laboratory and field measurements.</td>
<td>Kerr et al. (2002); Qu et al. (2007) Kempepa et al. (2008)</td>
</tr>
<tr>
<td>Hanging dams</td>
<td>Fair; submergence properties and under-ice transport of frazil ice and ice pans partially understood.</td>
<td>Shen and Wang (1995); Beltaos et al. (2007a)</td>
</tr>
<tr>
<td>Breakup and jamming</td>
<td>Nil to very good depending on aspect. Ice jam formation and release still intractable. Stability of ice jams, thickness and hydraulic resistance adequately quantified via theoretical analysis and laboratory/field measurements. Antecedent conditions, such as freezeup level, sheet-ice thickness, and ice strength, shown to play important roles in breakup evolution and severity. Prediction and measurement of pre-breakup ice strength remains elusive.</td>
<td>Beltaos (2008a*, 2008b*)</td>
</tr>
<tr>
<td>Javes</td>
<td>Fair; several field data sets have been obtained to date and can be modeled using simplified dynamic equations. Jave-driven interactions between moving and stationary ice not yet quantified.</td>
<td>Kowalczyk-Hutchison and Hicks (2007); She and Hicks (2006); Beltaos and Burrell (2005)</td>
</tr>
<tr>
<td>Climatic aspects</td>
<td>Rudimentary; most work done to date shows climate-related trends in ice phenology and ice cover duration; very few studies on how the severity of ice jams may be altered by changed climate.</td>
<td>Beltaos and Burrell (2003)<em>; Prowse and Bonsal (2004)</em>; Beltaos and Burrell (2006); Beltaos and Prowse (2008)*</td>
</tr>
</tbody>
</table>

(*) Review paper or text book

to accelerate in the coming decades, there is a growing need to identify resulting changes to river ice processes and consider adaptation measures. This can only be accomplished via physics-based simulation because the scope of the empirical, site-specific, option is becoming increasingly limited. Implicit in site-specific methods is the assumption of stationary climate and hydrologic regime. For example, the peak ice-jam flood stage at a particular site is determined on the basis of historical data, but cannot be extrapolated to decades in the future because relevant hydro-climatic conditions will change. Thus, the site-specific approach is now also time-specific.

The best approach for predicting future river ice conditions at specific locations would be to use river ice models that are driven by climatic conditions and by hydrographs, generated by climate
and hydrologic models, respectively. However, there are still many gaps in the scientific understanding associated with all three types of models (ice, hydrology, climate), which can combine to produce a “cascade of uncertainty”. An additional difficulty is the question of downscaling, which arises from the spatial coarseness of climate models.

4. Opportunities

As can be seen from Table 1, there is much more that remains to be discovered than what is already known. Yet, when one considers how things stood in 1974 (the year of the writer’s first “ice contact”) it is evident that river ice scientists and engineers are now much better equipped for research and practical problem-solving. Advanced instrumentation and remote-sensing capabilities have greatly enhanced the scope and quality of data collection programs, while increasing model sophistication allows quantification of complex problems. Ice jam mitigation is also an area that has benefited from new ideas in terms of both structural and non-structural measures. Some of these developments are discussed in this section. Again, the objective is not to provide a thorough review, but to highlight the potential of new technologies.

4.1. Instrumentation

In recent years, upward looking ADCPs (ADCP = Acoustic Doppler Current Profiler), IPSs (IPS = Ice Profiling Sonar), OBSs (OBS = Optical Backscatter Sonar), thermocouples, and SWIPSs (SWIPS = Shallow Water Ice Profiling Sonar) have been successfully deployed on river beds to record time series of several ice-related parameters (Morse et al., 2003b; Richard and Morse, 2008; Jasek et al., 2005; Jasek and Marko, 2008). An instrument set-up is illustrated in Figure 9.

![Figure 9](image_url)  
*Figure 9.* Schematic illustration of sensing-recording set up for SWIPS. After Jasek et al. (2005) with permission of the authors.
To date, unique data have been obtained on the surface concentration, diameter, and thickness of moving ice floes as well as on the velocity of ice and water. This information can be used with independent determinations of the width of the “ice train” on the river surface to calculate ice discharge, a key variable in many freezeup studies. Anchor ice detection is also possible with such instrumentation, while the concentration of frazil ice in suspension can be determined to the extent of arbitrary units, based on the strength of return acoustic signals. However, more work is needed in this direction because there is no known relationship between signal strength and actual volumetric concentration of suspended frazil. Once a stable ice cover has formed over the instrument package, a continuous record of the ice cover draft can be obtained and used to detect changes in cover thickness during the winter and pre-breakup periods. This allows determination of the rate of thermal thinning of the ice cover, a process that has been known to occur but not previously measured with any degree of accuracy.

The thickness of sheet ice covers can also be measured by means of Ground Penetrating Radar (GPR), using the return time of reflections from interfaces between media of sharply different dielectric constants (Fig. 10). This applies to the interface of solid ice and water as well as to the interface of the water column and the riverbed.

![GPR sample data set and post-process results. Ice thickness and water depth can be inferred from elapsed-time values in the upper and lower graphs, respectively. From Healy et al. (2007) with permission of the authors.](image)
The dielectric constants may vary spatially and temporally, so that acquisition of reliable thickness data requires simultaneous ground truthing. In a recent application, Healy et al. (2007) concluded that the GPR method has great potential even though a number of challenges remain to be addressed. Various attempts have also been made, albeit with inconclusive outcome, to detect the bottom surface of an ice jam, which comprises a mixture of frazil slush and/or solid ice blocks with water. In this case, serious difficulties arise, owing to the scatter from the multitude of reflections that occur within the thickness of the jam.

Ice jam thickness has a large effect on water levels; it is standard output of numerical models, but predictions cannot, as a rule, be confirmed because relevant measurements are unavailable. Model calibration and validation are typically based on observed water levels but this approach entails uncertainty because different combinations of rubble strength and hydraulic resistance can produce equally good predictions of water levels. For freezeup jams or mid-winter jams that have frozen in place, it is possible to access the ice surface for manual drilling, but this is not an option during breakup owing to safety considerations. In the 1990s a flow-propelled drogue named Ice jam Profiler (IJP) was used to obtain longitudinal profiles of the keel of breakup ice jams, using radio telemetry (Ford et al., 1991; Beltaos et al., 1996). The top portion of the drogue is slightly buoyant and hugs the bottom of the jam while sensing the local water pressure. The large number of thickness values that are reported by the IJP (Figure 11) can also be used to deduce the absolute roughness of a jam, which largely determines its hydraulic resistance (Beltaos, 2001).

**Figure 11.** Keel of jam measured with an IJP unit, Matapedia River, Quebec. From Beltaos and Burrell (2008).
Field reconnaissance to identify prevailing river ice conditions is an important element of flood forecasting and warning programs (e.g. Jasek et al., 2007). It is also essential in research studies that rely on acquisition of field data during the dynamic periods of freezeup and breakup. Aerial observation is usually the most efficient, and often the only, means of documenting the changing ice conditions over a river reach and locating such features as open water, moving ice floes, newly formed ice cover, ice jams, etc. This approach is limited by daylight and weather conditions; it is also expensive and labour-intensive where long reaches (hundreds of kilometres) are involved. Radar satellite imagery, which is relatively inexpensive and not constrained by daylight and cloud cover, can be a very useful addition to reconnaissance programs (see example in Figure 12).

Figure 12. Colour-coded fine mode RADARSAT-1 image (approximate spatial resolution 8 m by 8 m). Middle Channel, Mackenzie River Delta, May 27, 2008. Green = sheet ice cover; orange-brown = rubble; blue = open water. The coding was based on the level of backscattering received by the satellite, and was developed by Dr. Joost van der Sanden (Canada Centre for Remote Sensing, Natural Resources Canada). A low-level flight by the writer on the same day indicated good agreement with visual observations of river ice conditions. Flow is from South (bottom) to North (top of image). Size of depicted area ~ 25 km x 16 km.
In the past 10 years or so, there has been considerable amount of work on the potential applications of satellite imagery to river ice research and engineering. For example, Gauthier et al. (2007) and Drouin et al. (2007) explored the development of comprehensive ice classification schemes that assign different colours to different ice types or to the probabilities of encountering certain types, such as ice jams (Figures 13 and 14). Though it is not yet possible to interpret radar satellite imagery with certainty, ongoing research is very likely to produce important practical results within the next decade.

<table>
<thead>
<tr>
<th>Ice type</th>
<th>%</th>
<th>Legend</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>8.3</td>
<td></td>
</tr>
<tr>
<td>Presence of floating pans</td>
<td>10.2</td>
<td></td>
</tr>
<tr>
<td>Border ice</td>
<td>16.6</td>
<td></td>
</tr>
<tr>
<td>Ice sheet – Dominance of columnar ice</td>
<td>20.8</td>
<td></td>
</tr>
<tr>
<td>Ice sheet – A mix of columnar and frazil ice</td>
<td>18.3</td>
<td></td>
</tr>
<tr>
<td>Juxtaposed ice</td>
<td>11.9</td>
<td></td>
</tr>
<tr>
<td>Slightly consolidated ice</td>
<td>8.8</td>
<td></td>
</tr>
<tr>
<td>Moderately consolidated ice</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>Heavily consolidated ice</td>
<td>1.1</td>
<td></td>
</tr>
</tbody>
</table>

Moving ice in open channel

**Figure 13.** Ice map with 9 classes, Athabasca River, km 216.5, November 5, 2006. From Gauthier et al. (2007) with permission of the authors.

**Figure 14.** High probability of an ice jam (in red), low probability (in blue). Saint-Francois River (January 26, 2006). From Drouin et al. (2007) with permission of the authors.
4.2. Numerical Modelling

Steady-state 1-D (one-dimensional) river ice models reached a good level of performance and robustness by the mid-1990s. They are still used as both engineering and research tools [e.g. ICEJAM (Flato and Gerard, 1986); RIVJAM (Beltaos, 1993, 1996); HEC-RAS (available online) plus several proprietary models developed by consulting engineering firms]. Since the late 1980s, dynamic models, both 1-D and 2-D are being developed with increasing sophistication. Clarkson University and the University of Alberta are two centres where ongoing model development, led by Professors H.T. Shen and F.E. Hicks, respectively, is generating important new capability and insights (e.g. Shen et al., 2000; Shen, 2002; Liu et al., 2006; She and Hicks, 2006; She et al., 2008). Accounting for seepage flow through the voids of breakup jams (Beltaos, 1999) has resolved the problem of implausibly high velocities that are otherwise computed under ice jam toes or at other areas where the rubble may ground. Adaptation of the constitutive law for unconsolidated sea-ice covers (Hibler, 1979, and several subsequent elaborations and improvements), which relates strain rates to internal stresses, has enabled dynamic simulation of the formation and evolution of freezeup and breakup ice jams. This adaptation reduces to the more familiar Mohr-Coulomb criterion when a steady condition is established (Pariset et al., 1966).

Figure 15 shows numerical results at a late stage in the development of an ice jam. The simulation clearly reproduces the downstream increase in thickness, particularly in the toe area, in accord with what is known from both measurements and 1-D models. At the same time, the 2-D model shows transverse gradients, with a general tendency for the jam to be thinner near the banks and thicker near midstream. This is a new insight, hinting at collection of relevant field or laboratory data. Comparison with model predictions would lead to either increased confidence in, or improvement of, the model equations.

Figure 15. Thickness contours generated by two-dimensional dynamic simulation of a major ice jam, Grasse River, NY. From Liu and Shen (2005) with changes.
Modelling capability also extends to the highly dynamic processes occurring during the passage of a jave, as illustrated in Figure 16. On this occasion, the form of a jave that occurred in the Saint John River, NB, was documented at three different locations downstream of the released jam (Beltaos and Burrell, 2005). All three waveforms were successfully simulated with suitable selection of model coefficients (She and Hicks, 2006). The model-data comparisons helped assess the significance of bank friction during the ice run, a force that had been neglected in earlier models.

![Figure 16. Simulation of a jave caused by the release of an ice jam in the Saint John River, NB. Modelling: She and Hicks (2006). Measurements: Beltaos and Burrell (2005). From She and Hicks (2006) with changes.](image)

The aforementioned models are all based on the assumption that an unconsolidated ice cover, be it moving or stationary, can be mathematically treated as a continuum, such that a set of partial differential equations can be used to describe its response to external forces. A parallel development is the Discrete Element Model (DEM), which does not need to invoke the concept of a granular continuum. The motion and eventual arrest of each block within a breakup jam are predicted during small time steps by computing the forces applied on each block by the water and by the surrounding blocks. This approach provides important insights as to both the evolution and the final configuration of an ice jam and enables calculation of the forces exerted by jams on structures (Daly and Hopkins, 2001; Hopkins and Tuhkuri, 1999; Hopkins and Daly, 2003).

4.3. Mitigation

There are many different measures that can be taken to mitigate the impacts of events caused by river ice, both structural and non-structural. These methods have been summarized and discussed in detail elsewhere (e.g. Burrell, 1995; Tuthill, 1995, 2005; White and Kay, 1997; Haehnel,
In general, the selection of a particular measure will depend on the type of problem and on local hydraulic and climatic conditions, while the degree of success is not always satisfactory. Two mitigation methods that have proven to be very effective in recent years are outlined below.

On small rivers, a relatively inexpensive and ecology-friendly ice control structure (ICS) has gained popularity in recent years. It consists of in-stream piers or other obstacles designed to arrest incoming ice floes while allowing water to pass through. In this manner, a jam forms upstream of a vulnerable reach, in an area where it can do little damage. Nearby floodplains can provide ice storage areas, while allowing excess water to bypass the structure at high flows (Figure 17). This concept was first implemented on the Credit River, Ontario, using piers spaced at 2 m, centre-to-centre (Cumming-Cockburn & Associates, 1986). Similar low-cost structures have since been built in the United States (Lever and Gooch, 2007; Lever and Daly, 2003; Tuthill and White, 1997). Cost is reduced by relying on extensive physical model testing to establish design parameters and determine the maximum distance between obstacles that will retain ice blocks and hold the resulting jam for sufficiently high flows. An excellent practice that has been established in the US is to systematically monitor and assess the performance of ice control structures. Based on extensive laboratory tests, Morse et al. (2003a) explored two new varieties of ICSs, using either widely-spaced piers or cylindrical steel ice booms, both supporting steel nets.

Figure 17. Conceptual drawing of an ice control structure which retains incoming ice blocks behind boulders in the main channel while relief flow bypasses the structure over the floodplain. From Tuthill and Lever (2006). Digital version was kindly supplied by the authors.
On the non-structural side, an “amphibious excavator” (or “amphibex”) has been successfully deployed to partially or entirely dislodge threatening ice jams. This is an amphibious vehicle that can break sheet ice and dredge accumulations of ice blocks or frazil slush, as a preventive or emergency measure. Unlike ice breakers, the amphibex can be deployed in rivers of ordinary depth, while the dredging capability provides an advantage over hovercraft, which are also used for ice breaking. An amphibex was successfully deployed on the Red River near Winnipeg in 2005 to dislodge a breakup jam, held in place by an ice sheet of limited length (300-400 m) and thickness of 0.5 m (Rick Carson, pers. comm. 2005). In a much more challenging application (~10 km of 0.5 to 1 m sheet ice, underlain by up to 4 m of slush), another unit (Figure 18) was able to cut a lane of open water along a hanging dam, resulting in much lower water levels. This ice formation was causing extensive and persistent flooding (~3 months) of the Fort William Historical Park, situated next to the Kaministiquia River near Thunder Bay, Ontario (Beltaos et al., 2007a).

![Figure 18.](image)

An Amphibex unit operating in the Kaministiquia River. Looking downstream, March 22, 2006. Note open lane behind vessel and slush being carried away by the current (from Beltaos et al. 2007a).

More recently, another unit helped relieve major flooding of the City of Prince George, BC, where a freezeup jam caused prolonged and extensive damages during December 2007 and January 2008.

4.4. Prediction of climate impacts on river ice processes

It was suggested earlier that forecasting of climate-induced changes to river ice regimes could be achieved by means of physics-based river ice models driven by hydrologic, and ultimately by climate models. General Circulation (or Global Climate) Models, also known as GCMs employ grid sizes that are too coarse for adequate representation of the hydrologic response of river
basins to climatic inputs. Resolving this problem is a matter of spatial downscaling, which is often addressed by means of statistical and empirical approaches. Such methods are highly site-specific and require good historical records. Another approach would be to utilize a Regional Climate Model (RCM). An RCM is a limited-area model, nested within a GCM, which is specifically designed to simulate the climate of a limited domain. It allows for physics-based and computationally affordable long-term integrations at high spatial resolution. Increasingly sophisticated and reliable GCMs and RCMs are being developed in several countries. In Canada, these initiatives are pursued at the Canadian Centre for Climate Modelling and Analysis (GCMs) and at OURANOS (RCMs). More detailed information can be found online, at:  
http://www.cccma.bc.ec.gc.ca/ and http://www.ouranos.ca/

5. Summary

River ice is an important feature and controlling factor in the regime and ecology of northern and not-so-northern rivers. Extremes of high water levels caused by ice jams and of low winter flows caused by ice cover formation; major frazil accumulations and clogging of water intakes; and sharp waves caused by ice jam releases, are some of the more notable phenomena associated with river ice. Such events can have serious economic and social impacts on nearby communities and infrastructure, and can adversely affect transportation and energy generation. Ecological impacts can be both detrimental and beneficial, depending on type of habitat and time of year.

River ice science is a very “young” endeavour. Despite solid advances, there are still many challenges associated with gaps in knowledge, some of them seemingly intractable at present. The emergence of climate change and variability as an issue of major importance, accentuates the need for improved knowledge and physics-based predictive capability. Site-specific, empirical approaches, which have been employed in the past to by-pass lack of relevant knowledge and/or data, may no longer be credible for extrapolation to future decades.

There is therefore considerable scope for research and development on river ice problems. Advanced technological “tools”, such as new instrumentation and modelling capabilities, remote sensing technology, and innovative mitigation methods, offer exciting opportunities for acquisition of new data and testing of new insights.

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Evolution of Frazil Ice

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Michel’s provocative statement in 1963 that “the phenomenon of frazil...is similar to the more general one of crystallization in a supersaturated medium” led eventually to Frazil Ice Dynamics (Daly, 1984). Frazil Ice Dynamics was the development of a comprehensive, quantitative model of the process of frazil ice formation by relating knowledge of industrial crystallization to the conditions present in natural water bodies. This model provided a framework for addressing the basic questions surrounding the first phase of frazil ice formation: where does the ice come from? How do the crystals grow? Can we predict the form of the crystal size distribution as a function of time and space? These are important questions, but not the only questions. One of the chief characteristics of frazil is its continuous evolution of form. Frazil rarely remains as a collection of isolated, disk-shaped crystals suspended in a turbulent fluid for long. The influence of gravity, the geometry of the waterbody, the level of turbulence, the meteorological conditions, and the inherent physics of ice all cause the ice to evolve in form. In general, the tendency is for frazil crystals to join together and form larger and larger units. However, this tendency is not linear or continuous, and at this point, not well described. In the presence of supercooled water, frazil crystals seem to readily join together to form flocs and when deposited, to form anchor ice. Later, when the water temperature has returned to the equilibrium temperature, frazil crystals, often in the form of slush at the water surface, display only a weak tendency to join together. They can remain as slush for long periods of time, and travel long distances, only slowly forming pans and larger flocs. Does our understanding of ice sintering help to explain these observations? New instrumentation has provided new and often startling observations of frazil in suspension in rivers, but the results also highlight the complexity of the formation process. Finally, although our understanding of frazil may not be complete, there are many important, practical problems caused by frazil that need to be addressed. The impacts on water intakes in lakes and rivers during periods of frazil formation are reviewed as well as the practical responses for detection and mitigation.
1. Introduction

Frazil ice is created when seed crystals of ice are introduced into turbulent supercooled water. The formation and evolution of frazil in river and streams, where it is ubiquitous, is the focus of this article. Rivers and streams are not the only environments where frazil is formed. Frazil is also formed in turbulent surface layers of ponds, lakes, and oceans when the wind creates turbulence (Svensson and Omstedt 1998); in oceans frazil can also form between waters of different salinities, and through brine drainage under sea ice (Martin 1981); in tunnels and penstocks (Ettema et al 2009); off of large ice shelves in the Antarctic (Holland and Feltham 2005); beneath glaciers (Lawson et al 1998); and even in industrial crystallizers (Smith and Sarofim 1979). Three broad stages of frazil development in rivers and streams are described in this article. In the first stage frazil ice crystals are created and grow in supercooled water. This stage is a dramatic period of ice creation, with the number of ice crystals increasing from near zero to immense multitudes. It is also during this stage that the crystals grow in size and begin to join together into distinct masses. This stage ends when the water is no longer supercooled and rapid growth and crystal creation ceases. In the second stage changes continue to occur to the frazil crystals on both the micro-scale (crystal level), and on the macro-scale (the formation of floes) as the crystals are transported by the rivers and streams in which they formed. The third and final stage of frazil formation begins when the frazil ice crystals stop moving and are incorporated into stationary ice covers.

First, this article provides an overview of the frazil ice evolution using the three stages of frazil evolution as a framework. The likelihood of micro-scale evolution in each stage is discussed as are the probable mechanisms for causing changes in the crystal morphology. The micro-scale evolution of frazil ice has not been investigated to any extent at this time. It is expected that the micro-scale evolution of frazil and the micro-scale evolution of snow crystals in snow banks are both driven by the same process: the drive by all the crystals to reach a state of minimum surface energy. The micro-scale evolution of snow (often referred to as isothermal or equilibrium metamorphosis) has been investigated (see for example Colbeck 1982, Blackford 2007, and Kaempfer and Schneebili 2007) and the results may provide insights into the micro-scale evolution of frazil. In both cases, the reduction in surface energy drives single crystals or small flocs of ice to take on a more spherical shape and causes “grain coarsening” of the crystal size distribution resulting in fewer and larger crystals. Interestingly, this parallelism between frazil and snow was first mentioned by Bernard Michel in 1966, who remarked that the metamorphism of frazil was due to a “process similar to that which happens in snow covers” (Michel 1966). However, even though both snow packs and frazil crystals are driven to reduce surface energy, the mechanisms by which they achieve this can be as profoundly different as their environments. Therefore as certain amount of caution is called for in this discussion until the observations, laboratory work, and theory has reached a more advance stage. Nevertheless, understanding the equilibrium metamorphosis of frazil is an important step in understanding the overall evolution of frazil ice. Next, the theoretical framework for describing the development of the frazil crystals size distribution in the first stage, Frazil Ice Dynamics (Daly 1984), is presented. Finally, two selected topics in frazil ice evolution are presented; the first, anchor ice and intake blockage, which have been combined because they both result from the same process; and the second, flocculation of suspended ice crystals.
2. Overview

This overview is organized around the three major stages in the evolution of frazil ice crystals in river and streams. In each stage the environmental conditions are described, then the crystalline physics which mediate and control the crystals response to those conditions, and finally the interaction and feedback between the frazil formation and channel flow hydraulics. A simplified diagram of frazil evolution is shown in Figure 1.

In the first stage the frazil crystals are created and grow in size. This stage is characterized by the presence of supercooled water, rapid crystal growth, and the explosive creation of new crystals through secondary nucleation. The supercooled water temperature of each parcel of water represents a dynamic balance between the latent heat released by the growing crystals and the heat transfer from the water surface. It is dynamic because the degree of supercooling continually changes with time as the number and size of the frazil crystals in suspension changes with time. The dynamic supercooling curve has a classic form, with a definite maximum and a finite duration. (See for example Carstens 1966, Hammar and Shen 1995, and Ye and Doering 2004)

The anisotropic interface kinetics of ice mediate the crystal growth and leads to the formation of disk shaped crystals, the ubiquitous shape by which frazil is chiefly known (Daly and Colbeck 1986). This anisotropic growth produces a crystal shape that is far from the spherical shape of an ice crystal at equilibrium – that is, with minimum surface energy for the volume enclosed. The process that pushes the crystal towards equilibrium shape, and lead to “equilibrium metamorphosis”, is apparently overwhelmed during this period of rapid growth, and exerts little influence on the crystal form during this stage (Colbeck 1992). Frazil crystals suspended in supercooled water have long been called “active” to describe their ability to stick to any and all unheated underwater objects, including other crystals. Anchor ice, intake blockage, and flocculation into larger masses are some of the results of this stickiness of frazil in supercooled water. A fair amount of laboratory and theoretical research has been done to understand this stage. Frazil Ice Dynamics (Daly 1984) is a quantitative frame work for describing the time evolution of the size distribution of frazil crystals during this first stage.

The frazil crystals that are formed in this stage are largely suspended in the flow and are usually treated as passive tracers which exert little influence on the fluid flow. Anchor ice blocks a portion of the channel cross section and modifies the effective channel roughness (Kerr et al 2002). The impact of these changes on the channel flow conditions varies with the extent of anchor ice coverage in a given reach. In some cases, the build up of anchor ice causes upstream water levels to rise and flooding to occur (Daly 2005). Heat transfer out of the water surface to the atmosphere during the formation of supercooled water is a stabilizing buoyancy flux into the water (Turner 1979). If not countered by turbulent mixing of the flow, this buoyancy flux leads to a stratified water column with the coldest and least dense water at the surface and the initial
ice formation limited to a thin surface layer. Frazil formation occurs in reaches where turbulence generated by the water flow entrains the supercooled water throughout a significant portion of the depth of flow. Shallow, rapid flowing reaches are therefore prime areas, but frazil can be formed in any reach where the vertical turbulent mixing is sufficient to overcome the buoyancy flux into the water caused by heat transfer out of the water surface.

The second stage follows the first in time and begins when the dynamic supercooling curve has reached its end and the water temperature is effectively at or very near to the ice/water equilibrium temperature. In this second stage the ice crystals are advected by the river flow velocity and are continuously in movement either in open water or under stationary ice covers. Without supercooled water, crystal growth ceases and new crystals are no longer created, but the process of evolution still continues, with changes occurring on both the micro-scale, and on the macro-scale. Little is known about how the crystals evolve on the micro-scale (or crystal level) during this stage. Processes that occur over long time scales, such as equilibrium metamorphosis and grain coarsening, probably play an important, although currently undetermined role during this stage in controlling the crystal size and shape. Equilibrium metamorphosis leads to a spherical crystal form and results from variations in the crystal surface temperature due to the surface curvature (Colbeck 1992) and other processes (Kaempfer and Schneebili 2007). During equilibrium metamorphosis each individual crystal grows towards a spherical shape. The spherical shape is favored because it has the lowest surface energy for the volume of ice. Grain coarsening, the increase in size of larger crystals and melting away of smaller crystals is also driven by radius effects and results in the reduction of surface energy over the entire size distribution of crystals (Colbeck 1986). All the micro-scale evolution processes together lead to fewer, larger, more spherical crystals.

The macro-scale evolution results from the consolidation of ice crystals on the water surface due to buoyancy, surface convergence, buffeting by surface eddies, and mechanical interaction of the
crystals, especially above the water line. Solidification of the interstitial water between the consolidated crystals through heat loss to the atmosphere creates floes with increasing strength and rigidity. In highly energetic streams, the frazil crystals at the surface may never consolidate and the frazil will stay in the form of slush. In less energetic cases circular or “pancake” floes form with diameters of a meter or more. In slow moving streams very large floes can form and their effective diameter can be on the order of the channel width (Osterkamp and Gosink 1983).

Ice moving at or near the flow velocity has little impact on the channel flow conditions. The possibility for ice impact on the flow begins when the surface concentration and strength of floating ice increases to the point where significant shear stresses can be transmitted through the surface ice. The shear stress causes the velocity of the floating ice to slow relative to the water velocity, and this slowing in turn exerts stress on the flowing water, impacting the discharge rate and stage. In some cases, the surface ice can stop moving altogether and bridging occurs. Ice motion can also be arrested by ice control structures, obstacles such as bridge piers, an intact ice cover and other causes.

The third and final stage begins when the moving ice comes to rest in a stationary ice cover. The actual formation mechanism of the ice cover depends on the form of the frazil when it arrives at the stationary ice cover, the hydraulic conditions at the leading upstream end of the ice cover, and the heat loss rate to the atmosphere. The initial form of the ice cover can change abruptly through shoving or consolidation events that start immediately after the cover is formed and continue for some period of time (Hicks 2009). The strength and thickness of the ice cover increases as the interstitial water between frazil ice crystals solidifies through heat loss from the cover surface to the atmosphere (Calkins 1979). The evolution of the frazil crystals probably does not end when the frazil ice crystals are incorporated into the river ice cover. Grain boundary diffusion and other processes at the crystal level would tend to increase the crystal size with time; although the rate at which this metamorphosis would occur has not been estimated (Blackford 2007).

The ice cover makes a portion of the channel unavailable for flow and changes the flow characteristics of the channel by presenting an additional stationary, rough boundary, which modifies the channel wetted perimeter and hydraulic radius. The effects of these changes on steady flow are well documented (Ashton 1986). The ice cover's inertia and resistance to bending can significantly modify wave celerity and attenuation in ice covered channels (Daly 1993, Xia 1998). The formation of ice covers can lead to large transients in flow that cause upstream stages to rise and downstream stages to fall (Wuebben et al 1995). The breakup of river ice covers can also induce large transients in flow, especially when ice jams form and release. (Beltaos ed. 2008)

3. Frazil Ice Dynamics

Frazil Ice Dynamics (Daly 1984) is a quantitative framework for describing the time evolution of the size distribution of frazil crystals during the first stage of frazil ice when the crystals are actively growing and new crystals are being created. It allows concepts and knowledge of industrial crystallization to be adapted to the conditions present in natural water bodies. It is rooted in statements such as “the phenomenon of frazil...is similar to the more general one of crystallization in a supersaturated medium” (Michel 1963). Frazil Ice Dynamics constructs a
crystal number continuity equation that is well adapted to describe crystal creation and growth during the period when the frazil ice crystals exist underwater as a single individual crystal held in suspension by the liquid water. Later stages of frazil evolution, when the crystals have flocculated together into larger masses, are not so well described by this approach and require other conceptual frameworks.

The crystal number continuity equation describes the crystal size distribution per unit volume. We can use it to bring into play all the information we have regarding the specific mechanisms controlling crystal creation and growth. First a continuous function \( n(r,t) \) is defined as the population density of crystals in the volume \( R \) of a specific size \( r \) at a time \( t \). (Note that the dimensions of \( n \) are the number of crystals per length per unit volume or \( \#/L^4 \))

\[
N(t) = \int n(r,t)\,dR
\]

(1)

\( N(t) \) is the total number of crystals in the volume \( R \) at time \( t \). Crystal creation and seeding (the introduction of crystals from outside of the volume of interest) may cause the sudden appearance (often referred to as the “birth”) of crystals in the volume \( R \). Convection, flocculation or other processes may cause the disappearance (or “death”) of crystals of a specific size. The net appearance in an incremental region \( dR \) at a time would be \((B-D)dR\), where \( B(R,t) \) and \( D(R,t) \) represent birth and death functions. The population balance of crystals in some fixed region \( R \), which moves convectively with the particle velocity, can be defined as

\[
\frac{d}{dt}\int_R n\,dR = \int (B - D)dR.
\]

(2)

Expanding the first term using Leibnitz’s rule, noting that the region \( R \) was arbitrary, and letting \( G(r,t) \) be the growth rate of the ice crystal, we then see that

\[
\frac{\partial n}{\partial t} + \frac{\partial}{\partial r}(Gn) + D - B + \nabla \left( \vec{V}_e n \right) = 0.
\]

(3)

where \( \vec{V}_e \) = the external convective velocity. This is the number continuity equation in continuous form. The discrete form of this equation can be written by assigning each crystal to a specific size class; with the size classes selected using some arbitrary but useful scheme. The total mass of ice can now be written in continuous form as

\[
M(t) = \rho k_v \int_{r_{\text{MIN}}}^{r_{\text{MAX}}} n(r) r^3 \,dr
\]

(4)

where \( r_{\text{MIN}} \) and \( r_{\text{MAX}} \) define the largest and smallest radii of the frazil size distribution; and \( k_v \) = a “shape” factor that relates the radius of the particle and the volume. (For example, \( k_v \) for spheres would be \( 4\pi/3 \), and for cubes, 1). The above equation can be restated in discrete form as
\[ M(t) = \rho k \sum_{i=1}^{N} n_i r_i^3 \]  \hspace{1cm} (5)

In this case there are \( N \) classes of crystals, and each class has \( n_i \) crystals and a mean size of \( r_i \).

3.1 Frazil Ice Creation

At this point, we can turn our attention to the birth, death, and growth functions of frazil, starting with the birth. For many years the origin of frazil ice under natural conditions was debated. One early rather straightforward hypothesis was that once the water temperature became sufficiently supercooled, the frazil crystals would spontaneously appear. The process of spontaneous appearance is called nucleation. Nucleation is a general term referring to the formation of a new phase from a parent phase. In our case, the parent phase is obviously water, and the new phase that is appearing is, of course, ice. Once the nucleation process of ice was studied in detail, however, it soon became clear that spontaneous nucleation of ice was probably not possible. Homogeneous nucleation, that is the spontaneous appearance of ice in pure water containing no dissolved impurities nor undissolved particles such as dirt, bacteria, etc., can not occur until supercoolings of near 40°C are reached, far beyond the temperatures found in natural waterbodies. Heterogeneous nucleation, the spontaneous appearance of ice caused by the actions of catalysts, (also called nucleating agents), seemed a more likely source. Known catalysts for the formation of ice include silver iodide, organic particles of all types, and especially bacteria. As natural water bodies are not pure, heterogeneous nucleation would be demonstrated if the particular catalyst that was effective at the relatively low supercoolings found in nature could be identified. To date, however, no catalysts have ever been identified that are effective at these low supercoolings. At present, the most effective known catalysts are certain bacteria that can nucleate ice at supercoolings of about 1°C. These bacteria are probably not effective in rivers and streams, because 1°C is a full order of magnitude greater then the supercooling levels found in nature. In addition, spontaneous nucleation has never been observed in any carefully controlled laboratory experiment at these temperatures (Heneghan et al 2002, Wilson et al 2002). As a result, the hypothesis of spontaneous nucleation has been set aside and replaced by the concepts of seed crystals and secondary nucleation.

Seed crystals are ice crystals that are introduced from outside the natural water body to begin the process of frazil ice formation. Seed crystals can come from a number of different sources: vapor evaporating from the water surface on encountering cold air can sublimate into ice crystals, which fall back onto the water surface and are entrained by the turbulent motion of the flow; small water droplets generated by breaking waves, bubbles bursting at the water surface, and splashing; snow and sleet. Undoubtedly other sources of seed crystals can be identified, and at least one source is present where ever frazil ice formation is observed. But seed crystals are not the whole story. It is observed that once a very few seed crystals are introduced into turbulent supercooled water, very quickly many, many crystals are created. The process by which these new crystals are created is called secondary nucleation.

Strictly speaking, secondary nucleation refers to the formation of new crystals because of the presence of ice crystals and does not refer to the particular mechanism that creates new crystals.
Secondary nucleation produces very small crystals, often referred to as crystal “nuclei,” that can grow into new crystals. These new crystals can then further increase the rate of secondary nucleation with a multiplicative effect. The processes that govern the rates of secondary nucleation are poorly understood. Two general mechanisms of removal of the nuclei from the surface of the parent crystals have been suggested: collisions of the crystals with hard surfaces (including other crystals) and fluid shear. Because the ability of fluid shear to produce nuclei has never been demonstrated for ice it is generally accepted that collisions are the most likely mechanism for producing new crystals. From their experimental work, Evans et al. (1974a, b) concluded that the secondary nucleation of ice was limited by the collision rate. Therefore, it was possible to determine the overall nucleation rate $\dot{N}_r$, with two or more collision mechanisms, as the linear sum of the actual nucleation rate attributable to each collision mechanism ($\dot{N}_i$)

$$\dot{N}_r = \dot{N}_1 + \dot{N}_2 + \ldots \dot{N}_i$$  \hfill (6)

The nucleation rate of each mechanism of collision can be expressed as the product of three functions (Botsaris 1976)

$$\dot{N}_r = (\dot{E}_i)(F_1)(F_2)$$  \hfill (7)

where

$\dot{E}_i = \text{rate of energy transfer to crystals by collision}$

$F_1 = \text{number of particles generated per unit of collision energy}$

$F_2 = \text{fraction of particles surviving to become nuclei}$

At this time the values of $F_1$ and $F_2$ must be determined empirically. Therefore, to simplify matters, we can combine $F_1$ and $F_2$ and equation (7) is then rewritten as

$$\dot{N}_r = \dot{E}_i Z_N$$  \hfill (8)

where $Z_N = (F_1)(F_2)$. We expect that $Z_N$ to be a function of all the parameters that govern the surface morphology and the crystal growth, including supercooling, impurity concentrations, turbulence level, etc. (see Clark and Doering (2008) for experimental investigation of secondary nucleation.) However, given the little information available today, the value of $Z_N$ is not known very precisely, and is generally treated as a material property of ice with a constant value. The overall rate of secondary nucleation can be written as

$$\dot{N}_r = Z_N \left( \dot{E}_{i1} + \dot{E}_{i2} + \dot{E}_{i3} \ldots \right)$$  \hfill (9)

where $\dot{E}_{i1}, \dot{E}_{i2}$ etc. are the energy provided by each mechanism of collisions. Modeling the actual kinetics of each collision mechanism will not be discussed in detail here. Several possible collision mechanisms can be identified: collision between crystals caused by differential rising rates; collision between crystals caused by turbulent shear; collisions of crystals with hard
boundaries; etc. Once a collision mechanism has been identified, estimating the energy produced by the collision is not difficult.

3.2 Frazil Ice Crystal Growth

During the process of secondary nucleation, not only are new crystals being created, the new crystals are actively growing in size. The two primary processes that can potentially control the rate at which an ice crystal grows are the crystallization kinetics of ice and the transport of latent heat away from the surface. The crystallization kinetics (also referred to as the interface kinetics) of ice has been studied both theoretically and experimentally. At the crystal level, ice has two principal growth directions: $a$-axis and $c$-axis. The crystallization kinetics of each growth direction appears to be different. Growth in the $c$-axis probably proceeds by surface nucleation for perfect crystals and by a dislocation mechanism for damaged crystals. The interface kinetics of $a$-axis growth has not been completely defined; the mechanism is probably that of continuous growth. However, it appears that the kinetics is very fast and that for practical purposes the growth rate of the $a$-axis is totally controlled by the rate at which latent heat is transported away from the interface (Shibkov et al 2003). Growth along the $c$-axis is much slower than that along the $a$-axis for all sizes of crystals. This implies that $c$-axis growth is controlled by the crystallization kinetics (Forest 1986). As a result of these two different growth mechanisms, frazil ice crystals grow as thin circular disks with a major diameter that is 5 to 15 times greater than their thickness. Another result is that the growth rate of the major diameter can be estimated by determining the transport rate of the latent heat away from the growing crystal making the problem of determining the growth rate of frazil crystals a problem of heat transfer. Thus an “engineering” type of description of the growth rate, $G$, along the $a$-axis can be written

$$G = \frac{h}{\rho c_\lambda} (T_M - T)$$

(10)

where $h$, the heat transfer coefficient, is a function of the crystal size $r$ and the level of turbulence as determined by the rate of energy dissipation, $\varepsilon$; $T_M$ is the ice water equilibrium temperature, and $T$ is the actual bulk temperature of the fluid. The difference $(T_M - T)$ defines the supercooling of the mixture. A detailed discussion of the value of the heat transfer coefficient will not be undertaken here (see Holland et al 2007). Reasonable estimates of the heat transfer coefficient can be made by investigating the interaction of the ice crystal and the fluid turbulence. Turbulence can be visualized as numerous interacting eddies of all possible scales. The very largest eddies originate directly from the instabilities of the mean bulk flow. The scale and orientation of these largest eddies are imposed by the geometry of the flow situation. Energy is extracted from the large eddies through the inertial interaction of these eddies with smaller eddies. The energy cascade is not affected by the fluid viscosity until the smallest scales are reached, where this energy is dissipated by the viscosity. At equilibrium, the dissipation rate must equal the rate at which energy is supplied to the small-scale eddies. The dissipation rate $\varepsilon$ and the fluid dynamic viscosity $\nu$ form a length scale such that

$$\eta \sim \left(\frac{\nu^3}{\varepsilon}\right)^{1/4}$$

(11)
where $\eta$ is the dissipation length scale or the Kolmogorov scale.

If the crystal size is small relative to the Kolmogorov length scale, it is in the dissipative regime. In the dissipative regime the fluid eddies are strongly dampened and dissipated by the fluid viscosity. In effect, the crystal is smaller than the smallest scales of the turbulent eddies. It does not experience the turbulence as interacting eddies but rather as a fluid motion that varies linearly with position. In the limit as the particle size approaches zero relative to the Kolmogorov length scale, the heat transfer rate will approach that of pure diffusion from a point source. At larger sizes the heat transfer rate will be modified both by the crystal shape and the flow field around the crystal and in general the heat transfer rate will be inversely proportional to the crystal size.

If the crystal size is large relative to the Kolmogorov length scale, it is in the inertial regime. In the inertial regime, the ambient velocity can be characterized in many different ways, each corresponding to a different eddy size. It seems reasonable to assume, following Wadia (1974), that the predominant shear that the particle will experience will be produced by eddies closest to the particle that are of the same size as the particle. Eddies that are significantly larger than the particle will entrain both the particle and the fluid around it. Very small eddies relative to the particle size may enhance the overall transport by some mechanism of renewal of the boundary layer surrounding the particles, but eddies of a size comparable to the size of the particle that will cause the most significant gradients near the crystal surface. In this case the heat transfer rate will approach a constant value, independent of the crystal size.

### 3.3 Frazil Ice Dynamics Summary

Continuing, an energy conservation equation can now be combined with the crystal number continuity equation, equation (3), to arrive at

$$
\frac{\partial T}{\partial t} = \frac{\phi}{\rho C_p} + \lambda \frac{\partial}{\partial r} \int_{r_{min}}^{r_{max}} n(r) r^3 dr
$$

(12)

$$
\frac{\partial n}{\partial t} + \left( \frac{T_m - T}{\rho_0 \lambda} \right) \frac{\partial}{\partial r} (hn) = \dot{N}_r (r_{sat}, t) + \dot{N}_i (r_{seed}, t)
$$

(13)

where $\phi$ is the heat loss rate per unit volume; $\rho$ and $C_p$ are the water density and heat capacity respectively; $\dot{N}_r$ is the rate of creation of new crystals at the size $r_{sat}$ through secondary nucleation; and $\dot{N}_i$ is the number of crystals of size $r_{seed}$ introduced through seeding. Note that there is a small inconsistency between equations (12) and (13). This inconsistency results from the fact that not all of the change in mass of ice accounted for in equation (13) resulted from the release of latent heat; some resulted from the introduction of seed crystals, and some “re-distribution” of mass occurred because of secondary nucleation, both of which are shown in (13). However, we can also restate (12) in the following way to eliminate any inconsistency:
In this case $k_A$ is the “surface area” factor which relates the surface area of the crystal to its radius. These equations are augmented by additional equations such as a salt conservation equation for application to oceans; a fluid model to describe fluid flow in rivers; a turbulence model to estimate sub-grid turbulence parameters in rivers, lakes, and oceans; and equations of state to describe the ice/water equilibrium temperature as a function of pressure and salinity. Generally, there is no “feedback” from the frazil number continuity equation to the fluid model as it generally assumed that the frazil ice in suspension acts as a “passive tracer” and has no influence on the fluid turbulence.

In the above discussion we have outlined the physical processes that occur during the formation phase of frazil ice. There are essentially four independent parameters that control the rate of formation:

1. The heat transfer rate from the volume of interest, $\phi$. The heat loss can occur in a variety of ways. In rivers, lakes, and oceans the heat loss generally occurs through the water surface to the atmosphere. The presence of an ice cover at the water surface will reduce the heat loss rate and prevent the formation of supercooled water. Modification of the freezing point of ice due to changing pressure can result in an effective heat loss in systems comprised of suspended frazil and water, even if there is no net heat transfer from the volume of interest. This can occur in penstocks and tunnels with large vertical drops, beneath glaciers, and under large ice shelves in the Antarctic.

2. The rate of introduction and size distribution of the seed crystals that are introduced into the control volume. This is described by the parameter $IN$. Generally, it is assumed that seed crystals are created in the frigid air and seeding occurs due to ice crystals entering at the water surface. In the case of penstocks and tunnels the seed crystals must enter with the flow. In some cases the actual source of seed crystals is not clear. In any case, no matter how unclear the origin of seed crystals in any particular situation where frazil ice exists, the temptation to suppose that ice crystals are spontaneously nucleated from the liquid should be resisted. It is incumbent on any researcher suggesting such a mechanism to explain how spontaneous nucleation could possibly occur.

3. The turbulent intensity rate as defined by the rate of energy dissipation, $\varepsilon$. This last influences the heat transfer from rate from the suspended ice crystals and the energy of collision of crystals during secondary nucleation. In general, it has been observed that high levels of turbulence results in a large number of small disk shaped crystals and low levels of supercooling. Low levels of turbulence result in few suspended crystals and higher levels of supercooling.

4. The rate and size distribution of new crystals produced per unit of collision energy during secondary nucleation, $ZN$. While this cannot be strictly considered a material property of ice, (it is also dependent on the supercooling level, the impurities in the water, etc.) it is generally treated as such and probably rightfully so given our current level of knowledge regarding its value.
Examples of numerical modeling of this first stage of frazil formation, using a variety of approaches, can be found in the literature (Mercier 1984, Omstedt 1994, Hammar and Shen 1995, Ye and Doering 2004).

### 3.4 Anchor Ice and Intake Blockage

Anchor ice formation and water intake blockage are both initiated when frazil ice crystals suspended in supercooled water are deposited on the channel substrate, in the case of anchor ice, or the protective trash rack bars of a water intake, in the case of intake blockage. Supercooled water is required for build up of frazil ice to occur. First, unless heated, all the underwater surfaces of an intake or the channel substrate which are in contact with supercooled water are quickly cooled to below freezing. Once the temperature of a surface is below freezing, however slightly, ice can adhere to that surface. The adhesion force of ice is a function of the surface material and the ice/surface temperature below freezing. The magnitude of the adhesion force, even at the very small levels of supercooling found in nature, can be substantial. Further growth of the deposited ice occurs either by continued deposition of frazil crystals or by crystal growth through the release of latent heat to the supercooled water or possibly by a combination of the two.

Observations and laboratory experiments suggest that anchor ice formation and intake blockage in rivers typically results from deposition of frazil ice crystals (Kerr et al 2002). Anchor ice in rivers is typically composed of many small crystals and often has a milky appearance. In some cases the anchor ice includes sediment deposited along with the ice crystals and takes on a brownish appearance. Frazil build up on river intakes is generally thought to result from frazil colliding with, and adhering to, trash rack bars (Daly 1987, 1991, Svensson and Anderson 1988, Andersson and Andersson 1992, Andersson and Daly 1994). The masses of ice collected on water intake trash racks in rivers display little evidence that the ice crystals grew after they had been deposited. (Figure 2.)

In lakes, anchor ice formation and intake blockage result from initial adhesion and continued growth through the release of latent heat (Daly and Ettema 2006). Observations of anchor ice in the nearshore zone of southern Lake Michigan suggest that, unlike rivers, accretion does not dominate the formation of anchor ice, but rather the growth of individual crystals is the important process. Kempema et al (2001) found that the majority of the anchor-ice formations comprised single crystals of ice, 3-6mm in thickness, and 75-125mm in length, growing from the lakebed. Their observations are consistent with the few available observations of blocked intakes in Lake Michigan by divers, who all report large, individual crystals of ice on the intake structure. (Figure 3.)

This difference in the form of anchor ice and the frazil ice build up on intakes between rivers and lakes undoubtedly reflects the difference in the environmental conditions between river and lakes, especially the level of turbulence and perhaps the seeding rate. There is nothing known about the seeding rates in either case, but the rate of energy dissipation by turbulence can be
Figure 2. River water intake. Accumulated frazil on trash racks lifted from the water. (Daly 1991)

Figure 3. Lake water intake. Large platelet ice observed on water intake in Lake Michigan. (Source: http://www.youtube.com/watch?v=COMV3UEE47A)
estimated in both cases. Turbulent dissipation in lakes is created by the wind drag on the water surface, and it is estimated to range from $10^{-11}$ to $10^{-8}$ W kg$^{-1}$ (Wuest and Lorke 2003). Turbulent energy dissipation in rivers is substantially larger. In rivers, the turbulent dissipation rate roughly equals the rate of work done by the moving water, and it is estimated to range from $10^{-5}$ to $10^{-1}$ W kg$^{-1}$ (Nezu and Nakagawa 1993). Existing models of frazil development indicate that higher levels of turbulence lead to greater rates of heat transfer from suspended ice crystals and as result, increased crystal growth rates, and increased numbers of crystals created through increased rates of secondary nucleation (Ettema et al. 1984, Daly 1984). In rivers, as a result, the high levels of turbulence lead to relatively high concentrations of crystals and, because of the enhanced crystal growth, relatively low levels of supercooling. In lakes, on the other hand, the low levels of turbulence lead to relatively low concentrations of crystals that are available for deposition; and because of the low rates of crystal growth, relatively high levels of supercooling. This is qualitatively consistent with the different forms of anchor ice and frazil build up on intakes seen in the two environments.

4. Flocculation

Flocculation describes the process through which particles of any type join together to form larger masses. Generally, flocculation refers to particles bonding together while suspended in the liquid. The formation of large flocos at the water surface through the freezing of the interstitial water between the consolidated frazil ice crystals through heat transfer to the atmosphere would not be considered flocculation, for example; nor would the formation of anchor ice through continual deposition of frazil crystals. While there has not been systematic observations made of frazil flocculation in the field it has been studied and observed in the laboratory. The existence of frazil flocos has long been recognized and observed in rivers and streams (Figure 4). Sintering is often provided as the mechanism of flocculation of frazil crystals (Martin, 1981, Mercier 1984, Clark and Doering 2008). Sintering is a general term that usually refers to “bonding particles into a coherent, predominantly solid structure via mass transport events that...leads to...lower system energy” (Germain 1996). Generally, it is thought that the reduction in overall surface energy crystals brought together drives the crystals to join together, or sinter, into one larger floc. Observations and theory of the sintering of ice have a long history, going back to the observations by Faraday (1859, 1860). The equilibrium metamorphism of snow covers is thought to be based on the fundamental process of sintering. (Kaempfer and Schneebeli 2007)

Crystals which are brought together in the presence of supercooled water can join by freezing together through heat transfer from the contact area to the surrounding supercooled water. While this process can be considered sintering, it does not result from the reduction in the overall crystal surface energy typically used to explain sintering, but rather through the transfer of latent heat away from the crystals. If the water is not supercooled, it is not clear that sintering can occur at all. As implied by Colbeck (1997) and others, the bonds between ice crystals brought together in the presence of water are inherently unstable. This can be seen by reviewing the phase diagram of water. Consider the interface temperature of two crystals that are brought together under some slight pressure. The phase diagram shows that the temperature of the contact area must decrease as pressure is applied. If the interface temperature is decreased, heat is transferred to that location from the slightly warmer environment surrounding the interface, causing ice to melt. The interface is therefore inherently unstable, and bonds are unlikely to form between ice
crystals unless some external cause transfers heat away from the crystals. This inherently unstable interface has been used to explain observations that snow crystals saturated with water, that is slush, have “virtually no mechanical strength” (Blackford 2007) and that sintering of ice slush is not possible (Colbeck 1997).

Figure 4. Frazil Floc. Maximum width on the order of 5 mm. (Image by the author)

5. Summary

This article addressed the evolution of frazil ice in rivers and streams. It posited three stages of frazil formation: the first stage when ice crystals are created and grow in supercooled water; a second stage when the crystals are transported by the flow and evolve on both micro-scales and macro-scales; and a final third stage when the crystals are incorporated into stationary ice covers. The equilibrium metamorphosis of snow packs was used to provide insight into the micro-scale evolution of frazil, but the profoundly different environment of ice crystals in turbulent rivers and in snow banks suggests that caution must be exercised and the insights not pushed too far. The metamorphism of frazil ice crystals is an area of research where much exciting work remains. The theoretical framework for describing the development of the frazil crystals size distribution in the first stage was described. This framework provides a means of quantifying any insight into the processes that control ice crystal creation and growth. Next selected topics in frazil ice evolution were presented: anchor ice formation and intake blockage and flocculation of suspended ice crystals.

6. References

Andersson, A., and S.F. Daly (1992) Laboratory investigation of trash rack freezeup by frazil ice. USA Cold Regions Research and Engineering Laboratory, CRREL Report 92-16


Colbeck, S. (1997) A review of Sintering in Seasonal Snow. USA Cold Regions Research and Engineering Laboratory, CRREL Report 97-10, Hanover, NH


Daly, S.F. (1991) Frazil Ice Blockage of Intake Trash Racks. USA Cold Regions Research and
Engineering Laboratory, Cold Regions Digest No 91-1, March 1991


Daly, S.F. (1984) Frazil Ice Dynamics. USA Cold Regions Research and Engineering Laboratory, CRREL Monograph 84-1.


River ice, glaciers, and climate change
Global warming is now generally acknowledged to be having widespread impacts on many aspects of water resources including river ice covers in many countries around the world. To determine changes in ice cover regime in a region of BC where river ice cover formation in winter is normal but also problematic in some years due to mechanical breakup of ice covers in mid-winter or freeze-up at high discharges, published Water Survey of Canada data on active gauging stations in the region with more than 30 years of record were investigated to determine ice cover trends. A total of 22 stations on 18 different streams that met the four study criteria of little or no missing data over the annual ice cover period, no changes to gauge location over the period, no major disturbances in the watershed, and having some ice cover in all or nearly all winters were analysed. The 4 indicators of change in annual ice cover ultimately chosen from the readily available published streamflow data were: time from first ice to last ice, longest continuous ice cover duration, date of first ice cover, and date of last ice cover. The data for individual streams showed very weak linear regression correlations between all 4 dependent variables with time over the 30 year period from 1976 to 2005 due to the large variation in the 4 indicators from year to year. However, as a group, they showed clear regional trends: first ice to last ice cover interval is growing shorter (19 support and 1 contrary), longest continuous ice cover duration is getting shorter (22 support), first ice effect day is occurring later (19 support and 3 contrary), and last ice effect day is occurring earlier (16 support and 5 contrary). A look at the same ice cover data for 7 of these 22 stations over periods of record longer than 30 years show the same results for 3 of the 4 indicators.
1. Introduction

In the southern interior of British Columbia (BC) from time to time, mid-winter breakups of competent ice covers cause flooding and damage to man-made structures both in channels and on floodplains as well as degrade habitat. Less frequently, but just as destructively, freeze-up ice jamming on certain rivers in southern BC produces similar results. Figure 1 shows the area in BC encompassed by this study. Weather prior to, during, and for many weeks after freeze-up plays a large role in determining if ice runs and/or jamming will be problematic in any given winter. On regulated rivers, heavy precipitation for as long a period as an entire year can markedly increase the probability of having a problem ice jam develop the following winter.

Climate change is an acknowledged global phenomenon that is impacting many aspects of life already and will continue to have greater impacts as the 21st century unfolds (Lemke et al., 2007, Milly et al., 2008). The climate in BC has been undergoing noticeable change during recent decades and climate change is projected to continue (or intensify) for many more decades (BC Ministry of Environment, 2007; Morrison et al., 2002; Cohen et al., 2004).

The objective of this paper is to document how these climate changes have already affected river ice cover formation and duration in southern BC in very simple terms and then project how the regional ice cover might continue to adjust in coming decades as the climate continues to change. This initial review of annual ice cover data was to determine if any trends in the regional ice cover existed. If so, then more in-depth analyses of the ice cover changes taking place in the region would be warranted to better envisage how climate change may influence ice jam flooding and riverine/riparian ecology over the next century.

2. Methodology

Long-term Water Survey of Canada (WSC) gauging stations provide data on winter ice cover conditions on streams throughout Canada. The most readily available data about ice conditions at WSC gauging sites is the notation on published data that daily discharge in a stream has been deemed to have been affected by ice. This notation that ice has affected the discharge on any day in some fashion then forms a data set of “ice-affected” daily discharges for each winter season at all WSC gauging stations. This annual set of “ice-affected” daily discharges at a selected gauging site was analyzed to determine if a trend in relevant “ice-affected” variables existed at that site over time. The term “ice cover” is used for “ice-affected” discharge for simplicity in this paper.

All active WSC gauging stations in the southern interior of BC were evaluated for acceptability for use in this study. The criteria for station inclusion were data back to at least 1976 with no change in site location, no significant missing data, unregulated winter flow, and very few years of zero ice cover. A total of 22 stations on 18 streams in the southern interior met these criteria. Seven of these 22 streams had longer acceptable record that was also analyzed for trends in the same variables. Table 1 lists the 22 stations used in the study.
To determine temperature changes from 1976 to 2005 over the ice cover season (November through March) in the southern interior of BC, monthly temperature averages at 3 long-term climate stations that had no missing data and had not been re-located during this time were used in the study. The averages for each of the 5 winter month temperatures were then averaged to get an annual winter average temperature for each climate station. Table 2 lists the 3 study stations that are representative of the climate in the study area. Any trends determined during the recent 30 years could then be projected forward based on continued warming projected during this century.
Table 1. Water Survey of Canada streamflow gauges used in the study.

<table>
<thead>
<tr>
<th>Station Name</th>
<th>WSC Sta. No.</th>
<th>Drainage area (km²)</th>
<th>Analysis Period</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Short Period Analysis</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>McHale River</td>
<td>08KA009</td>
<td>254</td>
<td>1976-2005</td>
</tr>
<tr>
<td>McGregor River @ Lower Canyon</td>
<td>08KB003</td>
<td>4,780</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Quesnel River near Quesnel</td>
<td>08KH006</td>
<td>11,500</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Clearwater River near Clearwater Sta.</td>
<td>08LA001</td>
<td>10,200</td>
<td>1976-2005</td>
</tr>
<tr>
<td>North Thompson River @ Birch Island</td>
<td>08LB047</td>
<td>4,450</td>
<td>1976-2005</td>
</tr>
<tr>
<td>North Thompson River @ McLure</td>
<td>08LB064</td>
<td>19,600</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Barriere River below Sprague Creek</td>
<td>08LB069</td>
<td>624</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Bessette Creek above Beaverjack Creek</td>
<td>08LC039</td>
<td>603</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Salmon River @ Falkland</td>
<td>08LE020</td>
<td>1,040</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Seymour River near Seymour Arm</td>
<td>08LE027</td>
<td>806</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Nicola River near Spences Bridge</td>
<td>08LG006</td>
<td>7,280</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Coldwater River near Brookemere</td>
<td>08LG048</td>
<td>316</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Nicola River above Nicola Lake</td>
<td>08LG049</td>
<td>1,500</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Ashnola River near Keremeos</td>
<td>08NL004</td>
<td>1,050</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Similkameen River near Nighthawk</td>
<td>08NL022</td>
<td>9,190</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Tulameen River @ Princeton</td>
<td>08NL024</td>
<td>1,760</td>
<td>1976-2005</td>
</tr>
<tr>
<td>Similkameen River near Hedley</td>
<td>08NL038</td>
<td>5,590</td>
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<td>Pasayten River above Calcite Creek</td>
<td>08NL069</td>
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<td>1976-2005</td>
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<td>08NM116</td>
<td>811</td>
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<td>Kettle River near Laurier</td>
<td>08NN012</td>
<td>9,840</td>
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<td>Hedley Creek near the mouth</td>
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<td>389</td>
<td>1976-2005</td>
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<td><strong>Long Period Analysis</strong></td>
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<td></td>
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<tr>
<td>McGregor River</td>
<td>08KB003</td>
<td>4,780</td>
<td>1960-2005</td>
</tr>
<tr>
<td>North Thompson River @ Birch Island</td>
<td>08LB047</td>
<td>4,450</td>
<td>1960-2005</td>
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<td>North Thompson River @ McLure</td>
<td>08LB064</td>
<td>19,600</td>
<td>1960-2005</td>
</tr>
<tr>
<td>Salmon River @ Falkland</td>
<td>08LE020</td>
<td>1,040</td>
<td>1966-2005</td>
</tr>
<tr>
<td>Ashnola River near Keremeos</td>
<td>08NL004</td>
<td>1,050</td>
<td>1953-2005</td>
</tr>
<tr>
<td>Tulameen River @ Princeton</td>
<td>08NL024</td>
<td>1,760</td>
<td>1963-2005</td>
</tr>
<tr>
<td>Kettle River near Laurier</td>
<td>08NN012</td>
<td>9,840</td>
<td>1929-2005</td>
</tr>
<tr>
<td>Kettle River near Laurier (concatenated)</td>
<td>08NN012</td>
<td>9,840</td>
<td>1929-45,1976-05</td>
</tr>
</tbody>
</table>
Table 2. Environment Canada climate stations used in the 1976-2005 analysis.

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Station Identifier</th>
<th>Elevation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kamloops Airport</td>
<td>1163780</td>
<td>345</td>
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<tr>
<td>Penticton Airport</td>
<td>1126150</td>
<td>344</td>
</tr>
<tr>
<td>Williams Lake Airport</td>
<td>1098940</td>
<td>940</td>
</tr>
</tbody>
</table>

Five variables in annual ice cover were originally selected for detection of trends over the 30 year period from 1976-2005. These were date of first ice, date of last ice, length of time between first ice and last ice, longest continuous duration of ice cover, and number of ice-free occurrences in the ice cover season. The last was dropped since it was highly correlated to the fourth variable investigated and because the results were difficult to interpret due to the shortening ice cover duration over time and an inability to analyse the data properly.

The four selected variables were evaluated for trends with time first by simple linear regression of each against time and then by determining trend strength using Spearman’s nonparametric test for correlation. These two evaluations were done for the 22 stations in the study and again for 7 of those 22 stations that had longer suitable record. Figures 2 and 3 are examples of the linear regressions of all 4 ice cover variables against time for the 1976-2005 period for 1 of the 22 stations in the study.

3. Results

For the period 1976-2005, the trends for each of the 4 ice cover variables at each of the 22 sites were as follows:

- Date of first ice: 19 later, 3 earlier
- Date of last ice: 16 earlier, 5 later, 1 the same
- Time between first ice and last ice: 19 shorter, 1 longer, 2 the same
- Longest continuous ice duration: 22 shorter

To summarize the average changes that the 4 ice cover variables analyzed in this study have undergone in the period from 1976-2005:

- Date of first ice is between 1-2 weeks later
- Date of last ice is a week earlier
- Time between first ice and last ice is nearly a month shorter
- The longest continuous ice duration is more than a month shorter

For the longer periods ranging from 1929-2005 - 5 of the 7 longer periods began in the 1960’s - the trends for each of the 4 ice cover variables at each of the 7 sites were as follows:

- Date of first ice: 7 earlier
- Date of last ice: 5 earlier, 2 later
- Time between first ice and last ice: 5 shorter, 2 longer
- Longest continuous ice duration: 4 shorter, 3 longer

For the period 1976-2005, the average winter temperature (November through Mar) in the study area had risen more than 1.5 °C using the winter average temperature increases found at the three climate stations used for the study. (See Table 2.)

To determine trend strength in both the temperature records and the ice cover records for the period 1976-2005, a one-sided Spearman’s nonparametric test was used on each of the 4 ice cover variables at the 22 WSC stations, and the same test was used on the monthly average temperatures for each of the 5 winter months and the overall winter average temperature at the 3 climate stations.

The following summarizes how many of the trends were significant at the 5 % probability level for each of the 4 ice cover variables for the 1976-2005 period:
- Date of first ice: 4 of 19 later trends, 0 of 3 earlier trends
- Date of last ice: 2 of 16 earlier trends, 1 of 5 later trends
- Time between first ice and last ice: 10 of 19 shorter trends, 0 of 1 longer trend
- Longest continuous ice duration: 10 of 22 shorter trends

The following summarizes how many of the trends were significant at the 5 % probability level for each of the 4 ice cover variables at the 7 WSC stations that had longer usable record. (The data for the station with record going back to 1929 was tested for 2 time periods – one using the entire record from 1929-2005 and the other using the 1976-2005 period record concatenated with the previous warm phase Pacific Decadal Oscillation (PDO) period from 1929-1946 in a manner similar to an analysis in Hamlet et al. (2005) to try to minimize the effects of different phases of the PDO on the entire record making a total of 8 longer data sets.):
- Date of first ice: 0 of 1 later trend, 0 of 7 earlier trends
- Date of last ice: 5 of 6 earlier trends, 1 of 2 later trends
- Time between first and last ice: 1 of 6 shorter trends, 0 of 2 longer trends
- Longest continuous ice duration: 2 of 5 shorter trends, 0 of 3 longer trends

For the 3 climate stations, the warming trends for individual 5 winter months were significant at the 5 % probability level for the 1976-2005 period only at Kamloops in December and at Penticton in January and none of the entire winter average temperatures were significant. However, the entire winter average temperature trends were warming for both the Kamloops and Penticton stations at a less convincing 10 % level of significance and the month of December trend at Penticton was also significant at the 10 % level. Warming trends for February and March were quite weak at all 3 stations; these weak trends in February and March, and the stronger

**Figure 2.** Example of simple linear regression plots of trend lines for dates of first and last ice.

**Figure 3.** Example of simple linear regression plots of trend lines for ice cover duration and length of continuous ice cover.
4. Discussion

Clearly there has been a significant downward trend in the length of ice cover and the length of continuous ice cover in the 30 years from 1976 to 2005 on southern BC streams. The 22 WSC stations used in the study also show date of first ice occurred later and date of last ice occurred earlier over the same 30 years, but neither trend exhibited by these two variables was statistically significant for the region overall.

For the 7 WSC stations of the 22 with at least 10 more years of useable record going back to 1966 or before, the longer records showed the same regional trends for 3 of the 4 variables although the date of first ice trend was earlier in all but one case. A review of the data at each station indicates that this is partially due to a few years in the 1960’s that had very late dates of first ice that influenced the trend line enough to have it still sloping downward through 2005. Zhang et al. (2001) concluded that 30, 40, and 50 years of discharge record through 1996 showed that first ice was occurring earlier among other findings in BC but these conclusions were based on only a few station records in the southern interior of the province and did not include the most recent decade of data.

There could be a source of error in interpreting when ice has affected the stage data produced at a gauging station which is the foundation of each ice cover variable investigated. The notation of “ice affected” discharge on a station’s daily discharge record is subject to interpretation, particularly when the ice influence is minimal, but WSC personnel are trained to accurately read these charts so annual errors should be small. Grant McGillivray (personnel communication, 2007) stated that interpretation is better on bigger streams and uniform amongst WSC review staff.

The winter of 1976-77 was chosen as the beginning year of data analysis since that was the year that the PDO switched from cold phase to warm phase and the number of active gauging stations in 2006 dwindled to a very small number rapidly going backward in time to virtually none when the previous warm phase of the PDO ended in 1946. Wang et al. (2006) and Mantua et al. (1997), among many others, have shown that the PDO in cycles of 50-60 years has a large impact on climate and, consequently, on water resources, including ice cover formation. The PDO until now has been described as being in either a “warm” or a “cool” phase. However, scientists have not been able to decide if the warm phase that began in 1977 has ended. Nathan Mantua (personal communication, 2007) has suggested that the PDO may also have a “neutral” phase in which it may have been since the late 1990’s.

Another much shorter – in the order of a few years – cycle of climate variability is the El Nino/Southern Oscillation (ENSO) that can also have a major impact on climate in BC during the winter. Since ENSO has a relatively short cycle, the 30 years of record used in this study
spans many ENSO cycles eliminating most of the errors that could affect the results of a trend analysis that spanned a single phase switch of a major influence on BC climate such as the PDO. Recent research is uncovering changes associated with global warming that will likely further complicate analysis of short-term climate records even more (e.g., Salathe, 2006).

This basic study shows that the altering climate is now shifting river ice conditions in obvious ways in southern BC. There are more subtle ice cover changes that are likely already taking place and these changes will become more obvious as decades pass. These changes are bound to have large impacts on frequency, location, and severity of ice jamming in the entire BC interior, not just the southern interior. Beltaos (2008) reviews recent research and data compilation being conducted that will undoubtedly help improve our understanding of how the changing climate in Canada will shape future freeze-up, breakup, and ice jamming events in Canada and in BC. More in-depth research into ice cover parameters at WSC gauging sites in the BC interior, an expanded geographic study area, and correlating recorded ice cover parameters to recorded climate parameters are needed to advance the ability to quantify what future ice covers might look like given future climate scenarios. From there, better projections of problem freeze-up, breakup, and ice jamming on BC streams can be made using the best projections of future climate parameters.

Extending the anticipated impacts of climate change on river ice behaviour beyond the simple results of this study will not be easy. For example, determining the intertwined impacts of global warming on the future severity and frequency of mid-winter breakups in southern BC will involve, among other things, comparing the steady reduction in winter snowpack in western North America (Mote et al., 2005) in watersheds where snowpack depth and density play a large role in the size of the increase in stream discharge from a mid-winter thaw (Costerton and Doyle, 1995) to the reduced ice cover described herein.

5. Summary and Conclusions

For the period 1976-2005, river ice covers at 22 WSC gauging sites in the southern interior of BC have shown the following trends:

- Date of first ice is occurring later in the fall
- Date of last ice is occurring earlier in the spring
- Time between first ice and last ice has become significantly shorter
- The longest continuous ice duration has become significantly shorter

For longer periods of record exceeding the 30 years at 7 of the 22 WSC sites, 3 of the 4 trends described in the previous paragraph exist with only the date of first ice occurrence showing a different trend. The large variation in annual winter temperatures in the region results in a low level of significance in winter temperature trends at climate stations in the region with a corresponding low level of significance in many of the individual gauging site trends but the
overall regional trends are unmistakable. Additional years of record should strengthen the statistical relationships found in the ice cover trends through 2005.

A more detailed investigation of daily winter weather records at long-term climate stations, long-term snow courses and snow pillows, and WSC daily discharge records and gauging site field notes made during winter discharge measurements would yield greater insight into what future ice cover conditions and problems might be expected during the 21st century in the southern interior of BC.

References

www.env.gov.bc.ca/soe/ett07/04_climate_change/technical_paper/climate_change.pdf


Fish Protection, Wedgewire Intake Screens, and Frazil Ice

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Section 316(b) of the Clean Water Act requires the Environmental Protection Agency to ensure that the location, design, construction, and capacity of thermal power cooling water intake structures reflect the best technology available to protect fish. Cylindrical wedgewire screens comprise an efficient technology for decreasing impingement and entrainment losses at water intakes. Such screens draw water through a fine mesh at low flow velocities. Though mitigating EPA’s impingement and entrainment concern, wedge-wire screens run the risk of blockage by frazil ice. We review the characteristics of wedgewire intake screens, and discuss how these characteristics make wedgewire screens susceptible to frazil blockage. The same characteristics that make wedgewire screens successful protectors of fish make them efficient frazil collectors. The small slot openings facilitate clogging of the openings by frazil. The large surface area of the screen elements provides a large area for frazil adherence. Limited observations of operational wedge wire screens suggests that airburst cleaning systems may not be capable of clearing frazil adhering to the intake screens. New methods are needed for frazil-blockage control. The writers outline potential options to mitigate frazil blockage at cooling water intake structures using wedgewire screens.
1. Introduction
The purpose of this paper is to draw attention to the likely increase in frazil ice problems associated with water intake structures used for thermal power plants in the United States. The U.S. Environmental Protection Agency (EPA) has instituted regulations to reduce fish mortality at cooling water intakes. Wedgewire screens are an intake screen system intended to reduce fish mortality at intake structure. The design of wedgewire screens makes them susceptible to frazil blockage. In this paper we review EPA’s regulatory requirements for water intake structures, and describe the characteristics of cylindrical wedgewire screens. In addition, we review the characteristics of wedgewire screens from the perspective of how such screens are susceptible to frazil blockage. Methods that may reduce or mitigate frazil blockage are discussed.

2. Review of Clean Water Act Section 316(b) Regulations
The U.S. Clean Water Act (CWA) seeks to “restore and maintain the chemical, physical, and biological integrity of the nation’s waters” (EPA, 2004). CWA authorizes EPA to regulate the withdrawal of cooling water from U.S. water bodies. Specifically, Section 316(b) of CWA addresses adverse environmental impacts caused by cooling water intakes. It states:

   Any action established pursuant to section 301 or section 306 of this Act and applicable to a point source shall require that the location, design, construction, and capacity of cooling water intake structures reflect the best technology available for minimizing adverse environmental impact. (Environmental Protection Agency, 2004)

Section 316(b) does not define ‘best technology available.’ In implementing Section 316(b), EPA considered other sections of CWA and determined that this section requires minimizing adverse environmental impacts to the water from which cooling water is extracted. In practice, implementing Section 316(b) requires reducing the adverse effects of cooling water intake structures on fish. Section 316(b) of CWA has been implemented in three stages over a five-year period: Phase 1 (implemented in 2001) covers new generating facilities; Phase 2 (2004) covers large existing electrical generating plants; and Phase 3 (2006) covers certain existing facilities and new offshore oil and gas rigs. A complete discussion of Section 316(b) regulations and implementation can be found at http://www.epa.gov/waterscience/316b/basic.htm.

Thermal power plants are major consumers of water in the United States. This water is used largely for cooling purposes. Hutson et al. (2004) estimate that thermal power withdrawals accounted for 48% of the total water use in the U.S. in 2000. Two different types of cooling system are used in power plants: once-through and closed-loop systems. Once-through systems withdraw water from a source, circulate the water through heat exchangers, and then return the water to a surface water body. Closed-loop systems withdraw water from a source and repetitively cycle the water through heat exchangers and a cooling system. Subsequent water withdrawals for closed-loop systems are used to replace water lost through evaporation, leakage, and other losses. Once-through cooling systems account for 91% of water withdrawals for thermal power production.
EPA has determined that there are multiple ‘adverse environmental impacts’ associated with cooling water intake structures (EPA, 2004). Impacts include impingement; entrainment; reductions in endangered or threatened species; damage to important elements of the food chain; losses to commercial fisheries stocks; adverse effects on recreational fishing; and stresses to overall ecosystems and aquatic communities. Because of the difficulties of evaluating all of these impacts at all cooling water intake structures, EPA has determined that monitoring reductions in impingement and entrainment is a ‘quick, certain, and consistent metric’ (EPA, 2004) for determining performance of intake structures.

Impingement occurs when organisms are trapped against intake screens by the force of the water being drawn through the cooling water intake structure. Impinged organisms often die. Entrainment is when organisms are drawn through the cooling water intake structure into the cooling system. Entrained organisms are typically small, fragile, and weak swimmers, including the eggs and young of fish and shellfish. Entrained organisms passing through a cooling system are subject to thermal, mechanical, and chemical stresses which are often fatal. In once-through cooling systems, entrained organisms (living and dead) are discharged back into the water body. In general, meeting the Section 316(b) regulations require facilities to use technologies to reduce impingement by 80% to 95% and entrainment by 60% to 90% of all life stages of fish compared to a baseline estimate.

Examples of entrainment and impingement impacts of cooling water intake structures are given in EPA documents (EPA, 2004), including one for the Brayton Point Power Station in Somerset, MA. The EPA has determined that this plant’s cooling water intake system has contributed to the collapse of the fishery in Mount Hope Bay. The facility withdraws nearly 3.8 million m$^3$ day$^{-1}$ of water and entrainment and impingement fish losses are estimated in the trillions. The fish population in Mount Hope Bay is estimated to have declined by more than 87% since 1984 when modifications to one unit of the power plant changed the unit from a closed-loop system to a once-through system, which increased cooling water withdrawal by 45%.

There are a number of alternative technologies that can be used to reduce impingement and entrainment at cooling water intake structures, including traveling screens and fish handling return systems, cylindrical wedge wire screens, fine-mesh screens, fish barrier nets, louver systems, angular and modular inclined screens, velocity caps, porous dikes and leaky dams, and behavioral systems. In this paper we focus our discussion on potential frazil impacts on wedgewire screens for two reasons:

1. There are a number of different methods that facilities withdrawing cooling water can use to meet the performance standards for impingement and entrainment required for Section 316(b). One method of meeting the performance standards is through use of an approved design and construction technology. Using this method results in substantially streamlined permitting requirements, which should be attractive to plant operators. As of the promulgation of Phase II of Section 316(b), the EPA had approved only one technology: submerged cylindrical wedgewire screens (EPA, 2004).

2. Although Section 316(b) of the CWA applies only to cooling water intake structures, wedgewire screens are being adapted by a variety of users wishing to reduce fish impingement and entrainment mortalities. A web search shows that there are numerous
manufacturers designing and producing wedgewire screens. Large increases in the use of this technology at water intake structures are expected in the future.

3. Wedgewire Screens
A cylindrical wedgewire screen is a passive (no moving parts) screening system that admits water through an intake at low, uniform velocities. Figure 1 shows a typical Tee-shaped cylindrical wedgewire screen. Cylindrical wedgewire screens are 0.6 to 2.4 m in diameter up to 6 m long. The cylindrical outer portions of a screen are made of continuous triangular or wedge-shaped wires welded to a thin support frame (Figure 1B). Slots between the triangular wires admit water into the intake structure. To meet Section 316(b) requirements, slot widths vary between 0.5 and 10 mm. The required slot widths are determined on a site-by-site basis. In general, permitting requirements tend toward slot widths of less than 2 mm to reduce entrainment of eggs and larva. After intake water flows through the wedgewire elements, it is drawn into the center of the ‘T’, and then into the plant (Figure 1). For installations that require very large volumes of water, multiple cylindrical wedgewire screens are used.

Manufacturers list a number of advantages gained by incorporating cylindrical wedgewire screens into water intake systems, including
1. Low investment costs
2. Low maintenance
3. Ability to reduce impingement and entrainment mortality
4. Ease of cleaning

For most systems, this cleaning is accomplished by back flushing compressed air into the wedgewire screens structure. As the air bubbles through the screen slots it removes attached debris. This removal is enhanced by two features. First, the triangular cross section of the wedgewire elements means that debris is only in contact with the screen at two points. Second, only wedgewire screens installed in rivers meet approved design and construction criteria. The river current flowing past the screen face carries away any debris removed by the air backwash system.

Although wedgewire screens are a pre-approved technology, there are still five criteria that must be met in order to comply with the ‘Meeting Performance Standards through Use of an Approved Design and Construction Technology’ (EPA, 2004):
1. The cooling water intake structure must be located in a freshwater river or stream.
2. River current speeds must be >0.25 m s\(^{-1}\) past the screen face.
3. The through screen velocity must be <0.13 m s\(^{-1}\).
4. The slot size must be appropriate for the size of eggs, larvae, and juveniles of any fish and shell fish that need to be protected at the site.
5. Then entire cooling water flow must pass through the wedgewire intake screens.

An EPA technical fact sheet on wedgewire screens (http://www.epa.gov/waterscience/316b/phase1/technical/ch5fs.pdf) notes: ‘In northern latitudes, provisions for the prevention of frazil ice formation on the screen must be considered.’ And, slightly further down in the same fact sheet: ‘The physical size of the screening device is limiting in most passive systems, thus, requiring the clustering of a number
Figure 1. (A) Depiction of a wedgewire screen installation. (B) A cartoon cross section of the wedgewire screen elements attached to a reinforcing bar. From Amaral (2003).

of screening units. Siltation, biofouling and frazil ice also limit areas where passive screens such as wedgewire can be utilized.’

This fact sheet does not advise how to reduce or mitigate frazil blockages of wedge wire screens. However, as noted, siltation and biofouling also reduce flows, and manufacturers have come up with methods for clearing debris from the wedgewire screen surface. An EPA website with information pertaining to Phase III implementation of the Section 316(b) rule (EPA, 2006) states:

Due to the potential for build-up and plugging by debris, passive screens are usually installed with an airburst backwash system. This system includes a compressor, an accumulator (also known as a receiver), controls, a distributor, and air piping that directs a burst of air into each screen. The airburst produces a rapid backflow through the screen; this air induced turbulence dislodges accumulated debris, which then drifts away from the screen unit. Vendors claimed (although with minimal data) that only very stagnant water with a high debris load or very shallow water (<2 feet (ft) deep) would prevent use of this screen technology. Areas with low water velocities would simply require more frequent airburst backwashes, and few facilities are constrained by water depths as shallow as 2 feet.

Note that the EPA added the parenthetical ‘although with minimal data’, and that this section does not specifically address frazil blockage. We are not aware of any published,
refereed literature demonstrating that the airburst system effectively removes attached frazil from wedgewire screens.

4. Frazil Formation and Blockage of Intake Screens
Frazil are millimeter-sized discs or spicules of ice that form in turbulent, supercooled water (i.e. water cooled to below its freezing point). The amount of supercooling observed in rivers is very small, usually on the order of a few hundredths of a degree centigrade. This small level of supercooling is very difficult to detect without laboratory-grade equipment (Daly, 1991). Supercooling and frazil formation require a large net heat flux from the water to the atmosphere. In practical terms, this means frazil only forms in water not covered by a surface layer of ice.

Frazil is not a rare in rivers, and so poses a problem for water intakes. Formation of small volumes of frazil in a river is little cause for concern; frazil crystals simply float to the water surface and are incorporated into the growing surface ice layer. However, when conditions are right, large volumes of frazil can form overnight, and potentially clog water intakes.

The amount of frazil that forms in a river depends on a number of variables that fall into two broad categories: weather conditions and river conditions. Generally, conditions must be pretty severe to cause formation of enough frazil to be a hazard for cooling water intakes. The conditions favoring catastrophic frazil formation are well known (e.g., Tsang 1982). They entail zero solar radiation heat input, and large heat losses by long wave radiation, evaporation, and convection from a small water body. Frazil and anchor ice are likely to form at night when the wind is strong, the humidity of the air is low and the river is at minimum flow, especially if such a night follows a cold, windy and cloudy day. Daly (1991) reports that frazil formation is associated with air temperatures of -6 °C or lower, open water, and clear nights. He states emphatically that frazil cannot form (and, by extension, intake screens will not freeze) where a continuous, stable ice cover is present. In a study of intake screen blockages by frazil in the Great Lakes region, Foulds and Wigle (1977) report that frazil-producing conditions include clear night skies, temperatures of -7 °C or less, daytime water temperatures of less than 0.2 °C, and winds of 16 kph (10 mph) or greater.

Water supercools at its surface. Supercooled water is then mixed downward into the river by flow turbulence. Frazil crystals are mixed into the water column along with the supercooled water. Supercooled water will cool the riverbed and anything in the water column (like intake screens) to below the freezing point. Once the bottom or an object in the flow is below the freezing point, frazil will adhere to it (Daly and Ettema, 2006). This is often described as the frazil being ‘sticky’ or ‘active’ (Carstens, 1966; Michel, 1971; Foulds and Wigle, 1977; Tsang, 1982). Accumulations of frazil crystals can grow to be quite large, and stick tenaciously to the bottom (as anchor ice masses) for as long as the water remains supercooled. Usually, incoming solar radiation during daylight hours warms the water to the freezing point, releasing anchor ice from the bottom or structures (Wigle, 1970; Arden and Wigle, 1972). Although frazil usually forms at night, when conditions are particularly severe frazil can form in the water column at any time of the day, and anchor ice accumulations can last for more than 24 hours on the bottom or on intake screens (Daly and Ettema, 2006).
Records of frazil blocking industrial and municipal water supply intakes in northern countries go back almost as long as intakes have been placed in rivers and lakes. Altberg (1936) lists an early example where the entire water supply for the city of St Petersburg, Russia was choked off for three days by frazil blockage. Frazil blockage of water intakes is still a serious problem in cold climates. Daly and Ettema (2006) list nine frazil blockages of Lake Michigan water intakes during the winter of 2002-2003. These intakes ranged from 5 to 15 m water depth. For two of these sites Daly and Ettema (2006) report ice problems ~11 times in last nine years, and ice problems 5-6 times in the past couple of years.

Given the prevalence of frazil blockages, one would expect that the conditions leading to frazil blockage of intake screens would be well documented. This is not true. There is general information available for operating water intakes in water bodies subject to frazil formation (Daly, 1991; Daly and Ettema, 2006), information on frazil formation and characteristics, and some information on designing water intakes for cold regions (e.g. Chen et al., 2006). Daly and Ettema (2006) point out that there is very little information on the specific problems faced by intake operators fighting frazil blockages. They list several reasons for this, including lack of an agency or group to keep track of blockage events and the fact that blockages tend to be dealt with at the plant level, with little effort to address the problem among or across the power and water supply industries.

Even though there is little site-specific frazil blockage information in the literature, conditions leading to potentially hazardous frazil formation are well understood, and there is an understanding of how frazil clogs water intakes. Daly (1991) outlines the steps that lead to frazil blockages of intake screens:

1. Blockage of intake screens begins when the intake ingests supercooled water. Supercooled water promotes buildup on the screens through several processes. First, every component of the screen in contact with supercooled water is cooled below the freezing point. Once the temperature of screen elements is below freezing, however slightly, frazil can adhere to that surface. The adhesion force of the frazil is a function of the screen material and the level of supercooling. The level of adhesion, even at small levels of supercooling, can be substantial (although unmeasured to date). Any frazil crystals suspended in the intake flow are available to adhere to the intake screen. Once an individual frazil crystal adheres to the screen, it continues to grow as latent heat is convected away by the supercooled water. The blockage also continues to grow by continued frazil accretion.

2. The frazil accumulation grows upstream into the flow, increasing in width until the space between adjacent screen elements is ‘bridged’. At this point the intake screen is effectively blocked, with all of the frazil attached to the upstream side (outside) of the intake screen. The frazil accumulation is porous, and water will continue to flow through the screen, but there is an increase in head loss across the blocked intake screen. This head differential is necessary to force the water through the frazil blockage.

3. Frazil will continue to accumulate outside the intake screen. As frazil continues to accumulate on the intake screens, a greater and greater head differential is necessary to drive the same flow rate through the growing frazil ice mass. Daly (1991) notes that normally a considerable amount of frazil can accumulate on the intake screens before the head loss across the screens becomes noticeable.
4. As frazil continues to accumulate, the head differential across the intake screen pushes the attached frazil mass through the intake screen wires. This creates a greater surface area between the attached frazil mass and the intake screen. At this point the frazil mass can withstand pressure differentials ‘equal to many feet of water’ (Daly, 1991) and it may be difficult to impossible to remove the frazil from the intake screens.

Head loss associated with flow through an intake screen does not increase linearly as frazil accumulates on a screen. Laboratory experiments show that head loss is relatively small for a substantial period of time as frazil accumulates. Then head loss increases quite rapidly near the time of total blockage (Daly, 1991). This explains the many published observations of intakes being clogged quickly, in periods of minutes to an hour from the first observation of problems. Frazil clogging of intake screens will be advanced before there is a substantial head differential across the screens; once drawdown is observed it can increase rapidly if aggressive efforts are not made to remove the attached frazil ice (Daly, 1991).

Frazil blockage of intake screens or trash racks can be a serious problem regardless of the type of screen used. A study of trash rack blockage on three Swedish rivers (Andersson and Andersson, 1992) considered conditions leading to frazil formation and accumulation of frazil on trash racks (not wedgewire screens). This study found that frazil blockage occurred at temperatures as high as -3.9°C. This frazil blockage, which remained transparent as it grew on the intake screen, grew to 10 to 15 cm thick before it completely blocked the intake of one power plant. There was a 2.5 hour time lag between the first observance of head loss and complete clogging of the intake structure. At an intake on another Swedish river, Andersson and Andersson (1992) report that a 14 m head loss occurred across a power plant intake rack. Head losses of this magnitude reduce power production and put plant equipment at risk.

Frazil may be particularly difficult to remove from wedgewire screens for two reasons. First, as slot size decreases, the open area of the wedgewire screen decreases. The outer surface of a wedgewire screen with 1 mm triangular bars and 2 mm slot width is 33% solid surface. This provides a large area for frazil adhesion. Second, after the frazil blockage is forced through the intake screen by the head differential across the screen, it has an even greater surface to adhere to. In addition, the inverted triangular shape of the screen elements (Figure 1B) may cause the frazil to form a ‘dovetail’ that increases the difficulty of removing the attached ice mass.

5. Minimizing Frazil Blockage of Intake Screens
A variety of methods have been tried to reduce frazil blockage of intake screens or trash racks. They include applying heat, using ice-resistant coatings on the trash racks, vibration, blasting, mechanical removal, back flushing, and removal of the trash racks or screens. Daly (1991) discusses the pros and cons of these various methods. In addition, one way to reduce, but not eliminate, frazil blockage, is to design the intake screens with a maximum space between bars and the thinnest possible bar members (Daly, 1991). Foulds and Wigle (1977) recommend a spacing of 60 cm between trash rack elements to reduce the potential for frazil blockage.

As discussed above, however, wedgewire screens are designed with 0.5 to 10 mm openings specifically to reduce fish impingement and entrainment. There is no available published information on frazil blockage of wedgewire screens, or of how to reduce the potential for frazil
blockage of these structures. In a laboratory study of trash rack blockage by frazil, Andersson and Daly (1992) studied the effect of bar shape and spacing on frazil accumulation on intake trash racks. They included round, rectangular, pointed, and square screen elements in this study, and concluded that there is no clear advantage of one shape over another in reducing frazil blockage. Note that the ‘pointed’ bars they used were oriented in the opposite direction of wedgewire screens, that is, the pointed edge of the screen elements pointed into the incoming flow rather than out of the flow. In a series of experiments on the effects of bar spacing versus head loss (a measure of frazil blockage), Andersson and Daly found that decreased space between bars decreased the time required to reach a specific head loss. These experiments were conducted with rectangular bars with spacing of 25.4, 47.6, and 60.3 mm, considerably larger than the slot width on wedgewire screens. The slot width on wedgewire screens, especially those designed to reduce entrainment, are about the same size as a frazil crystal (i.e. ~1 mm). This suggests that wedgewire screens may be especially susceptible to frazil blockage, because the small slot width can be bridged by very few frazil crystals. Also, the limited data of Andersson and Daly (1992) suggests that the very narrow slots found on wedgewire screens may foul in a matter of minutes when sticky frazil is present.

A simple calculation illustrates how quickly frazil can accumulate on a wedgewire screen. Typical frazil concentrations in rivers are on the order of $10^6$ particles m$^{-3}$ (Daly, 1991). Assume these frazil crystals are discoid 2 mm in diameter and 0.1 mm thick (Chen et al. 2004), and the wedgewire intake screen has 2 mm slot width and 1 mm bar thickness. A maximum intake flow of 0.135 m s$^{-1}$ through the 0.67 m$^2$ open portion of the intake screen entrains 320 m$^3$ hour$^{-1}$ of water through each square meter of intake screen surface. This water contains 0.1 m$^3$ of frazil. If the collection efficiency of the screen is 0.5 (50%) and the porosity of the frazil blockage is 50% (Andersson and Daly, 1992), the frazil blockage will be 10 cm thick in one hour. This calculation assumes all frazil blockage growth is through frazil adhesion to the intake screen. As noted by Andersson and Andersson (1992), a 10-cm-thick frazil blockage is enough to completely block an intake. The writers note that ice accumulation on the screens can increase by two methods: frazil accumulation and in situ thermal growth of already-attached frazil.

Of the methods reviewed by Daly (1991) to reduce frazil blockage of trash racks, two have been adapted for wedgewire-screen installations: heating and back flushing. Johnson and Ettema (1988) report on a wedgewire screen heating system for a closed-loop power plant withdrawing water from the Kansas River. This system uses a manifold to inject warm groundwater at the upstream edge of the cylindrical wedgewire screen during periods of potential frazil formation (Figure 2). At the time this paper was published, the intake had operated for four years with no significant problems. Additionally, EPA documents (2004) note that, as of 2004, this intake system had still not experienced any operating difficulties. Daly (1991) notes that power plants using cooling water often discharge heated water downstream of the cooling water intake structure. Part of this warmed water (enough to warm the intake water 0.1 to 0.2 °C) could be injected into the intake water ahead of the wedgewire intake to inhibit frazil formation.

The airburst backwash system is such an integral part of wedgewire screen systems that phase III of the 316(b) regulations recommends it for all offshore cooling water intake structures incorporating cylindrical wedgewire screens. It appears that this airburst system is designed primarily to remove mud and debris from the wedgewire cylinders. Based on the EPA (2006)
Figure 2. Top: Longitudinal sketch of wedgewire screen intake system on the Kansas River. The de-icing pipes at the upstream (right) end of the concrete trough inject 0.2 m$^3$ s$^{-1}$ of 12$^\circ$C groundwater during periods of frazil formation. Bottom: cross section of the intake trough designed to keep bed sediment from inundating the wedgewire screens and to protect the screens from floating debris. The 12 m long trough is periodically cleaned by sluicing water into the bottom of the trough. This lifts trapped sediment into the water column where it is carried away by river currents. An airburst backwash system is used to clear debris from the wedgewire screens. Modified from Johnson and Ettema (1998).

Observation that there is minimal data supporting vendor claims that this system successfully removes debris, we suggest caution in relying solely on the airburst system to prevent or remove frazil blockages from cylindrical wedgewire screens. Analysis of weather conditions at one cooling water intake structure equipped with cylindrical wedgewire screens suggests that it was unexpectedly clogged by frazil during a period of cold weather in January 2004 (Kempema, 2007). This system was equipped with an air backwash system, but it is not clear whether the
backwash system was activated once a head differential developed across the intake screens. Given the manner whereby frazil may accumulate on and freeze to a wedge-wire screen, it is unlikely that air backwash would be effective in removing the frazil accumulation.

One way to reduce frazil blockage at some cooling water intake structures is to remove the cylindrical wedgewire screens during potential frazil formation periods. This would be possible in situations where the wedgewire screens are mounted to a bulkhead and equipped with guide rails for removing the screens for inspection (see Johnson and Ettema, 1988, for an example). However, this would remove all screens from the intake system, and as pointed out Daly (1991), intake screens should only be removed after carefully weighing the problems caused by frazil accumulation against the damage that may be caused by not having the screens in place. It should also be noted that removing the screens would violate the EPA (2004) requirement that all of the intake water must pass through the wedgewire screens. However, eggs, larvae, and fish densities are likely to be at very low relative concentrations in the water column during periods of frazil formation. Consequently, entrainment would not be a problem.

The best way to reduce the problems associated with frazil blockage of intake screens is to be both aware and prepared. When there is no surface ice cover at the intake site and water temperatures are close to freezing, forecasts of low air temperatures and clear nights should alert plant operators to the possibility of frazil formation and fouling of intake screens. When these conditions are present, the head differential across the intake screens should be continuously monitored. Continuous monitoring will allow operators to quickly determine the degree of intake screen blockage, and to take appropriate measures if blockage is observed. The limited available evidence suggests that the very narrow slot widths on wedgewire screens may clog very rapidly once frazil starts accumulating on the intake screens. Due to the lack of knowledge about frazil blockage of wedgewire screens, the response of individual cooling water intake structures to frazil presence will have to be determined on a case-by-case basis.

6. Discussion and Conclusions
It is relatively difficult to find specific information about frazil blockage of water intakes fitted with cylindrical wedge-wire screens. As a result, to determine the potential for wedgewire screen blockage, we needed to extrapolate the limited published data on frazil blockage of trash racks to wedgewire screens. This data may not be strictly applicable, because the slot spacing of wedgewire screens is of the same order as the size of frazil crystals. This suggests that wedgewire-screened intakes may clog very quickly to frazil events, and may clog at much lower frazil concentrations than intakes equipped with much coarser intake screens. Older cooling water intake structures that are retrofitted with wedgewire screens to reduce impingement and entrainment mortalities may face new frazil blockage problems.

Cylindrical wedgewire screens are a preferred technology for meeting the Clean Water Act Section 316(b) requirements for cooling water intake structures. They reduce impingement and entrainment for many types of intake structures, and they are being widely adapted in the U.S. However, the very design elements that make wedgewire screens attractive for reducing impingement and entrainment of organisms also make them efficient frazil collectors and aggravate frazil removal. Available data suggest that intakes fitted with cylindrical wedgewire screens will be more susceptible to frazil blockage than intakes equipped with more traditional
trashrack systems. As traditional trashracks are replaced with cylindrical wedgewire screens, intake operators will have to be prepared for a potential increase in frazil ice problems. The narrow slot spacing on wedgewire screens increases the probability of frazil blockage, and plants that may have been trouble free in the past may experience new frazil blockage problems with the new wedgewire screens. It may take a number of seasons for these problems to occur. We urge special attention be given to head differentials across the intake screens during periods of potential frazil formation. Also, for some intakes, it may be feasible to remove the wedgewire screens during frigid-water conditions, as biota levels are very low in river and lake water subject to these conditions.

References


Classification of arctic river mouths and regularities of currents in ice-covered estuaries

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The main hydrological and morphological features of the Arctic river mouth areas are discussed. Four types of the Arctic river mouth areas (simple, estuarine, estuarine-deltaic, and deltaic) are distinguished. It is shown that the type of river mouth area depends on the character of the nearshore zone (semi-enclosed or open) and existence of delta. It is established that the water, thermic and ice regimes of the arctic river mouths are determined by regional climate, morphological and hydrological type of the river mouth area, regime of the river and coastal zone of the ocean. Mouth areas of large rivers of Russian Arctic (the Onega, Severnaya Dvina, Kuloy, Mezen, Pechora, Ob, Nadym, Pur, Taz, Yenisei, Khatanga, Anabar, Olenyek, Lena, Yana, Indigirka, and Kolyma rivers) are studied. Characteristic features of formation of structure of ice-covered flows under different tide conditions are considered. Influence of tides and type of a river mouth area on the processes of mixing and salt water intrusion at the river mouth are discussed.
1. Introduction
Water, thermic and ice regimes of the Arctic river mouths depend on the region climate, morphological and hydrological type of the river mouth area, regime of the river and nearshore zone. Climate conditions of the arctic region are characterized by extremely irregular input of solar radiation over the year and intensive cyclonic action. Many mouth areas of large rivers of Eurasia are situated within the limits of continental arctic region. The Onega, Severnaya Dvina, Kuloy, Mezen rivers belongs to the basin of the White Sea, the Pechora River – to the Barents Sea. The Ob, Nadym, Pur, Taz, and Yenisei rivers empty into the Kara Sea. The Khatanga, Anabar, Olenyek, Lena, Yana rivers inflow to the Laptev Sea, and the Indigirka and Kolyma flow into the East-Siberian Sea.

In our paper, we consider the peculiarities of hydrological and hydrodynamic regime of the Arctic rivers’ mouths. Close attention is given to the characteristics of mixing processes, formation of ice-covered flows under different tide conditions. This article is based on the analysis of studies of these river mouth, carried out during the last decades in Russia. The experience gained in studying river mouth area in Russia, served as a theoretical basis of this work.

2. Definition of river mouth and its typification
The river mouth area is a peculiar geographical object covering the region of river inflow to a receiving basin (ocean, sea) and having a fluvial-marine hydrological regime. The river mouth area is formed under the influence of specific mouth processes, principal of which are dynamic interaction and mixing of river water with water of the receiving basin, deposition and redeposition of fluvial and partially marine sediments resulting in the formation of a delta.

Two types of water, i.e. river and sea water, quite different in their physical, chemical, and biological properties interact in the river mouth area. The river hydrological regime dominates in the mouth reach of the river; however, it is under an intense impact of the receiving basin (mean sea level long-term changes, tides, storm surges, sea water intrusion). The hydrological regime typical of the receiving basin dominates in the nearshore zone of the river mouth; however, it is under an intense impact of the river (river flow currents, propagation of river plume with fresh and turbid waters into the sea).

According to the morphological characteristics, all the mouth reaches of the river can be subdivided into mouth reaches without deltas (single-branch) and deltaic mouth reaches. The open nearshore zone of the mouth area can be subdivided into wide or narrow, deep or shallow types. Sometimes semi-enclosed coastal water bodies are situated between a mouth reach of the river and open nearshore zone. These intermediate parts of the river mouth areas can be presented as narrow sea bays, lagoons, limans, and estuaries. These semi-enclosed coastal water bodies are characterized by active interaction and mixing of river and sea waters.

Two morphological types of the deltas can be distinguished: filling (or bayhead) deltas, which are formed in the semi-enclosed coastal water bodies (limans, lagoons, estuaries) and protruding deltas, which are developed in the open nearshore zone. Therefore all the river mouth areas are subdivided into the following types (Figure 1) regarding their structure [Mikhailov (1998); Mikhailov & Mikhailova (2008)]: (I) Simple – with an open nearshore zone and without deltas;
(II) Estuarine – with semienclosed coastal water bodies and without deltas; (III) Estuarine-deltaic – with semienclosed coastal water bodies and with filling (or bayhead) deltas; (IV) Deltaic – with open nearshore zone and protruding deltas.

Figure 1. Scheme of river mouth areas of different types and their subdivisions into the parts [Mikhailov (1998); Mikhailov & Mikhailova (2008)].

According to the hydrological classification [Mikhailov (1998); Mikhailov & Mikhailova (2008)], the following indices can be used for the mouth reach of the river: water regime and river flow recharge patterns, mean water turbidity, and thermal and ice regime patterns. As for a semi-enclosed part of the river mouth and open nearshore zone, the following hydrological characteristics can be taken into consideration: the pattern of mean sea level changes, the rate of tides, and storm surges, the dominating current, the mode of waves, water salinity, and the peculiarities of thermic and ice regime.

3. Types of arctic river mouth and their water and ice-thermic regime

In accordance with above-mentioned classification, arctic river mouth areas can be subdivided into four types: simple (S), estuarine (E), estuarine-deltaic (E-D), and deltaic (D) (Table 1). The mouth area of the Onega River belongs to simple type. The mouth areas of the Kuloy, Mezen’, Khatanga, Anabar rivers pertain to estuarine type, and the Severnaya Dvina, Olenyek, Lena, Yana, Indigirka, Kolyma rivers – to deltaic ones. Emptying into the ocean, these rivers form protruding deltas. The mouth areas of the Pechora, Ob, Nadym, Pur, Taz, and Yenisei rivers belong to estuarine-deltaic type. Morphometrical characteristics of arctic mouth areas of different type are presented in Table 1.

The hydrological type of the mouth area depends on river water regime and tide action. A majority of arctic rivers is referred to the river with predominate snow melt feeding, spring high-water period, summer-autumn rain floods, winter low-flow period. Mean annual values of river
and sediment runoff are presented in Table 2. The distribution of river runoff within a year is very uneven. The greater portion of water runoff (~70–80%) falls on three months: May–July (West Russian Arctic) and June–August (East Russian Arctic). The lowest water runoff is typical of winter months (November–April). The distribution of suspended sediment runoff within a year is more uneven than water runoff. For example, during flood months (June–August), the total water and sediment runoff at the Indigirka River mouth makes up ~81 and 91% of the annual values, correspondingly.

Table 1. Morphometrical characteristics of arctic mouth areas of different type [Mikhailov (1997)]. Here and in Table 2 dash means lack of information.

<table>
<thead>
<tr>
<th>River</th>
<th>Catchment area, thousand km²</th>
<th>Length, km</th>
<th>Type of mouth area</th>
<th>Delta area, km²</th>
<th>length, km</th>
<th>Estuary area, km²</th>
<th>length, km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Onega</td>
<td>56.9</td>
<td>416</td>
<td>S</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sev. Dvina</td>
<td>357</td>
<td>744</td>
<td>D</td>
<td>900</td>
<td>45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Mezen</td>
<td>78.0</td>
<td>966</td>
<td>E</td>
<td>0</td>
<td>0</td>
<td>162</td>
<td>40</td>
</tr>
<tr>
<td>Kuloy</td>
<td>19.0</td>
<td>350</td>
<td>E</td>
<td>0</td>
<td>0</td>
<td>80</td>
<td>30</td>
</tr>
<tr>
<td>Pechora</td>
<td>322</td>
<td>1810</td>
<td>ED</td>
<td>3250</td>
<td>120</td>
<td>6500</td>
<td>80</td>
</tr>
<tr>
<td>Ob</td>
<td>2990</td>
<td>3650</td>
<td>ED</td>
<td>3250</td>
<td>144</td>
<td>40800</td>
<td>760</td>
</tr>
<tr>
<td>Nadym</td>
<td>64.0</td>
<td>545</td>
<td>ED</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Pur</td>
<td>112</td>
<td>389</td>
<td>ED</td>
<td>627</td>
<td>76</td>
<td>7760</td>
<td>300</td>
</tr>
<tr>
<td>Taz</td>
<td>150</td>
<td>1400</td>
<td>ED</td>
<td>831</td>
<td>82</td>
<td>–</td>
<td>24</td>
</tr>
<tr>
<td>Yenisei</td>
<td>2580</td>
<td>3490</td>
<td>ED</td>
<td>4500</td>
<td>196</td>
<td>20000</td>
<td>350</td>
</tr>
<tr>
<td>Khatanga</td>
<td>364</td>
<td>1636</td>
<td>E</td>
<td>0</td>
<td>0</td>
<td>5590</td>
<td>220</td>
</tr>
<tr>
<td>Anabar</td>
<td>100</td>
<td>939</td>
<td>E</td>
<td>0</td>
<td>0</td>
<td>–</td>
<td>24</td>
</tr>
<tr>
<td>Olenyek</td>
<td>219</td>
<td>2270</td>
<td>D</td>
<td>475</td>
<td>–</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Lena</td>
<td>2490</td>
<td>4400</td>
<td>D</td>
<td>32000</td>
<td>175</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Yana</td>
<td>238</td>
<td>872</td>
<td>D</td>
<td>3500</td>
<td>140</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Indigirka</td>
<td>360</td>
<td>1726</td>
<td>D</td>
<td>7600</td>
<td>130</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Kolyma</td>
<td>647</td>
<td>2130</td>
<td>D</td>
<td>3250</td>
<td>120</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Water level rise at delta heads during flood periods usually equals 8–10 m (Pechora, Yenisei, Lena, Yana), or 5–16 m (Ob, Indigirka, Kolyma). Seasonal water level variations rapidly decrease from the delta head to the delta coastline. Severe inundation is typical for all deltas.

The tidal range gradually decreases from west to east along the arctic coast of Russia. At the mouths of the Mezen River, the tidal range equals 4.8–7.6 m, and at the mouth of the Indigirka River, it is 0.3 m. The range of the tide in the mouth areas of the Pur, Taz, Yana, Kolyma rivers does not exceed 0.3 m (Table 2).

The principal characteristic feature of the ice-thermic regime of rivers inflowing into the arctic seas is that they flow from the south to the north. At first, ice arises on the downstream reaches to which water from the upper reaches comes forming strong icing fields. Many of Siberian rivers flow in the permafrost region, while the upper courses of these rivers in the southern regions have many tributaries yielding a great amount of water and heat to the main river. The
A distinguishing feature of freeze-up of Siberian rivers is the beginning of ice formation at the supercooled anchor ice then lifting up to the water surface. Ice regime of mouth reaches of arctic rivers is remarkable for annual prolonged and steady freeze-up. Climatic conditions in the Arctic deltas of Russia are very severe. Therefore, periods with water temperature above 1°C are very short: June–October in the west part of Russian Arctic and July–September in its east part. On the contrary, the duration of periods with ice cover is very long: from 200 to 240–250 days (from west to east). Freezing-up on the rivers begins, as a rule, in October and spring ice breaking – in May–June. The thickness of ice layer increases from 1 to 2 m from west to east, too. Several rivers in their shallow lower parts, as the Anabar River, freeze through the depth.

Table 2. Main hydrological characteristics of arctic mouth areas [Mikhailov (1997)].

<table>
<thead>
<tr>
<th>River</th>
<th>Runoff</th>
<th>Distance of maximum penetration into the river, km</th>
<th>Characteristics of ice phenomena</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>water, km³/year</td>
<td>sediment, ml. tn/year</td>
<td>tide</td>
</tr>
<tr>
<td>Onega</td>
<td>15.4</td>
<td>0.18</td>
<td>26</td>
</tr>
<tr>
<td>Sev. Dvina</td>
<td>108</td>
<td>4.4</td>
<td>135</td>
</tr>
<tr>
<td>Mezen</td>
<td>24.4</td>
<td>0.8</td>
<td>50</td>
</tr>
<tr>
<td>Kuloy</td>
<td>5.7</td>
<td>–</td>
<td>100</td>
</tr>
<tr>
<td>Pechora</td>
<td>130</td>
<td>8.5</td>
<td>190</td>
</tr>
<tr>
<td>Ob</td>
<td>402</td>
<td>13.0</td>
<td>51</td>
</tr>
<tr>
<td>Nadym</td>
<td>18.0</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Pur</td>
<td>32.3</td>
<td>0.62</td>
<td>0</td>
</tr>
<tr>
<td>Taz</td>
<td>43.4</td>
<td>0.91</td>
<td>0</td>
</tr>
<tr>
<td>Yenisei</td>
<td>597</td>
<td>4.9</td>
<td>445</td>
</tr>
<tr>
<td>Khatanga</td>
<td>105</td>
<td>5.2</td>
<td>227</td>
</tr>
<tr>
<td>Anabar</td>
<td>25.2</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Olenyek</td>
<td>36.1</td>
<td>1.23</td>
<td>–</td>
</tr>
<tr>
<td>Lena</td>
<td>528</td>
<td>20.0</td>
<td>&lt;100</td>
</tr>
<tr>
<td>Yana</td>
<td>31.9</td>
<td>4.2</td>
<td>30</td>
</tr>
<tr>
<td>Indigirka</td>
<td>54.0</td>
<td>11.9</td>
<td>24</td>
</tr>
</tbody>
</table>

Hanging dams are usually formed at the northern rivers due to frazil arising at the head river meets on the path down sufficiently firm ice cover [Il’ina, Grakhov (1987)]. The lower reach of the Severnaya Dvina River freezes up at 8–10 days earlier than that the upper reach. On the Pechora River, the freezing-up at the upper and lower regions begins at the same time but in the middle reach it begins later. Most often the hanging dams appear down stream the rapids or groundwater outlet since in these areas water opening occurs during long time in autumn. At the rapids of the lower reach of the Onega River hanging dam springing up every year. Every year hanging dam appears in the middle reach of the Onega River, for example, the backwater from it.
registered in 1969 (3.XII) produced the rise of level up to 4 m. Almost every year hanging dams appear in the middle reaches of the Severnaya Dvina and Pechora rivers, where, for example, it caused the rise of water level of ~3.8 m (20.XI 1969) and 4.8 m (8.XI 1943), correspondingly. After the water openings become frozen the formation of frazil stopped and ice jam begins to melt.

In winter, a sharp decrease of river runoff promotes sea water intrusion into dredged channels through mouth bars and into the delta branches. During winter period with low water flow, the length of salt wedges in the Indigirka delta can reach 60 km (Table 2).

4. Ice formation and interaction of fresh and sea waters at river mouths of different types

Ice formation on the water surface of the river mouth area depends on the air temperature, the range of tide and water salinity. Processes of fresh and salt waters mixing in ice-covered flow occur under the condition of isolation from the impact of wind that arise a severe decrease turbulent diffusion that promotes formation of multi layer density stratified stream.

Let us consider firstly formation of multi-layer density stratified stream in a nontidal estuary. Estuary stream consisting of layers of different water temperature \( t \) and salinity \( S \) in the zone of interaction of river and sea waters is defined by ratio of temperatures of maximum density \( t_d \) and freezing \( t_f \). For \( S<24.7\% \) temperature \( t_d > t_f \). As the air is cooling in autumn the temperature of water on the surface of a river decreases till \( t_d \), and water sinks to the sea water wedge of greater salinity creating intermediate cold layer of brackish water, which is kept after ice formation. The most of the rivers belonging to the basin of White Sea flowing into long, narrow bays of estuarine type [Mikhailov (1997)]. As an example of nontidal estuary let us consider the mechanism of formation of stream structure at the mouth of the Umba River (the Kola Peninsula) flowing into the White Sea. The mean annual water discharge \( Q \) is 78.2 m\(^3\)/s. The river freezes during the period from the end of October till the middle December, debacle and ice drift occur in May–June. The river width varies in the range of 30–60 m, the depths at rapids and pools are ~0.5 and 1.5 m, correspondingly. The velocity of the stream in summer reaches 1.7–2.0 m/s. The profiles of temperature and salinity in the zone of interaction of river and sea waters reveal formation of three-layer stream described above: the lower layer near the bottom with \( S \) of sea water 28\% and temperature \( t \approx +0.5^\circ C \), the layer above it of brackish cold water (\( t = -0.5^\circ C \)), and the thin upper layer of river water with \( t \geq 0^\circ C \) [Skriptunov (1976)]. The separation of the stream into layers of stable stratification suppress turbulent diffusion and mixing. Only in the upper layer there is a unilateral stream of river water moving to the sea. Turbulent mixing of the lower and intermediate layers can be neglected. The estimate of depth averaged vertical coefficient of turbulent momentum transfer in ice-covered flow as \( K_y \approx 1\cdot10^{-3} \text{m}^2/\text{s estuary} \) [Dolgopolova (2008)], which value can be used for assessment of diffusion in a nontidal.

At tidal river mouth, the location of the zone of interaction of river and sea waters changes periodically and \( S \) in this zone droningly increases towards the sea [Mikhailov (2007)]. To describe the estuary processes we use generally accepted classification for division of all estuaries into three types by the kind of stratification and water mixing: 1) complete mixing, weak stratification; 2) partially mixed estuary, moderate stratification; 3) salt wedge; strong stratification. In studies of mixing of river and sea waters the following terminology of water salinity is used: \( S<1\% \) – fresh water, 1–24.7\% – brackish water and \( S>24.7\% \) – salt water. The
main hydrological characteristics of the rivers flowing into the Arctic Ocean are presented in the Tables 1, 2, and below we consider the features of river mouths of the region laying emphasis on the estuarine and estuarine-deltaic types.

5. Simple river mouth area
The Onega River flows into the Onega Bay of the White Sea, forming a mouth of simple type. The reaches of rapids do not freeze in winter and encourage large amount of frazil blocking up the river bed (sometimes up to 70%).

The lower reach of the Onega River is under influence of tide. Investigation of propagation of tidal wave into the river mouth of the Onega River (at the distance 6 km above the river mouth cross-section) under ice cover shows the variation of $S$ through the depth at low water during tide cycle from 2 to 10‰ ($H=9$ m) at the depth 2.0 and 8.0 m, correspondingly [Debolskii et al. (1984)]. During the ebb-tide phase $S_{e}=2\%$ was observed at the depth 1.0 and increased till the value $S_{b}=10\%$ at the depth 3.5 m, that stayed constant to the bottom.

Estimation of the parameter of stratification $n=\Delta S/<S>$, where $\Delta S=S_{b}-S_{s}$, and $<S>$ is the depth averaged salinity, gives $n=1.33>1$, that corresponds to strong stratification and availability of salt wedge [Mikhailov (1998)]. Measured velocity profiles show unilateral movement of water through the depth during ebb-tide phase – to the sea, and during flood-tide phase – upstream.

Comparison of calculation of distance of intrusion of brackish water $L$ into the river by one dimensional model [Sezeman (1987)] with the measurements of $S$ at the mouth of the Onega River shows adequate description of the simple tidal river mouth by 1D model. At mean water discharge 180 m$^3$/s during winter low water period the calculation of penetration of brackish water into the river at ebb-tide phase gives the value $S=11.2\%$ in the river mouth cross-section and $S=2.8\%$ in the cross-section upstream the river (8 km) correspondingly, that is confirmed by the experiments. In winter, $S_{s}$ of the White Sea reaches 28‰ [Mikhailov (1997)].

6. River mouth areas of estuarine type
The Mezen and Kuloy rivers have estuaries of classical funnel-shape. Main hydrological characteristics of these river mouth areas are presented in Table 1 and 2. Ice cover formation in the estuaries of the Mezen and Kuloy rivers occurs under the influence of high tides. As a consequence of this the freeze-up of the Mezen River begins on the upper reach of the river and then propagates downstream in contrast to the majority of the rivers flowing into the arctic seas [Il’ina, Grakhov (1987)]. The feature of these rivers is formation of ice dams blocking the river mouth reaches in winter. At the Mezen River such a dam is formed, as a rule, at a distance of 20 km upstream the river mouth cross-section, and at the Kuloy River – 15 km. Fast ice is formed downstream the ice dam only at the banks of the estuaries, whereas the middle area of them is occupied by small and large ice floes and stamukhas. As mixing of river and sea waters in the Mezen estuary occurs under the influence of strong tides the type of well mixed estuary is formed. Mean horizontal gradient of $S$ is $\sim 1.2\%$ per 1 km, the variation of vertical gradient is quite small.
The Khatanga River flows into the Khatanga’s Bay of the Laptev Sea forming estuary. The Khatanga’s Bay is a large tidal estuary of the length 220 km, width 54 km, and depth ~29 m. As the river itself it is covered with ice from October till May.

The Anabar River flows into the Laptev Sea also forming long and shallow estuary. The length of the estuary is ~24 km, its width is 5–7 km and the depth varies in the range 3–10 m. In the southern part of the estuary there are many sandbanks. Freeze-up in the Anabar estuary begins at the end of September and it freezes through from December till May.

7. River mouth areas of estuarine-deltaic type

The Yenisei River belongs to the rivers with a mouth area of estuarine-deltaic type. Main hydrological characteristics of the Yenisei mouth area are presented in Table 1 and 2. On the lower reach, the river begins to freeze at the beginning of October with the formation of great amount of frazil which results in autumn ice drift. Freeze-up comes at the end of October at the lower reach of the river, at the middle of November in the middle stream of the river and at the end of November till December at the mountain part of the river. On some reaches of the river strong acing fields appear. Debacle of the river begins firstly on the upper reach at the end of April, and then on the lower reach at the beginning of June. Spring movement of ice is accompanied by strong ice jams especially on the upper reaches of the river, the water level increasing up to 20 m.

The mean width of the estuary varies from 6.2 km at the Cape Sopochnaya Karga to 12.9 km at the village Karaul. The mean depth of the Yenisei estuary abruptly changes at the Cape Sopochnaya Karga from the value of 30 m in the near-sea area to 8 m in the upstream zone. Such morphometry of the estuary promotes the increase of long surge waves, which reconstructed the ice-covered stream through the depth in the estuary. The propagation of the positive storm surge wave upstream the estuary was registered by values of fluctuations of water level at different stations: Island Dixon – 0.22 m, Cape Sopochnaya Karga – 0.5 m, village Karaul – 0.3 m, city of Igarka – 0.24 m. At the distance of 70 km upstream the Cape Sopochnaya Karga the stream became unilateral towards the river for 1 day, and at the distance of 377 km upstream the Cape Sopochnaya Karga the stream stopped for 5 hours. Due to the change of the stream to the negative storm surge the unilateral current towards the sea appeared for 1.5–2 days.

The modern study of salinity profiles shows vertical gradients of $S$ and $t$ reach 5–10‰ and 1.5–2.0ºC per 10 cm correspondingly [Harms et al. (2002)]. Longitudinal gradients of $S$ in summer reach 0.5‰ per 1 km. In the Yenisei River, salt water penetrates deep into the estuary due to strong stratification. The zone of interaction of fresh and salt water in summer is located in the middle of the estuary near the Cape Sopochnaya Karga (Figure 2).

The values and directions of velocity at ice-covered river mouth of the Yenisei River were measured with the help of autonomous system of probes in the surface and bottom layers of water [Graevskii et al. (1984)]. The measurements were made on the reach between Cape Sopochnaya Karga (71°50’ N) and Lipatnikovo rapid (555 km) (Figure 2). The last station at which the reverse near bottom current of 0.2 m/s was registered was located near the village Karaul (265 km from the Cape Sopochnaya Karga, ~70°10’ N). At all the stations downstream the Karaul the reverse currents were observed in the surface and bottom layers in correspondence
with the tidal fluctuations of water level. Comparison of the results of measurements presented by Harms et al. (2002) with those by Graevskii et al. (1984) shows the shift of the boundary of brackish water intrusion to the south in winter season.

The Ob, Pur and Taz rivers belongs to the rivers with mouth areas of estuarine-deltaic type too. On the lower reaches, they begin to freeze at the beginning of October and by the end of October the lower and middle reaches of the rivers are covered with ice. Debacle of the river begins firstly on the upper reach at the middle of April accompanied by the ice jams, and then on the lower reach – at the first half of May. At the end of January, the water on the lower reach of the river stagnates (its color becomes reddish), that results in suffocation of fish. This phenomenon is observed on the other Siberian rivers slowly flowing in the lower reach as the Ob River does. It is known that the suffocation comes from the upper reach of the river.

Investigations of the brackish water intrusion into the Ob-Taz estuary show sea water penetration only into the northern part of the bay (Figure 3) independently of the season. A special type of mixing in the Ob estuary is stable stratification during all the year including the flood period [Ivanov, Svyatskii (1986)]. It is destroyed only by strong storms after which it is quickly restored. The examples of the salinity profiles are presented in Figure 4. The distance of brackish water penetration \( L \) varies in wide range depending on the river flow and ice conditions. Mean depth of the estuary changes in the range 10–16 m, the length of the northern part of the estuary is ~500 m, the width varies from 35 to 85 km, the mean width being 52 km [Rusin, Svyatskii (1983)]. The calculation of \( L \) by the 2D model gives the most accurate results when the depths averaged through cross-sections were assigned [Ivanov, Svyatskii (1986)]. The distance is measured from the entrance into the bay at Cape Poelovo. The changes of \( L \) during the year presented in the Figure 5 show the shift of the boundary of brackish water inside the estuary in winter. It is known from observations that brackish water does not penetrate upstream the estuary than the Cape Trekhbogornyi. In the northern part of the Ob estuary in winter the salt water wedge of stable stratification is observed. Numerical calculations of the distance of brackish water intrusion into estuary depending on the bottom slope show the increase of the distance as the bottom slope decrease, that is confirmed by the experimental data [Ivanov, Svyatskii (1986)].

8. River mouth areas of deltaic type

The Lena River flows into the Laptev Sea forming one of the largest deltas in Russia with complex drainage network. The annual river water discharge at the delta head is 16700 m³/s [Mikhailov (1997)]. The braiding of river channels freezes at the middle of October, the debacle occurs at the end of May and the ice drift – at the beginning of June. The duration of freeze-up increased from 223 days a year at the delta head till 273 days on the delta coastline. During breaking up water from the upper reaches covers islands and widen branches of the river, the fairways of which are covered with unbroken ice. This ice does not drift to the sea melting completely in the river branches. Spring debacle often accompanied by ice jams on the reaches upstream of the delta head which results in water level rise by 5–10 m. The ice jams in the river branches of delta cause the redistribution of water into the adjacent branches without considerable water level rise.

The Yana River empties into the Laptev Sea (to the east from the mouth of the Lena River), forming the delta. At the delta head, the average date of appearance of ice phenomena is 29 IX, and freeze-up begins on 5.X (Table 2) The most of delta channels freeze through the depth in the
rapids, once in every three years the river freezes through the depth at the delta head. At the bars of the main branches the thickness of ice bar channels is ~1.8–1.9 m, but they do not freeze through the depth due to the considerable salinity of water (up to 20‰) [Lower reach…(1998)].

The value of spring flood is low due to small amount of snow supply in the river basin. During breaking up of the river the most amount of the spring flood flows in the presence of ice cover in the river delta. The ice in the edges of bar channels breaks, whereas in the bar flanks the melting of ice covered with water may lengthen till the beginning of July. Ice in the bar channels forms ice “bands”, which enable one to follow the direction of and determine the minimum depths of water from the air. In case of shallow reaches of bar channels freeze through the depth the ice dam appears which causes submergence of coastal tundra. Debacle of the Yana nearshore zone begins at the beginning of June. A little later the ice melting begins in the Yanskii Bay and the east part of the Laptev Sea. Fat ice of thickness more than 2 m reaches here extreme length ~400

Figure 2. Scheme of the estuary of the Yenisei River.

Figure 3. Scheme of the estuary of the Ob River.

Figure 4. Water salinity profiles at the northern part of the Ob estuary.

Figure 5. Distance of brackish water penetration averaged during 1965–1972 into the northern part of the Ob estuary.
km. In summer, it is located in a form of the Yana Ice Massive. Sometimes it remains till next winter. The location of ice massive in the bay and sea depends on prevailing winds over the sea.

The Indigirka River flows into the East-Siberian Sea, forming the delta. In winter, river water flow is low, and the lower reach of the river freezes to the bottom. The river freezes at the beginning of October and breaks up at the beginning of June. On the lower reach, at the distance of 940 km from the river mouth, the Indigirka River is open for navigation in July–September [Babich et al. (2001)].

At the gauging stations on the Lower Indigirka, mean annual dates of breaking-up fall on the first decade of June. Then the wave of breaking-up spreads downstream and leads to the opening of the area above the mouth bar firstly in the bar channel, and then in bar flanks. Here ice freezing together with ground may stay till the beginning of June, some separate ice bulks may even stay till August. Dates of breaking-up of the river mouth bar depend on the character of cyclonic activity, air temperature, value of solar radiation, and strong wind, causing surge-induced water level variations over the bar, hummocking and ice movement. The date of the breaking-up of the mouth bar is influenced by the storm surge phenomena during the freeze-up. Strong storm surges intensify sea water intrusion to the bar, which results in formation porous and slack ice, easily destroyed in spring; negative-surged winds, in the opposite, concentrate fresh water in the bar channel, which leads to formation of quite strong ice cover and later opening of water surface. The main hydrological factors influencing the date of break-up on the lower river reach are the character of spring flood and storm surges. Warm river water, coming to the mouth bar with a flood wave, heats the ice cover and destroys it mechanically. Strong negative storm surge contemporizing with the flood wave accelerates the breaking up in the mouth bar area.

The Kolyma River flows into the East-Siberian Sea forming a vast delta. The annual discharge of the river is 3800 m$^3$/s. Its water regime in contrast to the Yana and Indigirka rivers is characterized by the usual features for the East-Siberian rivers, as high spring flood, frequent summer rain floods, smaller than that the spring one, and low water winter period. The river freezes at the beginning of October and breaks up at the end of May – the beginning of June. The range of water level fluctuations in the lower reach runs up to 13 m. Strong ice jams observed during the spring ice drift which causes the water level increase up to 16 m over the low water period level.

13. Discussion and conclusions

Many factors such as tidal mixing, the shape and dimensions, ice conditions and geographical position affect the distance of brackish water intrusion into estuary. Because of absence of solid ice in the Mezen and Kuloy estuaries there is complete mixing of sea and river water. Long-term observation data show a permanent pycnocline in the Ob and Yenisei estuaries. Tidal range in the north-west of the Ob estuary reaches up to 1.8 m with velocity of tidal current ~0.7 m/s [Harms et al. (2002)]. In the Yenisei estuary, residual advection of brackish water due to tidal activity is negligible. Vertical profiles of temperature and salinity in the Ob and Yenisei estuaries in summer show well-defined two layer structure of water. Long-term average salinity at the bottom in the Yenisei estuary near the Cape Sopochnaya Karga fluctuates from 8 to 18‰, and in some years exceeds salinity observed in summer [Harms et al. (2002)]. This fact indicates the increase of $L$ under the ice cover that was also observed by Graevskii et al. (1984). The
observations show that in the Yenisei estuary difference between the bottom and surface water salinity in summer is larger than that in the Ob estuary. In a narrow and deep estuary of the Yenisei River the brackish water penetrates farther than in a wide and comparatively shallow Ob estuary, in spite of weaker tides in the nearshore zone of the Yenisei River.

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References
Bed deformation in under-ice rivers (result of numerical and laboratory modeling)

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Abstract
Numerical experiments performed with the 1-D and 2-D model of bed deformation in under-ice rivers demonstrated that disturbance wave in ice-covered flow causes bed movements in both reservoir water opening and cross-sections near ice edge, contrary to the case of flow with open surface. Silting is observed between these sections. Its amount is lesser for larger water openings. Amount of erosions depends on volume of discharge and its duration, length of water opening, coefficient of ice roughness, size of soil particles, porosity and density of soil. Affect from increase of discharge is greater at the power site than near ice edge; as roughness of ice increases, so does erosion near the edge concurrently decreasing at the power site; increase of water opening increases erosions at the power site and decreases them near the edge; increase of size of soil particles at both power site and ice edge decreases erosions. Acquired time dependence of rate of current erosion to size of maximal erosion in a moment of completion of discharge at the power site is the same, being independent from influence of all accountable factors in the model.
1. One-dimensional model of sediment transport in ice-covered flows

Sediment transport processes in ice-covered rivers can significantly differ from those occurring in open-water conditions.

The difference between sediment transport in open and ice-covered channels can be due, primarily, to changes in the overall bed resistance and the lack of fine sediment input from the watershed. Here, the suspended sediment field is formed mostly due to bed material.

The one-dimensional model of sediment transport in ice-covered flows (Debolskaya E.I. et al., 2006) is based, as most models for open flows, on Saint Venant and continuity equations for the calculation of flow velocities and water levels (in a nonconservative form) or discharges (in a conservative form) for the liquid phase and Eksner continuity equation for the solid phase.

It is known that significant erosions are observed after water discharges from reservoired power stations. Obviously as release wave front reaches the edge of ice cover after going through reservoir water opening, resistance of motion abruptly changes, causing rearrangement of velocity profiles and modifying processes of transport of suspended sediments and bed silt. What changes are caused in shape of bottom?

2. Results of numerical experiments with use of one-dimensional model

Calculations have been lead for the virtual river with average characteristics: the width 500m, depth of 4 m, the charge of water 1000 m$^3$/s, a slope 0.00005, $k_o=0.02\text{ s/m}^{1/3}$, an edge of an ice is located on distance of 1900 m from dam location that corresponds, for example, Svir river (North of Russian European Part).

On fig. 1 bottom level changes on river length are showed beginning from dam location in initial sites and sites, adjoining to an ice edge at various outflow volume. Continuous lines correspond to the outflow volume exceeding the household discharge in k=4 of time, dotted - in k=3 time and dashdot - in k=2 time. It is visible, that at k <4 erosion at an edge even exceeds erosion in dam location a little. The calculated absolute sizes of erosion exceed observed in the nature that is caused by application of one-dimensional model.

![Figure 1](image-url)

**Figure 1.** The bottom level on river length beginning from dam location (results of the numerical experiment): at initial sites (a) and at sites, adjoining to an ice edge (b).
3. Comparison of numerical experiments results with laboratory observation data

For testing model a series of experiments with the open surface, with imitation of an ice cover has been lead with different loadings at various charges of water. The hydraulic flume of circulating action represents the channel of rectangular section with 0,24 m width, 0,35 m depth and 9 m length. Lateral flume walls are executed from glass.

From laboratory experiment speed and depth of a stream, the ground particles size were known authentically. They have served as entrance parameters of numerical model. The dynamic slope and resistance coefficient are calculated from an assumption, that a stream is stationary. Experiments were spent at eroding velocity and noneroding velocity. Parameters of a stream at the open surface and with presence of a covering were identical.

Experiments have shown, that with other things being equal the covering causes change in a regime of sediment transfer, formation of bed forms and local deformations of a bottom. As a major factor for testing model formation of local erosion at an edge of a covering has been chosen at sharp increase in speed in entrance range. On fig. 2 the photo made at carrying out of such experiment is presented.

![Flume experiment with cover.](image)

From figure it is visible, that under an edge the bottom deepening starts to be formed with the further increase of a bottom level downstream. At the loaded covering process of deformations was intensified considerably. On fig. 3 curve of bed level changes on length at an initial site and at an edge of a covering which settled down on distance of 200 sm from a head of a flume are resulted.
Curves are constructed according to the calculations executed on model at parameters of a stream, corresponding the experiment presented on fig. 2. In numerical experiment speed of formation and erosion dimension (about 0.02 m) at an edge of the covering coincided with observable at carrying out of experiment in the flume.

4. 2-D model of the bed deformations in ice jams formation conditions.

The two-dimensional longitudinal-cross-section model of deformations has been developed for an estimation of bottom and bank deformations in conditions of ice difficulties on the wide rivers with curvilinear sites in view of passage of flood and discharge waves and an opportunity of formation of ice jams. The two-dimensional model is put in a basis of offered ice jam model (Debolskaya et al., 2004) developed earlier, added by a condition of a mobile bottom and the two-dimensional equations of deformations or sediment transfer

Under action of a disturbance wave (in our case is a release wave from waterside structure), which in itself serves as the reason of flooding, are created, except for that condition for formation of ice jams. Development of jams in turn or leads to increase in intensity of the flooding caused by a disturbance wave, or after stopping of a release wave is the unique reason of continuation of flooding. A task about bed and bank deformations in such conditions depends not only on parameters of water object, a release wave, characteristics of sediment and combinations of these parameters, but also and from time and a place of occurrence of a jam, its duration.

The basic equations, boundary conditions, criteria of an ice cover destruction, condition of ice floes plunging under an edge of a continuous ice cover, a condition in a place of formation of a jam and algorithm of calculation of 2-D model of ice jam forming have been described in work (Debolskaya et al., 2004). For brevity we shall show only the basic equations of these models. The 2-D equations of a liquid motion and the continuity equation, received at integration of the basic three-dimensional equations on depth of a stream at presence of an ice and durante absentia a wind, were used in the form of:
\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} = -g \frac{\partial H}{\partial x} + \frac{\partial}{\partial x} \left( A_x \frac{\partial u}{\partial x} \right) + \frac{\partial}{\partial y} \left( A_y \frac{\partial u}{\partial y} \right) + \left( \frac{\tau_{ix} - \tau_{hx}}{\rho h} \right) \tag{1}
\]

\[
\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} = -g \frac{\partial H}{\partial y} + \frac{\partial}{\partial x} \left( A_x \frac{\partial v}{\partial x} \right) + \frac{\partial}{\partial y} \left( A_y \frac{\partial v}{\partial y} \right) + \left( \frac{\tau_{iy} - \tau_{by}}{\rho h} \right) \tag{2}
\]

\[
\frac{\partial H}{\partial t} + \frac{\partial Hu}{\partial x} + \frac{\partial Hv}{\partial y} = 0 \tag{3}
\]

where \(x, y\) are the cartesian coordinates; and, the positive axis \(x\) is directed on a stream, and an axis \(y\) across; \(u, v\) are on depth-averaged longitudinal and cross-section components of water velocity accordingly; \(g\) is acceleration of gravity; \(H=h+h_0\) is a water surface level; \(h_0\) is a bed level; \(h\) is depth of a stream; \(\tau_{bx,x}\) and \(\tau_{by,y}\) are components of shear stress at the bottom and surfaces of an ice, accordingly; \(A_x\) and \(A_y\) are longitudinal and cross-section coefficients of turbulent viscosity accordingly.

For definition of turbulent viscosity coefficients it was used empirical ratios, where and \(A_x=\gamma_x hu, A_y=\gamma_y hv\); where \(x, y\) are empirical constants. Relation between shear stress on solid surfaces and other characteristics of a stream is set by ratios

\[
\tau_{bx} - \tau_{nx} = \frac{\rho \lambda U_i |U_i|}{2} \tag{4}
\]

\[
\lambda = \frac{2gn^2}{h^{1/3}} \tag{5}
\]

where \(\lambda\) is coefficient of hydraulic friction, \(n\) is the winter generalized coefficient of a group roughness in Manning formula, \(\bar{U}\) is a vector of horizontal velocity, indexes \(i=1,2\) correspond \(x\) and \(y\) to coordinates, \(b\) and \(i\) - to bottom and ice surfaces accordingly.

The mass conservation equation of transfer sediment or the equation of deformations in two-dimensional statement was used, as it is accepted for the open streams (D. Gessler et al, 1999) in following record:

\[
\frac{\partial}{\partial t} [(1-p)z_b] + \frac{\partial Q_{sx}}{\partial x} + \frac{\partial Q_{sy}}{\partial y} = 0 , \tag{6}
\]

where \(z_b\) is a deviation of a bottom surface, \(p\) - porosity of a ground material, \(Q_{sx}, Q_{sy}\) are longitudinal and cross-section components of the sediment charge on unit of width. For their record the simple Engelund dependence was used. This is not providing division of sediment on weighed and ground:
\[ Q_{xx} = 0.05 u^2 \left( \frac{\tau_{bx}}{(\rho_s - \rho)gd} \right)^{3/2} d \left( \frac{1}{\rho_s / \rho - 1}g \right) . \]  \[ 7 \]

\[ Q_{yy} = 0.05 v^2 \left( \frac{\tau_{by}}{(\rho_s - \rho)gd} \right)^{3/2} d \left( \frac{1}{\rho_s / \rho - 1}g \right) , \]  \[ 8 \]

where \( d \) is average diameter of particles, \( \rho_s \) is density of particles, \( \rho \) is density of water.

As parameters the task includes values of bed and flood-lands roughnesses, a longitudinal slope of a bed bottom, the form of cross-section sections changeable on length (width up to water edge \( B (x, y) \) and depths in each settlement point) in a stationary condition, characteristics of a release wave (the factor describing the attitude of height of a release wave to initial depth, duration and time of the beginning), thickness of an ice and its strength properties, characteristics of sediment (hydraulic size and porosity).

After receiving of the stationary decision of the equations (1) - (3), corresponding to morphometric parameters of a considered channel, on the left border "disturbance wave" is set in the form of increase in depth (break of dams, for example), that is height of a release wave. On solid surfaces (banks) both components of velocity were set zero, on the right border - a condition of radiation.

As undestroyed ice cover till the moment of performance of a break condition was considered frozen to coast, at performance of plunging criterion depth decreased for the size equal to thickness of an ice floe (one of assumptions of model is that the ice floe has the cross-section size equal to the initial size of an ice field as interactions between separate ice floes it is not considered and the reasons for their subsequent destruction in a cross-section direction are not present).

Speed of filling of a channel by the broken away ice floes can exceed speed of level rise due to action of a release wave, and the jam starts to be formed even during action of flash. Depending on parities of entering parameters of a task, such jam can not develop in the further up to a condition of full overlapping a channel, but below on current there can be a jam much more intensive, that is with full overlapping a channel. The series of jams was sometimes formed.

At other parities of parameters of a task the jam was formed after the flash stopping when current velocities are still great enough for performance of the plunging conditions, and the general depth of a stream does not increase any more. The output of water on an ice surface was not provided, as estimations show, that always at first break of an ice field from bank occurs and after that the broken away ice floe moves together with a stream of a liquid. At full overlapping a channel by consistently dived ice floes the velocities of water in the site of jam formations become zero.

5. Results of numerical experiments with use of two-dimensional model

As a result of calculations under various scripts it is possible to track dynamics of distribution of flooding and the bed deformations caused by action of a release wave and formation of jams.
Below illustrations are led to the script with initial depth of a stream of 5 m, thickness of an ice of 1 m, initial width of a channel 200 m, coefficient of a bottom roughness in a bed of 0.03 s/m$^{1/3}$, on flood-lands 0.045 s/m$^{1/3}$. The length of a settlement site of 25 km, the initial form of cross-section section is the oblique trapeze, an edge of an ice was in 5 km below dam location.

On fig. 4 there are bed relief changed after passage of a release wave: a - after 20 minutes release wave with height three times surpassed initial depth of a stream ($\kappa_{rw}=3$), one jam was formed on distance of 8 km from a dam; b - after 20 minutes release wave with height in 2 times surpassed initial depth of a stream ($\kappa_{rw}=2$), two jams were formed on distances of 6 and 8 km from a dam; c - after 20 minutes release wave ($\kappa_{rw}=2$) in absence of an ice. Dam location is in the foreground. It corresponds to the initial form of cross-section section of a channel.

**Figure 4.** The bed relief after 20 minutes of a release wave $\kappa_{rw}=3$ in the ice covered stream (a), $\kappa_{rw}=2$ in the ice covered stream (b), $\kappa_{rw}=2$ in the open stream (c).

On fig. 5 there are surfaces of a bottom level deviations from initial position (actually deformations) in two projections for a release wave $\kappa_{rw}=2$ for ice covered and for the open stream. All the same parameters, that on fig.4.
Figure 5. The bed deformations surfaces in two projections (a) and (b) for a release wave $\kappa_{rw}=2$ for ice covered and for the open stream.

Test calculations on two-dimensional model have shown conformity with the result received by one-dimensional calculations, as about significant deformations near damsite at passage of a release wave, and about occurrence of significant erosion under an edge of an ice cover and deformations in a place of jam formation.

On fig.6 the surface of deformations for release wave $\kappa_{rw}=3$ is presented. Comparison fig. 5 and fig.6 allows to draw a conclusion, that the increase release wave in 1.5 times causes increase in deformations almost on the order. Oscillating character of deformations in this case is caused by that before formation of a constant jam on distance of 8 km, two more jams existed during short time upstream. Then they have been broken through under action of a release wave.
**Figure 6.** The bed deformations surface at a release wave $k_{rw}=3$

Fig. 7 shows change of surfaces of deformations eventually: after 20 minutes double release wave (a) and after 40 minutes double a release wave (b). It is visible, as deformations in a zone of a jam eventually increase.

**Figure 7.** The bed deformations surfaces at double release wave through 20 (a) and 40 minutes (b).
Summary
Numerical experiments performed with the 1-D and 2-D models of bed deformation in under-ice rivers demonstrated that:

- unlike a stream with the open surface the disturbance wave in a ice-covered stream, causes erosion of a bottom both at damsite, and at river section, adjoining to an ice cover edge. Silting is observed between these sections. Its amount is lesser for larger water openings;
- in the absence of the ice jam the increase of a release wave acts on erosion at damsite, than at an ice edge more intensively; at formation of the ice jam the increase of a release wave acts on erosion in the range of the ice jam, than at power site more intensively;
- with growth of an ice roughness erosion at an edge increase, and at power site decrease;
- the increase of an ice-hole leads to increase of erosion in power site and to reduction of erosion at an edge;
- increase of size of soil particles at both power site and ice edge decreases erosions;
- acquired time dependence of rate of current erosion to size of maximal erosion in a moment of completion of discharge at the power site is the same, being independent from influence of all accountable factors in the model.

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References


Freeze-up processes, rivers and oceans
Frazil Increase and Ice Thickness Formation of Frozen Rivers

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1. Introduction

In Hokkaido, which is a cold and snowy region, ice forms on rivers mainly due to the temperature drop in winter. Yamashita et al. produced an ice formation map of rivers in Hokkaido for the period between December 1986 and March 1987. Fukuda et al. produced a distribution map of freezing index values (absolute values), which are found by accumulating only the negative values of daily mean temperatures, in Hokkaido for the period between November 1974 and March 1975. From these maps, it can be seen that ice is likely to form on rivers in eastern and northern Hokkaido compared with other parts of the prefecture, and that ice is thicker in these regions because the freezing index values, which are correlated with ice thickness, are larger. Against such a background of ice formation in winter, rivers in eastern and northern Hokkaido are facing problems, including the elevation of water surface caused by ice jams and difficulties in water intake due to frazil formation.

The elevation of the water surface caused by an ice jam was observed on March 18, 1995, on the Shokotsu River located in eastern Hokkaido. Ice flowed down to the 3.6-km-long section of the river, between KP 16.6 and KP 20.2, accumulated and blocked the river channel. The water surface rose and reached a peak of 14 cm below the estimated high-water level at KP19.3 (Kamishokotsu Gauging Station). Inland flooding and other damage was avoided as ice was removed immediately. According to the assumption of Shen et al. concerning the cause of this ice jam formation, snow melted by warm air and rain raised the water surface, and ice sheets destroyed by the elevation of water surface flowed downstream and accumulated and blocked the KP16.6 section, where the riverbed gradient is gentle and the river channel is narrow and meandering. It is considered important to obtain knowledge on longitudinal ice sheet profiles when discussing river disaster prevention in winter. The Yubetsu River is situated approximately 30 km south of the Shokatsu River. As shown in Fig. 1, which displays their average riverbed levels, the riverbed gradient (KP5.0 – KP24.0) of the Yubetsu River is 1/290, and is steeper compared with 1/410 of the Shokotsu River. While it is presumed that ice is more likely to form on the Yubetsu River since it is situated at a higher altitude, no ice jam formed on the river on March 18, 1995. This was probably because of the difference in the amount of rainfall, since the atmospheric temperature was at the same level in the two areas according to the weather data. However, since no ice jam formation has been reported on the Yubetsu River up to now, the difference in ice formation is thought to be affected not only by the difference in weather conditions, but also by the difference in river conditions.

Difficulty in water intake due to frazil formation was observed in 2000 at the Nagayama Intake Facility (near KP 166) in the upper reaches of the Ishikari River in northern Hokkaido.

![Figure 1. Average riverbed levels of the Shokotsu and Yubetsu rivers (measured in 2000)](image-url)
Frazil entered, accumulated and blocked the intake. To solve this problem, research on measures against frazil was conducted. Based on the findings of such research, around-the-clock anti-frazil measures are now taken by installing an iron ice boom, an antifreeze fence, a wooden raft, a submerged mixer and a surveillance camera. Therefore, no difficulties in water intake have occurred since 2000. However, if ice sticks to the ice boom, the ice grows upstream and forms an ice sheet, under which frazil submerges and flows down to the intake. As a measure against it, ice sheets are destroyed artificially whenever they form. While there is some empirical knowledge of frazil behavior, such as the high possibility of formation on cold days, it has not been fully clarified due to the difficulty in observation. It is desirable to acquire further knowledge on frazil behavior.

In this study, an attempt to clarify the difference in ice formation on different rivers and frazil behavior was made through periodic field observation, by establishing several observation points in the longitudinal direction along the rivers. Although detailed fixed-point observation has been made in past studies, there are few cases of consistent observation in upper and lower reaches as far as the authors know. The observation results of this study will thus contribute to future research and the examination of frozen rivers.

A “year” in this paper means from April of the current year to March of the following year. For example, the year 2000 means from April 2000 to March 2001. The term KP stands for kilo post, which is the distance (km) from the estuary and is a positive value if it is upstream. Frazil means ice crystals (10^{-5} to 10^{-2} m) and their aggregates (frazil slush, 10^{-3} to 10^{-1} m).

2. Field measurements

(1) Points and dates/times of measurements
Field measurements of the Shokotsu and Yubetsu rivers were conducted at the points shown in Fig. 2. There were nine measurement points in total, of which five were in the Shokatsu River and its branch and four were in the Yubetsu River and its branch. In this paper, codes are

![Figure 2. Observation points of the Shokotsu and Yubetsu rivers](image)
allocated randomly to measurement points, and the average water surface width B [m] of each point is indicated in the figure. The Setose Dam (Yubetsu River Dam Power Plant) is situated at approximately KP 35 in the upper reaches of the Yubetsu River. The measurement period was between January and March 2007, and the average time of measurement at each point was 9:00 (SH1), 10:00 (SH2), 11:00 (SH3), 14:00 (SH4), 15:00 (SH5), 14:00 (YU1), 11:00 (YU2), 10:00 (YU3) and 9:00 (YU4). It is known that the flow rate of a frozen river changes during a day\(^5\). It was thus presumed that the flow rate was increasing at the times of measurements in this study, since most measurements were made in the morning hours.

(2) Measurement items

Measurements were made 81 times in total, at intervals of approximately one week. Measurement items were cross-sectional surveying of ice and frazil heights at each point, photographing mainly of measurement sections and vertical flow velocity, and water temperature measurements on a traverse line of the cross section. The traverse line, which had a vertical average flow velocity correlating the most with the flow rate was selected based on the results of flow measurements during the ice-free period. Automatic measurements of the water level at each point were also made every ten minutes. The surface and traverse ice formation rates, ice

Figure 3. An example of a calculation of the surface ice formation rate (February 27, 2007, Nakayubetsu Station YU1)

Figure 4. An example of cross-sectional surveying (February 27, 2007, Nakayubetsu Station YU1)
sheet areas and frazil areas were found from the measurement results and examined. The calculation methods are as described below.

1) The areas of water and ice or snow surfaces in the analysis range (water surface width x 60 m) were found by vertically correcting photographs, and by calculating the surface ice formation rate by \([\text{area of the ice or snow surface}] / [\text{area of the analysis range}] \times 100\%\). Figure 3 displays an example of a calculation of the surface ice formation rate.

2) The traverse ice formation rate was calculated as \([\text{areas of ice sheet and frazil below the water surface}] / [\text{area between the water surface and riverbed height}] \times 100\%\). Figure 4 illustrates an example of cross-sectional surveying.

3) The ice sheet area is the area of the cross section between the upper and lower surfaces of ice. The frazil area is the area of the cross section between the upper and lower surfaces of frazil.

3. Ice formation rate in the longitudinal direction of the river

To clarify the difference in ice formation on different rivers, the surface and traverse ice formation rates were examined. A comparison was made between them as the two rates were thought to be correlated.

(1) Ice formation rate of each river

Figures 5-a and -b display the surface ice formation rates and 5-c and -d show the traverse ice formation rates. As can be seen in Figs. 5-a and -b, the ice formation rate in the downstream side was higher in the Yubetsu River than in the Shokotsu River. However, while the ice formation rate increased upstream of KP 20 of the Shokotsu River with an increase in altitude, the rate near KP25 of the Yubetsu River was low. Similar measurement results are observed for traverse ice.
formation rates in Figs. 5-c and -d. The result that the ice formation rate was low near KP 25 of the Yubetsu River indicates the absence of upstream ice, which would cause the formation of an ice jam. The low ice formation rate was probably due to the Setose Dam located at approximately KP 35 upstream catching the frazil flowing down the river.

The measurement results revealed that the ice formation rate varied by river and was not necessarily high in the upper reaches at high altitude. It is thus necessary to consider the difference not only in altitude, but also in hydraulic quantity, weather and river channel conditions.

(2) Surface and traverse ice formation rates

Figure 6 is a correlation chart of the surface and traverse ice formation rates. The traverse ice formation rate was lower than the surface ice formation rate and the difference was from approximately 17 to 53% in the range between the solid lines in the figure. It means that, in the data within this range, the traverse ice formation rate was 47 to 83% and the water flow area was 17 to 53% when the surface ice formation rate was 100%. This finding indicates that the traverse ice formation rate can be estimated to a certain degree from the surface ice formation rate, which can be checked visually. When the surface ice formation rate was 100%, the Froude number also changed from 0.07 to 0.21, indicating fluctuation in hydraulic quantity. For all data, the Froude number varied within the range of 0.01 to 0.42.

Outside the range between the solid lines are five data points, for which the measurement dates are indicated, and six data points when the traverse ice formation rate was zero, which means that ice existed within a range of 60 m upstream and downstream of the measurement section although there was no ice in the measurement section. Of the five data points with dates, three were obtained in the downstream section and two were from the narrow branch. While the surface and traverse ice formation rates were almost the same on February 22, other data indicated that there was less ice in the traverse section compared with the surface ice formation rate. Data outside the range between the solid lines were only 14% of all data.
4. Time series variation in frazil area

To grasp frazil behavior, a study was conducted focusing on the relationship between the frazil and ice sheet areas.

(1) Relationship between the frazil and ice sheet areas

Figures 7-a and -b and 9-a and -b show the measurement values of the Shokotsu and Yubetsu rivers, respectively. In each figure, the horizontal axis represents the distance from the estuary and the vertical axis shows the ice sheet or frazil area, and the measurement values of the same day are connected with a solid line. For both rivers, values vary by point. The ice sheet and frazil areas were not necessarily larger in upper reaches than in lower reaches, and the maximum values were observed at approximately KP20 in the middle stream.

The reason that the maximum values are observed in the middle stream can be seen from Fig. 11, which shows the river width in the longitudinal direction. The sections approximately KP 20 in both rivers were narrower than the upper reaches and, on assumption that the flow rate from upstream was uniform, it was presumed that ice, which formed and accumulated upstream where...
the rivers were wider and the flow velocity was lower, flowed down and blocked the narrow middle reaches (approx. KP 20), causing an increase in ice area. The measurement results also revealed that the Yubetsu River had greater amounts of ice and frazil compared with the Shokotsu River.

Next, Figs. 8-a and -b and 10-a and -b show the time series variation in frazil and ice sheet areas at points where the frazil area was not zero. The horizontal and vertical axes represent the frazil and ice sheet areas, respectively, and the values are connected with solid lines in a time series.

Frazil behavior was examined in the longitudinal direction of the rivers by numbering the values in the figures in chronological order. At KP 39.0 in the upper reaches of the Shokotsu River (Fig. 8-b), frazil formation decreased with the passage of time after peaking on January 24, and ice sheet formation increased. Since the traverse ice formation rate was between 61 and 79% and the water flow area was small during this period, it was presumed that some of the

Figure 9. Ice sheet and frazil areas in the Yubetsu River

Figure 10. Time series variation in ice sheet and frazil areas in the Yubetsu River
frazil accumulated and formed ice sheets while the rest flowed downstream. It can also be said that frazil was likely to form at this point considering the presence of frazil in the early stage of ice formation. At KP 19.3 in the middle reaches (Fig. 8-a), frazil formation peaked on January 31, one week later than in the upper reaches. It was thus presumed that frazil formed upstream, flowed down and accumulated in the middle reaches. No frazil was observed at KP 33.5 in the branch.

At KP 18.9 in the middle reaches of the Yubetsu River (Fig. 10-b), frazil formation fluctuated and reached its peak on January 16 and then decreased with the passage of time. Since the fluctuation in frazil at KP 5.4 in the lower reaches (Fig. 10-a) was corresponding to the fluctuation in the middle reaches, it can be presumed that frazil flowed down. The measurement results also revealed that the fluctuation in ice sheets in lower reaches was smaller than in middle reaches.

Since the relationship between the frazil and ice sheet areas varies by point and time, it was divided into five types with focus on time series variation. However, these types were derived from weekly measurement results without consideration to changes during each day. As shown in Fig. 12, the relationship was divided into the following types of periods based on measurement results:

I. Both the frazil and ice sheet areas increased.
II. The frazil area decreased and the ice sheet area increased.
III. Both the frazil and ice sheet areas decreased.
IV. The frazil area decreased and the ice sheet area did not change.
V. The frazil area increased and the ice sheet area did not change.

Figure 12. Time series variation in relationship between the frazil and ice sheet areas
I. Period in which both the frazil and ice sheet areas increased
II. Period in which the frazil area decreased and the ice sheet area increased (the period in which the accumulated frazil turned into an ice plate with the passage of time, thus increasing the number of ice plates)
III. Period in which both the frazil and ice sheet areas decreased
IV. Period in which the frazil area decreased and the ice sheet area did not change
V. Period in which the frazil area increased and the ice sheet area did not change

While the relationship between the frazil and ice sheet areas was Type II in the upper reaches and I and II in the middle reaches of the Shokotsu River (Type II observed in both sections), it was Type II, III and V in the middle reaches and III, IV and V in the lower reaches of the Yubetsu River (Types III and V observed in both sections). The measurement results of this study revealed that frazil was more likely to accumulate and form ice sheets in the Shokotsu River, which has a gently sloped riverbed. However, in the Yubetsu River which has a steep riverbed, frazil was unlikely to accumulate or contribute to ice sheet growth even though it increased, and it was presumed that the decrease in frazil and ice sheet areas occurred concurrently in the river.

5. Conclusion

The following findings were obtained through field measurements of the Shokotsu and Yubetsu rivers in winter:
1) The ice formation rate varied by river and was not necessarily elevated in upper reaches at high altitude. It is thus necessary to also take the hydraulic quantity, weather conditions and the effects of the river channel into consideration.
2) The traverse ice formation rate was 17 to 53% lower than the surface ice formation rate.
3) The relationship between the frazil and ice sheet areas was divided into five types with focus on time series variation. Based on this relationship, it was assumed that frazil in the Shokotsu River was more likely to accumulate and form ice sheets, and that the accumulation and flow-down of frazil varied considerably in the Yubetsu River, where the decrease in frazil and ice sheets occurred concurrently in the late stage of ice formation.

References


Insights from Anchor Ice Formation in the Laramie River, Wyoming

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This paper describes anchor ice formation in the Laramie River, a small river in Wyoming. Anchor ice in the Laramie River forms predominately in riffles at night, although minor amounts also form on sand substrates. Anchor ice crystals are large, indicating that in situ thermal growth is important. Anchor ice forms on the river’s bed, and, when released from the bed, rafts bed sediment downstream. The floating, released anchor ice is a major contributor to the river’s surface ice cover. A conceptual model for anchor ice formation is developed. This model suggests that water supercooled in pools supplies a heat sink that absorbs the latent heat of fusion of anchor ice growing in riffles. The net result is a non-uniform distribution of anchor ice on the bed. We discuss flow parameters important for determining anchor ice growth, and show how a substantial set of parameters influence anchor ice formation.
1. Introduction
Anchor ice forms on a river bed when the river is directly exposed to freezing air temperatures. We present observations of anchor ice formation in the Laramie River, a small meandering river in Wyoming. The observations provided intriguing new insights into anchor-ice and ice-cover formation, revealing that anchor ice is more than an evolutionary form of frazil ice, as commonly assumed, and that anchor ice is a major contributor to ice-cover formation in shallow rivers. Also, these insights indicate the difficulties attendant to extrapolating observations from laboratory studies to field conditions.

The observations are presented in terms of a descriptive model of anchor ice formation along a river reach. This descriptive model uses a more complete parameter characterization of anchor ice formation than has been used heretofore. In the last several years, anchor ice research appears to have focused on finding relationships between a few simple-to-calculate flow parameters (notably Froude and Reynolds numbers) and anchor ice formation (Doering et al., 2001; Hirayama, 1986; Kerr et al., 1997, 2002; Qu and Doering, 2007; Terada et al., 1998). Our observations show how a larger set of parameters influence anchor ice formation.

2. Basic Processes
The basic processes at play during anchor ice formation in a river are cooling of flowing water by heat loss from the water to the overlying atmosphere, turbulent mixing of supercooled water and seed ice crystals into the water body, latent heat release from ice growth, and ice accumulation. Flow-generated turbulence drives water column mixing. Buoyant ice and supercooled water both resist mixing and promote stratification of the water column.

A key point manifest in our observations is the substantial difference in the rates at which supercooled water and seed ice crystals mix into a water body. Supercooled water at, say, -0.1 °C is more readily advected into the stream interior than ice fragments, because this supercooled water is only $10^{-3}$ % lighter than water at 0.00 °C. Ice, in contrast, is about 8.4 % lighter than water at 0.00 °C. It is, therefore, much easier for turbulent eddies to advect supercooled water than ice crystals the river bed. An important, though tentative, deduction is that the river does not cool homogeneously. Rather, water supercools at the river surface, then turbulent eddies entrain ‘wisps’ or ‘blobs’ of supercooled water into the water body. In situ ice crystal growth at the stream bed supports this assumption; as reported subsequently in this paper and elsewhere (e.g., Arden and Wigle, 1972; Schaefer, 1950), supercooled water evidently reaches such crystals otherwise they would not grow.

The rates at which supercooled water and seed crystals mix into a water body depend on the rate of heat loss from the water body and the volumetric flux of seed crystal injection into the water body. They also depend on flow turbulence. The rate of anchor ice growth depends primarily on the rate of heat loss from the water to the atmosphere and the partitioning of ice growth between floating ice types (border ice, frazil, flocs, pans, and floes) and anchor ice. Anchor ice can grow through two complementary processes: frazil collision/adhesion and in situ thermal growth. These two growth mechanisms result in different ice crystal morphologies. Frazil collision/adhesion creates anchor ice masses composed of disk-shaped crystals ~1 mm in diameter. In situ thermal growth occurs when frazil attached to the bed grows as latent heat is carried away from the growing crystals by the supercooled flow. This results in anchor ice masses composed of irregularly shaped ice crystals that may be up to several centimeters in
diameter. Further, the rate at which supercooled water and seed crystals produce anchor ice varies with the height and disposition of roughness elements along the bed.

3. Parameters
An important question is which parameters should comprise a descriptive model of anchor-ice formation, and usefully quantify the processes outlined above. The essential parameters are those associated with the rates of mixing in flowing water, heat transfer from water to air, and latent heat of fusion released as anchor ice grows. Bottom roughness and length of time water loses heat to the atmosphere are also important parameters controlling anchor ice formation. When there is a partial ice cover, the length of time water is exposed to the atmosphere can be expressed as a distance downstream of the ice cover. On cold nights, river water must be exposed to the atmosphere for 30 to 50 m before the first anchor ice forms on the bed. This is the distance required for the water to supercool to the level that drives anchor ice formation. This along-stream distance is non-dimensionalized by water depth \((X/Y)\). For water flow in a fairly simple, straight river or stream, these parameters can be expressed as

\[
A = \varphi(Re, Fr, \tau/\tau_c, d/Y, X/Y, (p-p_i)/p, t_{\phi}/t_m, t_a/t_m, C) \quad [1]
\]

Table 1 defines the physical significance of each parameter. Here, \(A\) is one of the various dependent parameters (e.g., anchor ice volume or crystal size, suitably normalized) describing anchor ice growth; \(t_m\) is a characteristic mixing time scale; \(t_a\) is a characteristic heat-loss time scale; \(t_{\phi}\) is a characteristic ice-growth time scale (Ashton, 1986); \(Y\) is water depth; \(U\) is bulk flow velocity; \(d\) is a representative bed particle diameter or bedform height, \(\tau\) is the shear stress (\(\tau_c\) is its critical value for entrainment of bed material); \(\nu\) is the kinematic viscosity of water; \(\rho, \rho_i\) and \(\rho_s\) are the densities of water at 0°C, ice, and supercooled water, respectively, and \(C\) is volumetric flux of seed crystals into the water. As indicated in Table 1, the parameters \(t_{\phi}/t_m\) and \(t_a/t_m\) express the comparative rates of mixing to anchor-ice growth, and to heat loss to the atmosphere. Ashton (1986) discusses these time scales in detail. For typical flow values, he concludes that mixing is rapid compared to either heat loss or ice crystal growth rates, so temperature can be considered homogenous through the entire water column. As discussed later, we disagree with this conclusion. Small vertical and along-stream temperature variations \((0.1 \, ^\circ C)\) are very important for determining where anchor ice forms on the river bed. Model results (Hammar and Shen, 1995) support the idea of vertical and horizontal variations in the level of supercooling along a frazil-producing channel. The time parameters \(t_{\phi}, t_m, t_a\) (expressed as nondimensional rates in Table 1), along with the density-difference parameters, could alternatively be re-cast to include parameters expressing mixing depths attained by turbulence advection of supercooled water and seed crystals.

4. Observation Reach and Methods
Anchor ice observations were made in a 500m-long reach of the Laramie River near Laramie, Wyoming for eight years between 1999 and 2008. The Laramie River is a shallow, meandering, riffle-and-pool river flowing through a large, semi-arid intermountain basin straddling the Colorado/Wyoming border. The elevation at the study reach is 2200 m and the drainage area upstream of the reach is 2,000 km². Pools with coarse sand beds make up 85% of the river. Riffles, with gravel beds and occasional bedrock outcrops make up the remainder of the river.
parameters affecting anchor ice formation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Physical Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Re )</td>
<td>Reynolds number is ratio of inertial to viscous forces. ( Re = UY/\nu ) is a more meaningful parameter than particle Reynolds number, ( Re* = u<em>d/\nu ), which essentially relates particle diameter, ( d ), to thickness of an erstwhile laminar sub-layer, ( \nu/u</em> = \delta' ).</td>
</tr>
<tr>
<td>( Fr )</td>
<td>Froude number expresses the ratio of inertial to gravitational forces (( Fr = U/\sqrt{gY} )). ( Fr ) is useful for characterizing the flow as sub- or super-critical. ( Fr ) could be combined with buoyancy parameters to form a densimetric Froude number.</td>
</tr>
<tr>
<td>( \tau/\tau_c )</td>
<td>Characterizes mobility state of bed particles</td>
</tr>
<tr>
<td>( d/Y )</td>
<td>Representative relative roughness height of bed elements</td>
</tr>
<tr>
<td>( X/Y )</td>
<td>Normalized length of open water channel (upstream ice-free channel length/water depth)</td>
</tr>
<tr>
<td>( (\rho-\rho_i)/\rho )</td>
<td>Densimetric buoyancy of ice</td>
</tr>
<tr>
<td>( (\rho-\rho_s)/\rho )</td>
<td>Densimetric buoyancy of supercooled water.</td>
</tr>
<tr>
<td>( t/\tau_m )</td>
<td>Mixing rate/cooling rate (Ashton, 1986) (( &gt;&gt;1 ); rate = 1/(time scale))</td>
</tr>
<tr>
<td>( t/\tau_m )</td>
<td>Mixing rate/ice growth rate (Ashton, 1986) (( &gt;&gt;1 ); rate = 1/(time scale))</td>
</tr>
<tr>
<td>( C )</td>
<td>Volumetric flux of ice seed crystals entering water body; volume of ice per unit volume of water body</td>
</tr>
</tbody>
</table>

Between October and March, the river is 8 to 15 m wide, discharges is less than 1.2 m³s⁻¹, water depth is 0.20 to 0.70 m, and flow speeds are between 0.15 and 0.60 m/s⁻¹. Depending on weather conditions, the study reach produced anchor ice and frazil for 5 to 20 days in November and December, and for 3 to 15 days in February and March. Between these periods the river had a continuous surface ice cover.

Most observations were made in the morning between 0700 and 0900 AM during the spring and fall when weather conditions favored anchor ice formation. Each morning trip looked at the previous night’s anchor ice formation. For the most part, we were not out watching the anchor ice form on the bed although we did occasionally make a night-time trip. We can correlate observations for the Laramie River with weather conditions recorded at Laramie Airport located 10 km northeast of the study reach. We also monitored water temperature at the site. In addition to visual observations, between fall 2003 and spring 2005 we used an underwater video camera system to record underwater images of anchor ice near the bed.

The study reach consists of four distinct sub-reaches (Figure 1). Table 2 lists characteristic features of each sub-reach during periods of anchor ice formation. We use the term “typical” for the depth/velocity pairs because they are typical of the range of flow conditions where anchor ice formed in each reach. The listed depth/velocity pairs represent ice-free conditions before anchor ice began to form. Anchor ice significantly changed local flow conditions, increasing depth and/or velocities in some areas and decreasing them in others. As a result, most anchor ice masses observed in the morning had been exposed to changing flow (and possible temperature) conditions throughout their growth during the previous night. Two velocity/depth values are given for each sub-reach; these represent a typical high and low depth/velocity pairs where anchor ice was observed.

On most mornings, by the time we arrived at the reach, considerable amounts of anchor ice had released from the bed and were floating down the river. Our night-time excursions revealed that some anchor ice was continuously released from the bed throughout the night. However, the
Figure 1. Map of Laramie River study reach. Numbers correspond to the sub-reaches listed in Table 2. The horizontal ‘V’ shows the field of view of the inset photograph of sub-reach 3.

Table 2. Characteristics of the observation reach

<table>
<thead>
<tr>
<th>Sub-reach Number</th>
<th>Y (m)</th>
<th>Slope (%)</th>
<th>d (mm)</th>
<th>U (m s⁻¹)</th>
<th>Re</th>
<th>Fr</th>
<th>τ/τc</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.58</td>
<td>0.08</td>
<td>40°</td>
<td>0.30</td>
<td>97,400</td>
<td>0.12</td>
<td>&lt; 1</td>
</tr>
<tr>
<td></td>
<td>0.28</td>
<td></td>
<td></td>
<td>0.48</td>
<td>75,200</td>
<td>0.29</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>2</td>
<td>0.22</td>
<td>0.31</td>
<td>15.7</td>
<td>0.60</td>
<td>73,900</td>
<td>0.41</td>
<td>&lt; 1</td>
</tr>
<tr>
<td></td>
<td>0.45</td>
<td></td>
<td></td>
<td>0.32</td>
<td>80,600</td>
<td>0.15</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>3</td>
<td>0.27</td>
<td>0.02</td>
<td>0.55</td>
<td>0.50</td>
<td>75,600</td>
<td>0.32</td>
<td>≈ 1</td>
</tr>
<tr>
<td></td>
<td>0.22</td>
<td></td>
<td></td>
<td>0.27</td>
<td>33,200</td>
<td>0.15</td>
<td>≈ 1</td>
</tr>
<tr>
<td>4</td>
<td>0.30</td>
<td>0.20</td>
<td>19.0</td>
<td>0.65</td>
<td>109,000</td>
<td>0.38</td>
<td>&lt; 1</td>
</tr>
<tr>
<td></td>
<td>0.16</td>
<td></td>
<td></td>
<td>0.19</td>
<td>17,000</td>
<td>0.15</td>
<td>&lt; 1</td>
</tr>
</tbody>
</table>

*aThe jointed shale bedrock weathers out as 0.250m diameter, 0.60mm thick shingle. We chose 40mm, a representative thickness, as d = d₅₀; approximate estimates of τ/τc given.*
most intense portion of anchor ice runs occurred shortly after the sun hit the water in the morning. Even on the coldest days (windy with air temperatures down to about -25 °C), anchor ice never remained attached to the bed through the entire day. Anchor ice formation and release caused large-scale, short-term variation in both flow and water depth. When anchor ice dams breached, local current speeds often exceeded 1 ms⁻¹, resulting in local scour. Water level often dropped 0.15 to 0.20 m as the water behind a breached dam drained. This is why Table 2 lists only a few representative flow speeds and depths. As anchor ice formed, local flow conditions changed continuously throughout the night, making it very difficult to correlate anchor ice characteristics and flow conditions.

5. Observations
In this section we discuss anchor-ice morphology, distribution, crystal size, sediment transport (rafting), release from the bed, and contribution to ice cover formation along the reach.

**Anchor-ice morphology and distribution** includes the size and shape of anchor ice masses along with the distribution of anchor ice in the stream channel and the relationship between anchor ice masses and local conditions (channel slope, bed substrate, etc.). The most common anchor ice morphology in the study reach comprised large, continuous sheets of low-relief, highly porous ice up to 0.40 m thick, covering the entire channel bed for tens of meters (Figure 2A, C). These sheets formed in gravel-bed riffles and in the bedrock areas (Figure 1, Table 2). The extent and thickness of the anchor ice depended on night time air temperatures, with larger masses of anchor ice forming on colder, clear nights. The largest sheets covered entire riffles (several hundred square meters). Locally, these anchor ice sheets were up 0.60 m thick and formed anchor ice dams that rose up to 0.20 m above the pre-ice water level (Figure 2A). Anchor ice dams always formed at the same locations (within a few meters), just below the upstream ends of sub-reaches 2 and 4 (Figure 1). The river bed is gravel in both of these areas (Table 2). The location of anchor ice dams (and the continuous sheets they form on) appears to be controlled by a steepening of local bed slope (Table 2). The dam locations were remarkably consistent from year to year.

Surprisingly, clumps and layers of anchor ice up to 80 mm thick often formed on the sand bottom of pools during very cold nights. This anchor ice formed in the upper section of sub-reach 3 (Figure 1), where sand moved as low intensity bedload and developed low-amplitude dunes (about 40 mm high). Advancing dunes enveloped the anchor ice masses, making them very hard to identify. The resulting anchor ice had high concentrations of sand. The largest anchor ice masses in this area covered several square meters of sand. For any one event, though, the total volume of anchor ice in sand was always small compared to the amount formed in gravel riffles. As a result, anchor ice that forms on sand beds had relatively little effect on flow conditions; its main effect was to cement the dunes and locally reduce the rate of bedload transport.

The most common location for abundant anchor ice formation was in gravel-bedded riffles (Figure 1, sub-reaches 2 and 4). Initial anchor ice formation comprised scales or balls (Kerr et al., 2002) on the stream bed. Similarly, on relatively warm nights when only small amounts of anchor ice formed, it formed as 30 to 150 mm diameter balls on the riverbed. On colder nights, these scales and balls coalesced into continuous anchor ice sheets that filled entire riffles up to
Figure 2. Anchor ice in the Laramie River. (A) Continuous, 30-cm-thick anchor ice coverage in riffle leads to a 50-cm high anchor ice dam. Released, floating anchor ice is trapped behind the dam. (B) Released anchor ice bridging the open-water surface at the head of sub-reach 3 in November 2002. View is upstream. (C) Large, randomly oriented anchor ice crystals with the anchor-ice-choked channel in the background. (D) Bottom side of a medium-grained anchor ice mass removed from the riverbed. (E) Underwater photo of anchor ice on a gravel bed. (F) Incipient release of anchor ice. This anchor ice has begun lifting from the bed, but has not begun to move downstream. Partially-released anchor ice can remain in place for more than 1 hour.
the water’s surface. In general, the colder and windier the night, the more anchor ice formed. After riffles, anchor ice grew in the intermediate-slope bedrock stretch (Figure 1, sub-reach 1). On the coldest nights, anchor ice formed on sub-reaches with sand beds (sub-reach 3).

**Crystal size** observations yielded another unanticipated finding. For our Laramie River reach, the average anchor ice crystal was greater than 10 mm in diameter (Figure 2C & D), with a thickness up to about 1mm. Individual ice crystals often ranged from 50 to 150 mm in diameter. Anchor ice masses composed of ice crystals greater than 50 mm in diameter were found in relatively quiet water behind anchor ice dams. The ice crystals in these masses were randomly oriented and well bonded together (Figure 2C). Ice crystals in anchor ice dams themselves were 10 to 50 mm in diameter, and formed a honeycomb or imbricated structure that developed into a relatively hard mass that bonded strongly to the bed (Figure 2D). The very large ice crystals observed in the study reach were very different than reported by prior studies which indicate that anchor ice consists mainly of accumulated frazil (Hirayama et al., 1997; Kerr et al., 1997, 2002; Qu and Doering, 2007; Stickler and Alfredsen, 2005). We never saw anchor ice masses consisting solely of accumulated frazil crystals. This observation shows that in-situ thermal growth is a major contributor to anchor ice growth in the Laramie River. However, when snow fell the anchor ice masses consisted of smaller, frazil-like crystals, and anchor ice accumulations were smaller. On such occasions, the influx of seed ice was considerably larger than on colder clear nights. Consequently, more seed crystals mix into the water, the extent of super-cooling is reduced, and anchor ice crystals do not grow substantially.

As turbulence increases, anchor ice crystals may not withstand the buffeting hydrodynamic force of the water, and will not grow to large sizes. Kempema et al (2001) proposed that crystal size in Great Lakes anchor ice masses is controlled by turbulence; the Laramie River observations support this proposal. In a companion set of observations of anchor ice formation in a steep reach of the Cache la Poudre River, Colorado, anchor ice crystals were smaller than in the Laramie River study reach (unpublished data). The Cache la Poudre River reach had a boulder and cobble bed, and thereby was subject to strong, large-scale turbulence that impinged against growing anchor ice accumulations.

**Anchor ice release** occurred mainly during mornings, but some anchor ice is also released during the night. Nighttime anchor ice release was a minor part of the total diel anchor ice run. The major volume of anchor ice formed on any given night released from the bed after the sun hit the water, between 0800 and 1030. During this period, meter-square sheets of anchor ice released from the bed. Released anchor-ice masses often collided with and dislodged downstream, still attached masses, as described by Kerr et al. (2002) in lab experiments. When this occurred, an entire riffle sometimes cleared of anchor ice in a few minutes.

Anchor ice release occurs at the boundary between the ice and bed sediment. That is, anchor ice releases from the bed, it does not release (or fail) along horizontal discontinuities within the anchor ice mass. The weakest bond occurs at the sediment-ice interface. Underwater video images revealed that the large anchor ice sheets that formed in riffles release from the bed in a sequential fashion (Figure 2F). There is commonly a 30-50 mm thick gap between a portion of the sheet and the bed for ½ to 1 hour before the anchor ice finally breaks free and drifts downstream. These partially-released sheets are still attached in places, and still contain some
entrained sediment (Figure 2F). They are free-floating in the center, but still have enough internal strength to remain in place for some time before succumbing to the combined forces of buoyancy and shear. It is possible that differences in sun exposure or in hyporheic flow driven by ice-induced stage change (Kempema and Konrad, 2004) drive the sequential release of anchor ice from the bed.

**Ice rafting** (transport of bed sediment by ice) is controlled by the poorly-understood nature of the bond between the ice and sediment. There was no evidence that anchor ice completely surrounds or encapsulates bed particles as it grew, or that anchor ice grew into the bed of the Laramie River, as reported by Stickler and Alfredsen (2005) for Norwegian rivers. Instead, anchor ice usually formed on the top half of pebbles and cobbles on the bed (Figure 2D). The video system facilitated views of anchor ice on the bed, but it was difficult to get clear images of the ice/bed interface (Figure 2E). However, when anchor ice lifted from the bed, it commonly carried along some bed sediment (Figure 2F). The video records show that this sediment is not completely encapsulated. Released anchor ice rafts sediment as it drifts downstream. As the floating anchor ice mass decays, it drops its ice rafted sediment load back to the riverbed (Kempema et al., 2002). Individual ice-rafted particles can be very large; the largest ice-rafted boulder observed in the Laramie River weighed 8.5 kg!

**Ice cover formation** along the study reach resulted largely from the accumulation of released anchor ice and border ice growth. Released anchor ice drifted as slush downstream until it became trapped by channel constrictions, border ice growth, or a downstream ice cover. The trapped anchor ice then froze and extended the ice cover upstream (Figure 2B). Repeated growth and release of anchor ice from riffles over several days covered the intervening pools and eventually congealed into a continuous ice cover across the river. Our observations differ from some observations in other rivers (Daly, 1994; Michel, 1971) in that we saw almost no frazil flocs, floes, or pans floating on the surface of the Laramie River. Formation of anchor ice indicates that frazil forms in the water column. It appears that essentially all of this frazil is trapped on the bed as anchor ice. Release of this anchor ice contributes to surface ice cover formation. Thus, ice cover growth in the Laramie River consists of a combination of surface border ice growth and released, floating anchor ice that accumulates at constrictions and freezes into a continuous ice cover. The ice cover formation process occurred over a period of days during fall and, together with border ice growth, eventually produced a continuous ice cover over the study reach. As the ice cover formed, less anchor ice grew on the bed because of reduced heat loss from the river to the overlying atmosphere.

6. Descriptive Model

Our observations of anchor ice in the Laramie River, together with findings reported in prior studies, lead us to a conceptual model of anchor ice growth for shallow rivers. This model is sketched in Figure 3. We assume that a partial ice cover has already formed at the upstream end of the reach shown in Figure 3. This corresponds to conditions along our study reach about midway through a freeze up period, when a continuous ice cover forms just upstream of sub-reach 1. A fully turbulent flow of water emerges from under the ice cover at $0^\circ$C, and begins supercooling as soon as it is exposed to the frigid air. Supercooling continues throughout low-gradient, sandy sections of the river, and reaches a minimum temperature at the start of gravel-bed riffles. Wisps or blobs of supercooled water, with varied concentrations of seed ice crystals,
Figure 3. Longitudinal cross section depicting supercooling and anchor ice formation in the Laramie River. Top shows temperature of a water particle flowing through this section, and bottom shows location of anchor ice formation on the bed. Water supercools as it flows through relatively quiet pools, and supercooling is ‘frozen out’ by frazil and anchor ice formation in riffles.

are swept into the flow, penetrating down to the bed. Increased turbulence at riffles further promotes mixing and some frazil formation in the water column, but the main ice growth is anchor ice formation in the riffle. This is, essentially, turbulence “freezing out” the accumulated supercooling, as described by Carstens (1966) in flume experiments. This process does not necessarily raise the water temperature to a residual temperature (Carstens, 1966). Seed crystals and frazil that adhere to the bed grow thermally as the anchor ice is continuously bathed in supercooled water. The thermal growth is evidenced large crystal size in the anchor ice masses.

As stated previously, there was very little evidence of frazil flocs, pans, or floes in the Laramie River. We therefore conclude that essentially all frazil formed in the river is incorporated into anchor ice masses. Based on the large size of crystals found in Laramie River anchor ice masses we conclude that the frazil or seed ice concentration is very low (but unmeasured), enabling water to supercooling in quiet pools. This supercooled water absorbs the latent heat of in situ anchor ice growth in riffles. The net result is that anchor ice is not uniformly distributed along the river bed. Anchor ice preferentially grows in riffles because conditions (high levels of supercooling and turbulence) enhance anchor ice growth there. However, supercooling of the water column that drives anchor ice growth occurs in the pools between riffles (Figure 3).

Note that this descriptive model is based on our observations of the anchor ice distribution in the Laramie River study section, and that we do not have along-stream temperature measurements to support the hypothetical temperature curve shown in Figure 3. There are, however, published observations to support our conclusions. First, laboratory experiments (Carstens, 1966; Ettema et al., 1984) show that low levels of turbulence enable greater supercooling in the water column. Increased turbulence reduces supercooling and enhances rapid ice growth. Second, Ashton’s Figure 5.3 (1986) presents a temperature curve for the St Lawrence River that is very similar to
the hypothetical temperature curve shown in Figure 3. Water is supercooled in the river upstream of Lachine Rapids, and then this supercooling is dissipated at the rapids when anchor ice and frazil grow (this cooling history occurs over an 8 kilometer reach of the river). Third, Qu and Doering (2007) removed floating frazil flocs from the flume in their experiments. Early in their experiments, removing frazil resulted in renewed supercooling of the water column. Removing the suspended frazil reduced the number of ice nuclei to the level where the release of latent heat of fusion was less than heat loss to the atmosphere, so the level of supercooling increased. However, after anchor ice formed removing flocs did not increase supercooling. Qu and Doering conclude that at this point all heat lost to the atmosphere was going to absorb the latent heat of in situ thermal anchor ice growth. The net result is that anchor ice growth is not uniform in the river, and that very thick accumulations of anchor ice can grow in riffles because of the reservoir of supercooled water that arrives from upstream.

7. Discussion
There are similarities, but also important differences, between our observations of anchor ice formation in the Laramie River and prior published observations of anchor ice formation in rivers and flumes.

Some flume observations by Kerr et al. (1997 and 2002) and Qu and Doering (2006) are similar to our observations. Anchor ice growth began as scales and balls on the reach’s bed; with continued growth these forms coalesced into more-or-less continuous layers of anchor ice. Yet, the individual ice crystals in the anchor ice masses we observed were much larger than those reported by prior studies. The large ice crystals formed in the Laramie River indicate that a large portion of the anchor ice masses in it formed through in-situ thermal growth; i.e., the anchor ice mass increased through growth of attached ice crystals rather than by the accumulation of frazil crystals. We did see anchor ice composed of small crystals, but only when snow fell into the river. It is not clear whether snow inhibits anchor ice formation because the numerous snow crystals inhibit supercooling of the water column, or because cloudy conditions associated with snowfall reduce heat loss to the atmosphere. It is clear that snowfall significantly reduces the amount of anchor ice that forms and the size of individual ice crystals in anchor ice masses.

The manner of anchor ice growth on the bed (whether through frazil accretion or thermal growth) plays an important role in the strength or adhesion between the anchor ice mass and the bed. The large ice crystal sizes in the Laramie River indicate that these crystals were continuously bathed in a flow of supercooled water that absorbed the latent heat of fusion of the growing ice, thereby emphasizing the importance of parameters relating rates of mixing, heat loss, and ice growth \((\tau_d/\tau_m, \tau_s/\tau_m)\), relative densities \(((\rho-\rho_i)/\rho, (\rho-\rho_o)/\rho)\) and as influx of seed ice \((C)\). Osterkamp and Gosink (1983) note that once an ice crystal becomes attached, its growth rate may increase by an order of magnitude as it is exposed to increased flow. Although the recent papers listed above observed anchor ice composed of frazil crystals, other studies report fluvial anchor ice masses containing ice crystals up to 40mm in diameter (Michel, 1971; Osterkamp and Gosink, 1983; Schaefer, 1950; Wigle, 1970). So, it appears that the large ice crystals (and attendant thermal growth) observed in the Laramie River is not an anomaly. Michel (1971) reports that large ice crystals are formed in relatively quiet water. The Laramie River observations confirm this: the largest anchor ice crystals were found in backwaters behind anchor ice dams. We occasionally observed gradients in anchor ice crystal size, with larger...
crystals growing above small crystals. We interpret this trend as indicating that thermal growth became more important (relative to frazil-accretion growth) as flow velocity decreased behind a developing anchor ice dam. Arden and Wigle (1972) observed smaller ice crystals resting on larger ice crystals in Niagara River anchor ice samples. Ashton (1986) interprets this stratigraphy as an indication that the larger crystals were deposited first and had a longer time to grow in the supercooled water.

Hirayama et al. (1986), Kerr et al. (1997 and 2002), Terada et al. (1998), Doering et al. (2001), and Qu and Doering (2007) conclude that anchor ice formation and release are functions of the Froude number of the flow, $Fr$, or Reynolds number, $Re$. We suggest that $Fr$ and $Re$ are insufficient parameters for characterizing anchor ice formation (Eq. 1 and Table 1). The pitfall of using just $Fr$ and $Re$ to characterize anchor ice formation is highlighted by comparing sub-reaches 3 and 4 of our observation reach (Table 2). These sub-reaches have similar depths and velocities, resulting in similar values of $Fr$ and $Re$. However, regular, thick accumulations of anchor ice formed in sub-reach 4 while anchor ice was rare in sub-reach 3. When anchor ice formed in sub-reach 3, it was sparse and low lying. A predictive model based on $Fr$, $Re$, and rate of heat loss would predict similar anchor ice formation in both sub-reaches. This is obviously not the case. Although $Fr$ and $Re$ are in the same range for both sub-reaches, the height of the bed roughness elements (expressed as $d/Y$) differ greatly.

The majority of anchor ice observed in the Laramie River reach formed in the relatively steep, gravel-bedded riffles (Table 2, sub-reaches 2 and 4). Large volumes of anchor ice also formed on the bedrock bottom of sub-reach 1 (Figure 1). On the coldest nights, relatively small amounts of low-lying anchor ice also formed on the sand bed of sub-reach 3 (Figure 1). A major factor controlling where anchor ice forms for a given set of meteorological conditions is bed material size. Unlike many previous studies, we observed anchor ice forming on a sand bed, particularly when the sand moved as dunes advancing over a forming anchor ice mass. The anchor ice observed on sand was never very thick, it never continuously covered more than a few square meters of the bed, and it was often buried beneath advancing sand bedforms. As such, it was very difficult to observe. Whenever anchor ice was observed on the sand of sub-reach 3, sub-reaches 2 and 4 were completely covered with a continuous layer of anchor ice up to 0.40 m thick. The anchor ice that formed on a sand bed had much less effect on the flow than did anchor ice dams formed in the gravel riffles between sand beds.

The amount of anchor ice that formed on the gravel bed riffles was impressive. Figures 2A & C show anchor ice accumulations 0.30 to 0.40 m thick, covering the entire gravel and pebble bed of sub-reach 4. The median particle diameter in this reach is 19 mm (Table 2); pebbles this size weigh approximately 7 g. In their flume experiments, Qu and Doering (2007) noted that anchor ice was more readily dislodged from smaller gravel compared to larger gravel. They conclude that the bonding force between small gravel and anchor ice was insufficient to hold anchor ice to the bed, and small masses of anchor ice are dragged away by hydrodynamic or buoyant forces. Our observations contradict this conclusion; the sub-reach’s gravel beds very effectively held thick masses of anchor ice to the bed. The difference between the flume and our Laramie River observations may be attributable to the difference in ice thermal growth observed in each setting. Qu and Doering saw evidence of thermal growth in their temperature time series, but did not identify increased ice crystal size in anchor ice masses. In contrast, we observed most anchor ice
accretion to be by in-situ thermal growth, as shown by the large ice crystals in Laramie River anchor ice masses. It is possible that in-situ thermal growth makes the anchor ice adhere more tenaciously to the bed of the Laramie River study reach. As noted above, we did not see complete encapsulation of bed particles (Figure 2D, E, &F), or growth of ice into the substrate, as reported by Stickler and Alfredsen (2005). In our reach, anchor ice grew almost entirely on the top surfaces of bed particles (Figure 2D). The differences in anchor ice formation we observed, along with observations described in prior studies, confirms that anchor ice formation depends on a reasonably extensive set of parameters like that given as Eq. (1).

8. Concluding Comments
Our field observations of anchor ice formation in a reach of the Laramie River, Wyoming yield important new insights. They reveal anchor ice to be more than an evolutionary form of frazil formation, as commonly assumed. Further, they show that anchor ice can be a major contributor to ice-cover formation in shallow rivers. A comprehensive set of parameters, like Eq. (1), is needed to adequately characterized anchor ice growth. Our observations have great relevance for the large set of channels similar to the Laramie River. For instance, our observations are very pertinent to the Platte River drainage, of which the Laramie River is a tributary.

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Freeze-up Study on the Lower Athabasca River (Alberta, Canada)

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The complex water-ice interactions that occur on a river during freeze-up are highly dependent on its unique planform characteristics, human influences such as river engineering works, and the ambient weather conditions that evolve from year-to-year. For this reason, it is often quite difficult to model freeze-up processes without a detailed understanding of the ice regime of a particular river and reach of interest. On the lower Athabasca River north of Fort McMurray, there are growing demands for water due to expansions in industrial development related to the oil sands, yet nothing is known about how the formation and presence of an ice cover will affect winter water availability as water use changes.

As a step towards addressing this knowledge gap, a combination of field study and modeling of the freeze-up regime of the lower Athabasca River began in the fall of 2006. The goal of the field program is to document the freeze-up regime and provide a framework for 1-D (and, ultimately 2-D) freeze-up modeling within the reach. This paper reports on the freeze-up field monitoring program, and presents preliminary results of the freeze-up modeling efforts undertaken based on data collected in the first two years of this study. Simulation results using the University of Alberta’s River1D river ice process model are presented and compared with various data collected in the field.
1. Introduction

Water supply is becoming an issue of increasing importance across Alberta, as a result of expanding demands due to population and industrial growth. This is particularly true for the Athabasca River at Fort McMurray, which faces significant increases in water demand due to expansion of the oil sands mining efforts, not to mention for human consumption as the population of Fort McMurray, AB continues to increase rapidly. Although extensive efforts have and are being undertaken by various organizations and government agencies to assess the potential implications of water demand on the lower Athabasca River, one significant outstanding question centers around the issue of river ice. The nature and extent of ice cover development on large rivers is highly dependent upon flow rates, depths and velocities; therefore, a key question arises as to whether substantive flow withdrawals will have a significant impact on the ice regime of the river. This question is particularly critical for the Athabasca River, since it is unregulated and natural winter flows tend to be quite low.

A major factor contributing to the past success of Albertan industries has been the relative abundance of fresh water resources compared to other parts of the world. Unfortunately, success and prosperity has lead to growth in demand for water resources in Alberta that will soon exceed the availability of water in some places. For users dependent on large unregulated rivers (such as the Athabasca River) for water supply, the winter low flow period represents the most critical time of year in terms of water shortage risk, as illustrated in Figure 1. On average, mean monthly flows in winter are in the order of 200 m$^3$/s, with mean daily flows as low as 75 m$^3$/s documented in recent years. Fall and winter flows for the two years of this study (2006/07 and 2007/08) are also presented in Figure 1 for comparison.

![Figure 1. Mean monthly flows on the Athabasca River at Fort McMurray, Alberta.](Data source: Water Survey of Canada, available record from 1957 to present)

In addition to the issue of winter water supply, potential changes in the winter ice regime could also have significant implications for water quality, as the winter low flow period is when effluent dilution capacity and oxygen replenishment are at a minimum along northern rivers. Open water areas are critical to oxygen replenishment, and in this context, potential changes to
the river’s ice regime could play an enormous role. Impacts to fish habitat will also be an issue; if ice formation processes are changed, then potentially significant, and detrimental, impacts on the winter habitat might result. As the effects of climate change begin to influence the ice regime of the Athabasca River, this might alleviate or exacerbate these other potential effects.

As a first step in addressing these unknowns, a three year study was initiated in 2006 to investigate the ice regime of the lower Athabasca River in the 80 km reach extending downstream of Fort McMurray to Bitumount (Figure 2). The specific objectives of this study are as follows:

- To conduct field studies of river ice processes on the Athabasca River in order to establish the baseline conditions describing the winter ice regime in this reach, and to obtain essential quantitative data for numerical ice process model calibration and validation.
- To calibrate and validate river ice process models (both 1-D and 2-D) for the lower Athabasca River.
- To apply these river ice process models to investigate the potential impacts of future flow withdrawals and climate change on the ice regime, and thus the future water supply, of the Athabasca River downstream of Fort McMurray.

This paper reports on the progress of this study to date, focusing in particular on the freeze-up monitoring program and the 1-D ice process model calibration and validation efforts.

2. Past Studies

Despite the pressing concerns regarding the winter water supply of the lower Athabasca River, little was known about the winter ice regime of the river in this reach prior to commencement of this research project. Hydrologic and hydraulic studies of the lower Athabasca River date back to the late 1970’s, but most focus on the potential environmental impacts from the oil sands mining efforts. For example, Doyle (1977) characterized the hydrology and hydraulics of the lower Athabasca River spanning approximately 180 km from Fort McMurray to Embarras. Data reports by the Alberta Research Council and other agencies, also provide an insight into the behavior of the river in this reach. Although no documentation of ice cover formation processes could be found in this archival material, they do provide information regarding historical accounts of ice jam related flooding events at some locations downstream of Fort McMurray.

Beltaos (1979) studied mixing characteristics of the Athabasca River under ice-covered conditions, spanning approximately 300 km from Fort McMurray to Lake Athabasca. As part of a research program to investigate mixing characteristics of rivers in Alberta, this study contained hydrometric information and geomorphic characteristics of the reach, as well as information regarding the thickness of the fully formed ice cover. However, no data pertaining to ice cover formation or deterioration processes was reported. Van Der Vinne (1993) also conducted winter low flow tracer dye studies in the reach of the Athabasca River spanning 80 km from Fort McMurray to Bitumount. As part of the Northern River Basins Study, this investigation focused on characterizing the cumulative effects of development on the water and aquatic environment of the study area. Field investigations were conducted for relatively low discharges, ranging from 81 to 188 m$^3$/s, and so are pertinent to this study. Valuable information regarding the hydraulics of the reach, including times-of-travel and the variation of Manning’s roughness were provided by their study.
Figure 2. Lower Athabasca River study reach.
Since 1998, the University of Alberta, in cooperation with Alberta Environment, has undertaken an extensive monitoring program of breakup processes in the reach upstream of Fort McMurray. Even though the focus has primarily been to document dynamic ice jam formation and release events in that area, monitoring of the reach downstream of Fort McMurray during the breakup period has also taken place. Kowalczyk Hutchison and Hicks (2007) report on these monitoring efforts, and also summarize and interpret historical data from Alberta Environment and Alberta Research Council breakup reports.

Since 1997, participants in the Regional Aquatics Monitoring Program (RAMP) have worked to conduct environmental monitoring programs aimed at assessing the ecological health of the lower Athabasca River (and nearby streams and lakes) in the vicinity of the oil sands projects north of Fort McMurray. Much of their work to date has involved data collection quantifying water quality, fish, and benthic invertebrates, but they have also conducted climate and hydrological data measurements at selected sites. Together with the Cumulative Environmental Management Association (CEMA), it was agreed that the winter low flow period represents a critical time in terms of stream ecology, especially as it relates to fish habitat. Thus, as part of their ongoing efforts, CEMA funded field survey programs documenting ice characteristics and channel bathymetry at selected sites on the Athabasca River downstream of Fort McMurray. Complementing this same CEMA effort, Katopodis and Ghamry (2005) of Fisheries and Oceans Canada, used this data to conduct 2-D hydraulic modeling to facilitate winter fish habitat assessment efforts at these sites for documented (known) ice conditions. These data and 2-D model input files have been provided to this study as part of an informal collaboration.

It is also known that a number of ice related hydrotechnical investigations have been conducted along the reach to facilitate the design of river water intakes for oil sands operations, as well as to facilitate the design of bridge crossings and ice bridges. Most of this information is proprietary and unavailable to researchers, but some of this data has been made available to this study as part of a data and information sharing effort.

3. Field Study

3.1 Reach Description

The ~80 km study reach spanning from Fort McMurray to Bitumount is illustrated in Figure 2; river stations, referenced in terms of km upstream of the mouth, are shown as well. This lower portion of the Athabasca River drains a total of 58,000 km², which includes the Clearwater River basin. Major tributaries along the reach include the Steepbank, Beaver, Muskeg and MacKay Rivers, which together drain approximately 8220 km² contributing ~4% to the total flow of the river in this reach (Doyle, 1977). Downstream of Fort McMurray, the Athabasca River flows north through lowlands and the Athabasca tar sands deposits, with an average slope of 0.13 m/km (Kellerhals et al., 1972). According to Conly et al. (2002) the river is laterally stable and deeply entrenched in its valley, flowing straight with occasional islands and sand bars of considerable size. Doyle (1977) states that the thalweg shifts within a year in much of the reach downstream of Fort McMurray. Bed material is predominantly sand, with local gravel over limestone, and Manning’s roughness values range between 0.018 and 0.030 for the entire reach at different flood frequencies (Kellerhals et al., 1972). Van der Vinne and Andres (1993) reports a Manning’s roughness of 0.017 for the entire reach under ice-covered conditions; however, they considered this value to be low due to difficulties in determining the under ice top width.
3.2 Freeze-up Monitoring Program and Observations

For this study, a freeze-up monitoring program was implemented during the fall of 2006 and continued in the fall of 2007. This included measurements of meteorological conditions (particularly air temperature), water temperature, water depth, and ice cover development. The Water Survey of Canada (WSC) operates a gauge in the reach, and this data was obtained as well (courtesy of Alberta Environment). Details of our monitoring program are provided below.

Ice Cover Development Observations

The development of the ice cover over the freeze-up period was documented from the air, through multiple observational flights, as well as from the ground using automated cameras at stations M288.1 and M216.7 in 2006, and M268.1 and M288.1 in 2007 (Figure 2). Figure 3 presents typical images obtained from these two sources in 2006. These were used to create ice development maps spanning the freeze-up period. Figure 4 illustrates examples of these maps for 2007. Ice concentrations were also measured from these photos using image analysis software.

![Figure 3: Examples of photographs taken to document freeze-up in the study reach.](a) remote camera (M288.1)  
(b) aerial observations

In general, it was found that freeze-up in the study reach is highly complex, involving multiple bridging points. Incoming pans tend to juxtapose upstream of these bridging points, with the upper portion of most of these sub-reaches tending to freezing thermally. Thermal ice development (border ice) also tends to dominate in the shallow side channels. No hummocky ice was documented in the study reach in 2007, but localized areas of hummocky ice were observed near the upstream end of the reach and near Bitumount in 2006. Otherwise, the ice cover development was quite consistent between the two years. Classification of ice formation processes was slightly hampered in both years by early snow cover obscuring the ice surface.

Meteorological Data

Environment Canada (EC) operates a weather monitoring station at the Fort McMurray Airport, providing all data relevant to river ice processes except solar insolation. Since 1999, the University of Alberta (UA) has operated a meteorological station at Fort McMurray, just upstream of the study reach (Figure 2) sampling on 30 minute intervals, and including
measurements of air temperature, wind speed, barometric pressure, rainfall and solar insolation. Since air temperature is the primary meteorological parameter of interest for the freeze-up period, an additional air temperature sensor was installed in the study reach in the fall of 2006, at river level near Bitumount (station M216.7 on Figure 2).

![Image of GIS ice maps](image)

**Figure 4.** GIS ice maps developed for November 16 (left) and 30 (right), 2007.

![Graph of mean daily air temperature](image)

**Figure 5.** Comparison of mean daily temperatures measured on top of valley wall at Fort McMurray (UA and EC Met) and at river level near Bitumount (M216.7) in 2006/07.
Figure 5 shows the mean daily air temperatures measured at these three sites for the 2006/07 ice season where it is seen that differences are minor and mostly evident for extreme cold conditions.

Figure 6 presents the accumulated degree days of freezing (ADD\(_F\)) at Fort McMurray (EC station at the airport) for the two years of this study to date, in comparison with the historical averages at the same station (1971-2000). Also shown are the dates of first ice and complete ice cover in the study reach for the two years of observations. It is possibly significant to note that there does appear to be some consistency between the two years for both first ice (ADD\(_F\) = 8.8 and 10.2°C-days, respectively) and the complete ice cover (ADD\(_F\) = 50.4 and 50.6°C-days, respectively).

![Figure 6](image-url)

**Figure 6.** Accumulated degree days of freezing in 2006 and 2007, as compared to historical.

**Water Temperature and Water Level Data**

Water temperature and water level (pressure) sensors were installed at four sites along the study reach in the fall of 2006, specifically at stations M288.1, 268.1, 245.6, and 216.7 (Figure 2). Figures 7 and 8 present examples of the data collected. As Figure 7 illustrates, it was found that water temperatures essentially cooled simultaneously at all four stations. Figure 8 provides the water level data obtained during freeze-up 2006, illustrating the stage-up associated with the ice front passing. An earlier water level rise at M268.1 likely reflects backwater effects due to bridging downstream.

**4. Freeze-up Modeling**

Using the data collected during freeze-up 2006 and 2007, a preliminary calibration and validation of the UA River1D ice process model (Andrishak and Hicks, 2008) could be attempted. Boundary conditions for the modeling efforts consisted of an inflow discharge condition at the upstream boundary (station M288.1) based on data from the nearby WSC gauge...
(Figure 2) just prior to freeze-up. Inflows during the freeze-up period were estimated by extrapolation of the pre freeze-up recession curve of the measured hydrograph.

**Figure 7.** Water temperatures measured during freeze-up 2006 (air temperature shown for reference purposes as well).

**Figure 8.** Water levels measured during freeze-up 2006.
Water temperatures at the inflow boundary (station M288.1) were based on field measurements of water temperature at this site. Inflow ice conditions, in terms of surface ice concentrations, were obtained based on analysis of photographs taken from the ground and air at M288.1.

**Water Temperature Modeling**

The River1D ice process model handles heat transfer processes using a linear heat transfer approximation. Solar radiation effects were neglected in the heat budget for this preliminary effort. Figure 9 shows the results obtained for the water cooling phase of freeze-up for 2006 and 2007, comparing modeled and measured water temperatures at station M268.1. The simulations for 2006 suggest generally reasonable agreement between modeled and measured water temperature, with little sensitivity of the results evident for values of the heat transfer coefficient between air and water \( H_{wa} \) ranging from 15 to 20 W/m\(^2\)/°C. The results for 2007 appear slightly more sensitive, with a value of \( H_{wa} = 10 \) W/m\(^2\)/°C seeming to be the most appropriate. However, overall it would appear that the quality of the water temperature boundary condition is far more significant than the ambient cooling effect within the simulation. Note that the gap in the simulation results for 2007 are due to a temporary equipment malfunction at station M288.1, which resulted in missing inflow water temperature boundary condition data for this period.

**Ice Concentration Modeling**

As mentioned above, detailed photographic analyses enabled the determination of surface ice concentrations as a function of space and time during freeze-up. Figure 10 presents an example of the simulation results obtained for freeze-up 2007, in comparison with the observed ice concentrations, where it is seen that the model generally shows the correct trend. Further analyses are planned to test the sensitivity of the model results to the inflow ice concentrations, and to try two independent methods for determining ice surface concentration from the photos.
5. Summary and Conclusions

This paper presents preliminary results of an ongoing study investigating the winter ice regime of the lower Athabasca River from Fort McMurray to Bitumount, AB. In particular, the focus here is on the freeze-up processes documented, and preliminary 1-D ice process modeling efforts.

![Graph](image)

**Figure 10.** Comparison of measured and modeled surface ice concentrations (2007).

At the onset of freeze-up water temperatures were observed to cool uniformly over the entire study reach. As the Athabasca River in this reach is characterized by a relatively flat slope, sand bed and numerous bars and islands, ice cover development in the shallow back channels is

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dominated by thermal (border) ice growth, while the more active flow zones predominately experience a juxtaposed ice cover. It was observed that there are multiple bridging sites in the reach, and therefore multiple ice fronts progress upstream during freeze-up. A localized section of hummocky ice was observed near Bitumount in 2006. There appear to be some consistency between the two years for both first ice ($ADD_F \sim 9$ to $10 \, ^\circ C$-days, respectively) and the complete ice cover ($ADD_F \sim 50 \, ^\circ C$-days). Preliminary analyses with the University of Alberta’s River1D ice process model indicates that inflow boundary conditions describing water temperature and ice influx play a dominant role in influencing simulation of freeze-up processes.

Further research is underway to obtain one more year of freeze-up observations. In addition, model simulations are underway to further assess the sensitivity of model predictions to inflow boundary conditions. Efforts are also underway to incorporate water cooling and ice cover formation processes in the University of Alberta’s River2D hydrodynamic model.

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References


An Experimental Study of Wave Induced Ice Production

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ABSTRACT

In this paper, we present a laboratory study that measures the ice production in a wave field. This ice production is directly related to the heat transfer rate between the freezing water and the much colder air above it. We conducted the experiment at the Hamburgische Schiffbau-Versuchsanstalt GmbH (HSVA) in Hamburg Germany. Standing waves were produced in a cold room to create ice in a small tank. A reference tank with calm water was located next to the wave tank for comparison. Four cases with different wave frequency and amplitude were used to measure the ice production. It was found that the total ice production is enhanced under wave conditions. However, because of the range of wave conditions that could be generated in this small tank is limited, to quantify the wave effect on ice production is not possible with the small amount of data. This study demonstrated a systematic approach to determine possible wave enhancement for ice production in a controlled environment.

1. Introduction

Arctic sea ice is melting faster than any of the model predictions [Stroeve et al. 2007]. As of 2006, the general belief was by 2050 the Arctic will be ice-free in the summer. But the trend from recent data suggests that such event can happen as early as 2030 [NSIDC new release, Oct. 2007]. Fig. 1 from National Snow and Ice Data Center (NSIDC) shows the 2007 late September ice cover in the Arctic Basin. The magenta line represents the median September extent based on
data from 1979 to 2000. The distance from the coast of Alaska to the nearest ice edge in this image is estimated to be over 500km. Long stretch of open water that becomes available when ice cover shrinks provides the wind fetch necessary to generate unprecedented wave conditions in the Arctic. Up to the present, the majority of Arctic sea ice is formed under calm conditions. The opening up of the Arctic Basin implies future ice formation may be strongly influenced by the presence of waves. With the reduction of ice, exploration and navigation in the arctic region will increase. Along with the opening of the Arctic Basin there will be more advancing/retreating ice events to provide opportunities for new ice formation accompanied by salt water formation. The effect of these projected increasing events of new ice formation over open water will no doubt affect ocean circulation and primary production. To understand the global implications of arctic ice reduction and to manage the local demand for oil and gas, mineral, and transportation development require a drastically accelerated modeling capability.

In this paper we report a laboratory study carried out at the Hamburgische Schiffbau-Versuchsanstalt, GmbH (HSVA). A small wave tank in a cold room was utilized to generate standing waves, where environmental parameters such as air temperature were controlled. In each test the wave tank began with an ice-free condition to simulate an open ocean. Throughout each test the wave paddle was kept constant, with a fixed frequency and amplitude. The ice cover thicknesses at the end of approximately three hours were measured to determine the total ice production. For each test, a calm water tank was placed next to the wave tank under the same environmental condition. The ice produced in this reference tank provided the necessary comparison to show wave enhancement.

2. The Experiments

Four tests were carried out at HSVA, Germany on October 28-31, 2007. Each test consisted of two tanks, one with calm water and one with waves. Both tanks were insulated on all sides and the bottom, with only the top surface exposed to air. Stationary wave was generated using a cylindrical rod in the middle of the wave tank. This cylindrical rod was force to move sinusoidally in and out of the water surface. The initial salinity of each tank was measured under the open water condition. The frequency and average amplitude along the standing wave tank was determined at the beginning of the test. Each test lasted about 3 hours. At the end of the test, a 40cm by 40cm area of ice cover from each tank was collected and drained. The thickness of the ice cover from each tank was recorded. The salinity of the tank water in both tanks and drained ice samples were measured. The drained ice samples were then melted in room temperature and

Figure 1. Sea ice extent for September 2007. (source: NSIDC web site.)
the volume converted to solid ice. A thermister string with four sensors located at 5cm and 11cm above and 5cm and 8cm below the mean water level was installed in the wave tank. The wave frequency and amplitude were obtained with a single ultrasonic sensor at locations 5cm, 7cm, 11cm, 17cm, 30cm, 35cm, 47cm and 56cm from the side of the tank. Figure 2 shows the actual small tank and the dimensions of both tanks.

![Figure 2. The standing wave tank with the calm water reference tank in the background. The ultrasonic sensor is shown to be moved along a track above the wave surface. The recorded time series is later processed to obtain the frequency and amplitude at each location. The depth of the wave tank is 39cm and 18 cm for the calm water tank.](image)

3. The Data

The wave data were obtained by an ultrasonic sensor (ToughSonic-TS-30S1). The sampling frequency at each location was 125Hz and tracked for 18 seconds at each of the locations along the straight line. A sample of this data and the power spectrum are given in Figure 3. The amplitude is calculated using the Parseval’s theorem.
The salinity data was obtained using a conductivity meter (HANNA H1 8733), and the UNESCO formula (UNESCO 1981) to covert the conductivities (ms) to salinity (psu). The ice cover thickness was first measured using a mesh scoop as shown in Figure 4, then melted to convert to equivalent solid ice thickness. This way the porosity of the ice cover may be obtained in addition to ice production. The porosity is calculated as the ratio \(\frac{([\text{ice cover in-situ thickness}] - [\text{ice cover converted thickness}])}{[\text{ice cover converted thickness}]}\).

The wave condition, salinity data from water and ice, ice sample thickness and porosity as measured at the end of the test, and converted solid ice thickness for each test are summarized in Table 1.

![Figure 3. A sample of the data and its power spectrum](image)

![Figure 4. The wire scoop used to measure the ice thickness](image)
Table 1. Wave and ice data.

<table>
<thead>
<tr>
<th>Test</th>
<th>Duration (hr:min)</th>
<th>Wave freq/amp (Hz/cm)</th>
<th>Salinity (psu)</th>
<th>Ice cover thickness (mm)</th>
<th>Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Water Before</td>
<td>After</td>
<td>In-situ</td>
</tr>
<tr>
<td>Test A</td>
<td>3:50</td>
<td>1.53/0.42</td>
<td>N/A</td>
<td>N/A</td>
<td>20</td>
</tr>
<tr>
<td>Test B</td>
<td>3:00</td>
<td>2.14/0.45</td>
<td>33.20</td>
<td>33.75</td>
<td>14</td>
</tr>
<tr>
<td>Test C</td>
<td>3:20</td>
<td>2.87/0.22</td>
<td>36.07</td>
<td>36.36</td>
<td>13</td>
</tr>
<tr>
<td>Test D</td>
<td>3:00</td>
<td>1.22/0.14</td>
<td>35.63</td>
<td>36.07</td>
<td>11</td>
</tr>
</tbody>
</table>

The temperature record for each test from the start of the test is given in Figure 5. This temperature data is needed for the ice production calculations.

**Figure 5.** The temperature records for each test. The positions of the sensors are: Sensor A (11cm); Sensor B (5cm); Sensor C (-5cm); Sensor D (-8cm) all relative to the mean water level.
4. Results and Discussion

In this section, the derivation of the equations used to estimate the ice growth rate is presented. The method to estimate the thermodynamic ice growth rate is based on well-established theory (Ashton, 1980) and is adapted to the laboratory environment.

The ice growth is the result of heat balances at both the top and the bottom surfaces of the ice cover. In these tests, the water was always at the freezing point, there was no melting from the bottom, no precipitation, no short wave radiation, and negligible long wave radiation. Under these conditions, the top surface of the ice cover, the total heat loss rate $Q_{ia}$ can be expressed in terms of a simple heat transfer coefficient, the difference between air temperature and surface temperature and the area of the top surface. For an area of unit length in the direction of wave tank, the total heat loss rate can be expressed as

$$Q_{ia} = h_{ia}(T_s - T_a)W$$  \[1\]

Where, $h_{ia}$ is the ice-air heat transfer coefficient, $T_s$ is the temperature of the top surface of the ice cover, $T_a$ is the air temperature, and $W$ is the width of the wave tank.

The heat balance at the bottom of ice is expressed as Eq. [2]

$$\rho_i L_i W \frac{dh}{dt} = Q_c$$  \[2\]

Where, $Q_c$ is the conductive heat flux through the top surface, $\rho_i$ is the density of ice, $L_i$ is the latent heat of fusion of water, $h$ is the ice cover thickness.

For thin ice we may assume a linear temperature distribution in the ice cover, the conductive heat transfer in ice can be described by Eq. [3]

$$Q_c = k_i(T_f - T_s)W / h$$  \[3\]

In which $k_i$ is the thermal conductivity of ice, $T_f$ the freezing point of water. Assuming there is no ice growth on the top surface of the ice, the boundary condition at the ice-air interface can be expressed as

$$Q_{ia} = Q_c$$  \[4\]

Combining Eq. [1] through Eq. [4], we get
\[
\rho_i L_i \frac{dh}{dt} = \left[ \frac{T_f - T_a}{h + \frac{1}{k_i/h_{ia}}} \right]
\]  \[5\]

Eq. [5] can be further simplified as follows. The typical \( h_{ia} \) value is about 20-40 \( W/\text{m}^2\text{C} \) and the thermal conductivity of ice \( k_i \) is 2.24 \( W/\text{m}^\circ\text{C} \), hence \( h/k_i << 1/h_{ia} \). when the ice cover is only a few centimeters thick. Eq. [5] can be approximated by

\[
\rho_i L_i \frac{dh}{dt} = h_{ia}(T_f - T_a)
\]  \[6\]

Assuming \( h_{ia} \) is independent of time, integrating Eq. [6], we have

\[
\rho_i L_i h = h_{ia}S
\]  \[7\]

Here,

\[
S = \int_{t_0}^{t} (T_f - T_a)dt
\]  \[8\]

Since the equivalent solid ice thickness \( h \) was obtained for each test, and the air and water temperature were also obtained, Eq. [7] may be used to calculate the heat transfer coefficient \( h_{ia} \). We use this heat transfer coefficient to represent the efficiency of ice growth rate. The results are given in Table 2.

**Table 2. Estimation of ice-air heat transfer coefficient \( h_{ia} \) for calm and wave cases**

<table>
<thead>
<tr>
<th>Case</th>
<th>Equivalent Solid Ice Thickness (cm)</th>
<th>Experiment Duration (s)</th>
<th>( S (\circ\text{Cs}) )</th>
<th>( h_{ia} (W/\text{m}^2\text{C}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calm</td>
<td>Wave</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test A</td>
<td>1.18</td>
<td>1.25</td>
<td>13800</td>
<td>1.22E+5</td>
</tr>
<tr>
<td>Test B</td>
<td>1.05</td>
<td>1.20</td>
<td>10800</td>
<td>0.96E+5</td>
</tr>
<tr>
<td>Test C</td>
<td>0.88</td>
<td>0.98</td>
<td>12000</td>
<td>1.09E+5</td>
</tr>
<tr>
<td>Test D</td>
<td>0.85</td>
<td>0.89</td>
<td>10800</td>
<td>0.85E+5</td>
</tr>
</tbody>
</table>

The mean value of \( h_{ia} \) for calm water is 29.54 \( W/\text{m}^2\text{C} \) and the standard deviation equals to 3.24 \( W/\text{m}^2\text{C} \). For the wave case they are 32.31 \( W/\text{m}^2\text{C} \) and 4.01 \( W/\text{m}^2\text{C} \) respectively. Thus a slightly elevated heat transfer is produced by the waves. The higher standard deviation indicates different waves may have different enhancement effect. However with the limited data this point cannot be verified.
5. Conclusions

In this study we showed a possible way to determine wave enhanced ice production. This process is very difficult to study in the field because the uncontrolled environmental conditions such as wind, solar energy input, and ocean current. However, due to the opening of Arctic Ocean, new parameterization for sub-scale processes such as the one presented here is increasingly important. More laboratory studies are thus recommended to better quantify this process.

Acknowledgments

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References

Ice Platelets Observed in Saroma-ko Lagoon, Hokkaido, Japan

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Abstract
Saroma-ko Lagoon, located on the northeast coast of Hokkaido, Japan, is a semi-enclosed embayment connected with two openings to the Sea of Okhotsk, resulting in a salinity of about 32 psu. Freshwater input to the lagoon from a river causes a gradual decrease in water salinity from the opening towards inshore. The lagoon surface is covered with sea ice during the winter from late January/early February to early April with significant interannual variability. The freshwater input to the lagoon allows for studies of ice formation along a strong salinity gradient. In a number of field campaigns over several years, ice platelets attached to the bottom of the ice cover were found almost every year. The ice platelets’ morphology, size and crystallographic orientation were quite similar to those previously studied in the Antarctic and Arctic Oceans. The characteristics of the ice platelets are described and then possible mechanisms of their formation and growth are discussed on the basis of temperature, salinity, current and oxygen isotopic composition data of the ice and the underlying water.
1. Introduction

Saroma-ko Lagoon, located on the northeast coast of Hokkaido, is the third largest water body in Japan, with an area of about 150 km² and a maximum depth of about 20 m (Figure 1). Exchange of lagoon water with the Sea of Okhotsk is mainly forced by tidal currents through two inlets (Shirasawa et al., 1993), resulting in a salinity of about 32 psu (Practical Salinity Unit = 1/1000) throughout the northern part of the lagoon, i.e., similar to that of the Okhotsk Sea. Freshwater input from a river causes a gradual decrease in water salinity from the inlet towards inshore. The lagoon surface is covered with sea ice during winter from late January/early February to early April with large interannual variability in onset and duration of the ice cover. The ice thickness increases gradually to reach between 30 and 80 cm at the peak of the season. The freshwater inflow provides for good opportunities to study ice growth and ice-ocean interaction along a strong salinity gradient.

From a number of field studies in Saroma-ko Lagoon over several years, it was found that three different processes contributed to thickening of the ice cover: upward growth by formation of surface granular snow ice due to surface flooding, downward congelation ice growth leading to columnar sea ice textures, and downward growth through formation of frazil ice of granular texture (Kawamura et al., 2004). Stratigraphic studies of the ice cover established that a large amount of snow cover contributed to sea ice growth, exceeding that of ordinary columnar ice. In addition, ice platelets were found underneath the ice at the inshore site near the river mouth almost every winter.

The ice platelets exhibited crystal morphologies similar to those previously observed in the Antarctic and Arctic Oceans, such as large single-crystal ice platelets collected by Dieckmann et al. (1986) at 250 m depth off the Filchner Ice Shelf, Antarctica. Another type of platelet ice incorporated into the ice cover was observed in the Weddell Sea (Lange, 1988; Eicken and Lange, 1989; Lange et al., 1989) and McMurdo Sound (Jeffries et al., 1993; Smith et al., 2001, Leonard et al., 2006), Antarctica, as well as in the Arctic Ocean (Eicken, 1994; Jeffries et al., 1995).

This paper describes the characteristics of ice platelets observed in Saroma-ko Lagoon and discusses possible formation and growth mechanisms.

Figure 1. Location map of Saroma-ko Lagoon (a) and observation sites in the lagoon (b).
2. Observations

In this study, we report observations carried out in February-March of 2004 and 2006. We established five observation sites along a transect between the mouth of the Saromabetsu River, the largest river entering Saroma-ko Lagoon, and the opening in the eastern part of the lagoon (Figure 1). The influence of river water extended to 2 to 3 km from the river mouth, i.e., at Sites 1 and 2. The water depth was 1-2 m at Site 1, deepened sharply to about 10 m between Sites 1 and 2 and gradually increased to 12 m at Site 5.

A 3-dimensional electromagnetic current meter (Model ACM32M, Alec Electronics Co., Japan) mounted with conductivity and temperature sensors was moored under the ice at the middle of the water column during the period of the experiment at Site 2. The accuracies of the conductivity and temperature sensors are 0.05 S/m and 0.05°C, respectively. A CTD profiler (Seacat Profiler, Model SBE 19, SeaBird Inc., USA) was deployed to obtain conductivity and temperature profile at each site. The accuracies of conductivity and temperature sensors are 0.001 S/m and 0.01°C, respectively.

Sea ice cores and/or blocks were collected at Sites 1 to 5 to analyze ice stratigraphy and crystal structure. In the cold laboratory, the ice samples were split lengthwise to obtain vertical thick and thin sections along the entire length of the core or block. The thick and thin sections were photographed in plain and polarized light, respectively, to assess the ice stratigraphy and crystal structure. The samples were also cut horizontally to acquire horizontal sections at approximately 50 mm intervals. The cut pieces were then melted, and the chlorinity of melted ice, snow and seawater samples was determined by the titration method (Model SAT-210, Toa Electronics, Japan). The chlorinity was converted to the salinity according to Bennett (1976) with an accuracy of 0.1 psu. The samples were also analyzed for oxygen isotopic compositions (δ¹⁸O values) using a standard mass spectrometer (Finigan MAT DELTA Plus) technique with an accuracy of 0.05 ‰.

3. Results

Dendritic ice crystals with a peculiar form were observed in the inshore areas at Sites 1 and 2 almost every winter, while such crystals were never found at Sites 3 to 5 further towards the mouth of the lagoon. Two types of ice crystals were found: loose masses underlying the ice cover and ice platelets protruding downwards from the solid ice.

3.1. Underlying ice platelets

Loose masses of underlying ice platelets were observed at Site 2 in February 2004. After extracting an ice core or block large numbers of dendritic ice platelets floated up to the water surface in the hole from underneath the ice. Clumps of ice platelets appeared to be attached

![Figure 2.](image)

Figure 2. Photographs taken (a) in scattered light and (b) between crossed polarizers of underlying ice platelets collected at Site 2 on 17 February 2004. The minimum increment in the scale bar is 1 mm in this and all other figures.
loosely to the bottom of the sea ice. The surface of the ice platelets was corrugated and rough, and their size was approximately 5-10 cm in diameter and 1-2 mm in thickness (Figure 2a). The platelets were inspected between crossed polarizers (Figure 2b), establishing that they consisted of single crystals with the crystallographic basal plane parallel to the plate itself. Lewis and Weeks (1971) described ‘underwater’ ice consisting of lamellar plates with a size of 2-15 cm and a thickness of 1-2 mm. Dieckmann et al. (1986) observed large single-crystal ice platelets with an average diameter of about 2 cm and a thickness of 0.5 mm, at 250 m depth in the vicinity of the Filchner Ice Shelf, Antarctica. The size and morphology of the platelets observed at Saroma-ko Lagoon is similar to those described by Lewis and Weeks (1971), and slightly larger than those of Dieckmann et al. (1986).

3.2. Protruding ice platelets

Ice platelets protruding downwards from the ice cover were observed at the bottom of ice blocks at Site 2 in February 2006 (Figure 3). Some platelets remained without damage to their delicate structure, indicating rows of sub-parallel ice lamellae up to 10 cm in size, spaced several cm apart. The platelets protruded vertically from the bottom exhibited parallel edges at the same water depth roughly 10 cm underneath the ice bottom (Figure 3). Along this bottom edge, platelets exhibited a rough dendritic interface.

Figure 4 shows photographs of an ice lamellae removed from the ice bottom. These photographs show small-scale, irregular patterns on the surface of the

Figure 3. Photograph of bottom of ice block, about 40 cm wide, extracted at Site 2 on 18 February 2006.

Figure 4. Photographs of ice platelet broken off from the bottom of the ice sampled at Site 2 on 18 February 2006, taken under (a) plain and (b) cross-polarized light.
crystals which are most likely artifacts. The plate was shown to be a single crystal whose c-axis was perpendicular to the plate itself. In general, the protruding ice lamellae exhibited characteristics very similar to those of the underlying loose ice platelets described above.

4. Discussion

4.1. Characteristics, growth and incorporation of ice platelets

The salinity and $\delta^{18}$O of the ice platelets, the bottom of the ice and the water just under the ice for the two different types are given in Table 1. The underlying and protruding ice platelets were collected at Site 2 on 17 February 2004 and 18 February 2006, respectively. The values of salinity and $\delta^{18}$O of the ice platelets for both types were similar to those of the solid ice bottom layers. The ice platelets would have been incorporated into the ice body at the time of the ice in growing. The salinity of the ice platelets was lower than that of the under-ice water, and was likely overestimated due to drops or films of water adhering to the platelets during sampling. The $\delta^{18}$O of the ice platelets was about 3‰ higher (more positive) than that of the under-ice water. The deviation of about 3‰ in $\delta^{18}$O is representative equilibrium isotopic fractionation in the ice-water system as determined experimentally by O’Neil (1968) and Lehmann and Siegenthaler (1991), suggesting formation from the same water mass.

Table 1. Salinity and $\delta^{18}$O values of ice platelets together with bottom ice and water just under the ice for two types of ice platelets. The values on the left and right parts represent the underlying and protruding ice platelets, respectively.

<table>
<thead>
<tr>
<th></th>
<th>underlying</th>
<th>protruding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>salinity (psu)</td>
<td>$\delta^{18}$O (%)</td>
</tr>
<tr>
<td>ice platelets</td>
<td>0.69</td>
<td>-9.00</td>
</tr>
<tr>
<td>bottom ice</td>
<td>1.18</td>
<td>-8.38</td>
</tr>
<tr>
<td>unde-ice water</td>
<td>17.09</td>
<td>-11.21</td>
</tr>
</tbody>
</table>

Figure 5 shows vertical and horizontal thin sections of the core sample collected at Site 2 in February 2004. The vertical section shows that the sample is composed of granular and columnar ice in the upper and lower part, respectively (Figure 5a). The lowermost layers exhibited convoluted and ragged-edged crystals, which are also apparent in the horizontal section (Figure 5b), and which have been shown to be typical of platelet ice (Eicken and Lange, 1989). Jeffries et al. (1995) found moderate quantities of platelet ice in the Arctic Ocean, with similar structural characteristics, indicating that loose accumulations of ice crystals contributed to the solid ice cover in the form of platelet ice.

Figure 5. Photographs of (a) vertical and (b) horizontal thin sections taken between crossed polarizers of the sample, 24 cm in thickness, collected at Site 2 on 17 February 2004. The horizontal sections represent depths of 12.5 (upper) and 23.5 (lower) cm.
Figure 6 shows a vertical thin section photograph of the solid ice, i.e., above the platelets protruding from the ice block, collected in February 2006. The ice is composed of mostly typical columnar ice with exception of the very bottom of disordered ragged-edged crystals oriented obliquely to the direction of growth. This disordered structure was most likely caused by incorporation of misaligned ice platelets into the solid ice (see Figure 3).

4.2. Supercooling

Jeffries et al. (1993) found platelet ice in Antarctic landfast sea ice and indicated that platelet ice commonly forms from aggregates of platelets, either underlying the ice cover or incorporated into congelation ice. Leonard et al. (2006) found that platelet ice consisted of both ice crystals that had nucleated in the ocean and grew at depth or formed in direct attachment to the solid ice cover on the ice-water interface. Jeffries et al. (1995) found platelet ice in the Arctic Ocean and indicated that it most likely originated in the form of underwater ice grown during the summer melt season from supercooled water as described by Eicken (1994). Antarctic ice platelets and platelet ice have also been linked to the presence of supercooled water, mostly originating from interaction of water masses with floating ice shelves (Leonard et al., 2006). Ice platelets observed in this study in Saroma-ko Lagoon also most likely originated from supercooled water under the ice, due to heat and salt exchange between river water and more saline water at the sites sampled near the river mouth.

Shown in Figure 7 are time series of under-ice water temperature, salinity, and current direction and speed obtained at Site 2 during the period from 17 February to 9 March, 2006. No data were obtained prior to the sampling period because the instrument was only deployed during the time of first sampling. The sea water salinity at 0.5 m depth during this period was recorded to be about 31.5 psu. The corresponding freezing point of the water was calculated at -1.73 C. Because the actual temperature at that time was at a minimum -1.5 C, the water at depth was not supercooled. However, the moored sensors were deployed at a depth of 0.5 m, just below the ice bottom, i.e., not within the layer that harbored the ice platelets.

However, we have also obtained CTD data at every site on 16 February, which provide temperature and salinity profiles. The CTD data at Site 2 indicate that the salinity at a depth of 0.4 m, i.e., just a few cm below the ice bottom, was 14.4 psu with a temperature of -0.88 C. The water at this depth was supercooled, with a temperature significantly below the freezing point of -0.79 C. Also, the CTD measurements indicate that the 10 cm or so of water underneath the solid ice were strongly influenced by the river plume, in contrast with the water at greater depth which was roughly of ocean salinity. During the next observation on 9 March 2006, no supercooling was observed throughout the depth of the water column, as water temperatures had increased gradually above the freezing temperature as shown in Figure 7, resulting in elimination of the existing ice platelets. Furthermore, ice platelets were never found at the offshore sites (Sites 3-5)
as described above, where the water column below the ice was not recorded to be supercooled at any time in the CTD data.

These observations indicate that growth conditions for ice platelets are principally similar to those in the Antarctic and Arctic where platelet ice was reported to form in supercooled water masses underneath a solid ice cover (i.e., different from frazil ice formation in the open ocean). However, the degree of supercooling is somewhat larger, most likely due to continuous advection of freshwater with heat extracted at its base as a result of double-diffusive exchange processes (Martin and Kauffman, 1974). Assuming an attached platelet width of 2 mm and a mean platelet spacing of 5 cm in a layer thickness of 10 cm underneath the ice, then the amount of supercooling (0.09 K) observed would only be able to explain a few percent of the actual volume of protruding ice platelets grown if the water underneath the ice were stagnant. However, it is highly likely that the growth of ice platelets is in steady state with a heat flux sustained through double diffusion at the interface between freshwater (replenished through river flow) and seawater (replaced through tidal flow). This is supported by the sharp horizontal edge of the ice platelets which appears to conform with the depth of the supercooled layer underneath the ice.

4.3 Orientation of ice platelets
Platelets were subparallel and tilted by about 10 to 30 degrees from a line running normal to the observation transect, pointing ENE. The direction of the platelets was identical to that of the near-constant current direction of roughly ENE (Figure 7), which suggests that the platelets were in approximate alignment

**Figure 7.** Time series data temperature, salinity, and current direction and speed recorded at a depth of 0.5 m below the ice bottom at Site 2 during the period from February 17 to March 9, 2006.
with the prevailing current direction. Since currents in the lagoon are fairly stable, it is reasonable to assume that such co-alignment persisted throughout the time period of platelet growth.

Weeks and Gow (1978) demonstrated parallel alignment of sea ice crystals and ice lamellae in a steady current. They found that ice lamellae were preferentially oriented normal to the current, which is in contrast with the findings of the present study. This suggests that other mechanisms than those driving alignment of lamellae protruding merely millimeters at most into the supercooled water underneath an ordinary ice sheet are governing growth of the lamellae found here extending several centimeters below the ice bottom. While larger crystals may have been dislodged by the current if oriented normal to the flow, the mechanism of platelet crystal alignment observed in Saroma-ko Lagoon is not understood at present. However, further work may also shed light on crystal alignment in ordinary columnar sea ice for which open questions remain.

Figure 8 shows a horizontal thin section photograph of the ice at a depth 2 cm above the bottom of the solid ice. The ice texture includes long, thin needle-like crystal outlines in the ice matrix. Closer inspection shows that these crystals roughly run from upper right corner to lower left corner. Therefore it is likely that these crystals correspond to the ones appearing as large protruding ice lamellae below the solid ice.

5. Summary and conclusion
In a number of sea-ice field projects in Saroma-ko Lagoon, two types of ice platelets were found beneath the solid ice cover at the inshore sites almost every winter over several years of observations. One type consisted of aggregations of platelets attached loosely to the bottom of the sea ice or floating freely up in bore holes. The other type consisted of lamellae protruding several cm vertically down from the ice bottom. In appearance, size and crystallographic orientation, both types of ice lamellae were quite similar and strongly resembled those of previous studies which found underwater ice platelets in the Antarctic and Arctic Oceans. The platelets were found exclusively at inshore sites near a river mouth. From the temperature and salinity data, supercooling was shown to exist in the uppermost decimeter of the water column just under the ice bottom, suggesting that platelet occurrence was linked to supercooling associated with double-diffusion at the interface between fresh and saline water. It remains to be determined what processes drive the formation of aligned attached platelets as opposed to loose aggregations of crystals.
Acknowledgments
We wish to thank the personnel of the Saroma Research Center of Aquaculture in Sakaeura for their kind offer to use their facilities.

References


Freeze-up processes, rivers and oceans, river ice-structure interaction
Fundamental Study about the Formation Process of the Initial Pancake Ice

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ABSTRACT
Frazil-ice crystals which appear in polar regions are affected by wave conditions and temperature, then develop into initial discoid pancake ice. Sometimes, this initial pancake ice may be called “silver dollar”. Initial pancakes grow in size with rafting, collisions and freezing between pancakes and form larger ones. So far the study about mechanisms of formation process from initial pancakes to larger ones and the maximum size of pancakes has been done. However, the formation process from frazils to initial pancakes, the conditions and the size have not been made clear yet. In this study we concentrate on the frazil-initial pancakes stage, and discuss the relation between the wave conditions and initial pancakes formation. We suppose that the movement of frazil particles on the water surface follows a fluid movement, and use the linear theory of the wave motion. When we calculate the wave field, we can see that waves compress the distance between frazil particles and acceleration acting on particles is the biggest at wave peak. We assume that surface tension is acting on particles further at wave peak, and try to describe the relation between the wave conditions and formation of initial pancakes from balance of the power by surface tension to draw to particles and the power by the acceleration to pull apart.
INTRODUCTION
From old research, it is pointed out that frazils become pancakes in response to the action of a wave. In the state where there is no wave, frazils grow up to be an ice sheet instead of pancakes. Moreover, it is also observed that formation of pancakes is influenced by the wave conditions (period, wave height, etc.) in the HSVA experiment. In the wave with shorter wavelength or with higher wave height, pancakes are not formed but only forming of frazils is observed. It is clear from this that there is relation between wave conditions and forming of pancake ice. Since the area of ice sea is changing with the influences of global warming in recent years, the influence of wind and wave to ice has also been changing. However, since the relation is not solved, it aims at clarifying relation between wave conditions and formation of pancakes focusing on the frazil-initial pancakes stage in this paper.

It is thought that it needs that particles touch and not to separate as conditions since formation of a pancake ice begins from the freezing frazil particles. Then, the situation of movement of frazil ice is expressed using the linear theory of the wave motion. And we assume surface tension as power for frazils not to separate. It is introduction first, we performed observation by easy water vein experiment (in Iwate University), and model construction based on it. Details will be discussed below.

![Figure 1. Picture of initial pancake ice (silver dollar).](image)

EXPERIMENT
If it generates, frazil ice will appear on the water surface and will form a thin layer. We experimented using the wave tank in Iwate University in order to observe how the object which appears on the water surface moves in response to the action of a wave. A schematic of the layout of the tank can be seen below.

![Figure 2. Schematic of the layout of the tank.](image)
The polypropylene particles assumed as frazils were uniformly floated on the water surface, and the wave was occurred. The diameter of polypropylene particles is about 4.5mm. The length of the x direction was about 3m (arbitrary). When the wave was generated in this state, we can observe that particles approach each other at wave peak and separate at wave trough. Fig. 3 shows this situation typically. This situation can be observed notably when the wave period is shorter and the wave height is higher, since movement of particles is more intense. That is, the situation that the distance between particles which approaches or separates was large was seen.

Another feature other than this result was observed. Some polypropylene particles adhered with adjoining particles, and didn’t separate at wave trough too. From this, it can expect that another external force which pulls particles other than wave action is working to particles. We assumed that this external force is surface tension, and decided to take in to the model described below.

MODEL SETUP
First, it assumes that frazil ice follows the fluid movement completely, we describe the locus of movement of frazil ice using the linear theory of the wave motion. We use deep sea wave condition. Calculating points \((x_{0n})\) are arranged at equal intervals \((\Delta x)\) on an x-axis, and if we calculate the locus of the particles which move considering this point as a center by equation (1), a circular orbit as shown in figure4 will be drawn.

\[
\begin{align*}
x_n &= a \cos(kx_{0n} - \omega t) + x_{0n} \\
z_n &= a \sin(kx_{0n} - \omega t)
\end{align*}
\]

Where \(x_n\) is the x-coordinate of the n-th particles, \(z_n\) is the z-coordinate of the n-th particles, \(a\) is the amplitude, \(k\) is the wavenumber, \(\omega\) is the angular frequency, \(t\) is the time.

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Figure 3. Schematic of relation of particles position at wave field.

Figure 4. A circular orbit considering calculating point as a center.
The ratio of $L$ to $\Delta x$ is expressed with the following equation when distance between two adjacent particles is set to $L$.

$$\frac{L}{\Delta x} = \sqrt{1 + (ka)^2 - 2ka \sin(kx_0 - \omega t)}$$

[2]

**Figure 5.** The change by the time of $L/\Delta x$.

Fig. 5 expresses the change by the time of $L/\Delta x$. This figure indicates that two particles which moves considering two points which $\Delta x$ separated on the x-axis as a center do not move, always maintaining a fixed distance, but making the distance elastic. If we put in another way, particles approach each other at wave peak and separate at wave trough. So when this situation is expressed with a schematic, it is in agreement with Fig. 3. It will be said that this equation can express movement of frazil particles well.

To the next, we consider the external force which works to frazils. In the experiment in HSVA, frazils gathering first and making about ten thin lines in the one wave length was observed. Also in the water vein experiment which floated polypropylene particles in Iwate University, signs that they did not get separate completely and some adhered were seen as above-mentioned. It is thought that the objects which float on the water surface become stable in the state where it gathered as this reason. And it can be expected whether power “surface tension” which pulls each particles together is working as power for gathering. Then, the following model is built about the surface tension worked to frazils. Here, for simplification, we liken frazil ice with a cube and consider the surface tension worked among 2 particles.

**Figure 6.** Definition sketch of the frazil ice model.
We define one side of cube as d, distance between cubes as l, surface tension as T, an angle of contact as \( \theta \) and water surface height risen by surface tension as h. Distance between the cubical centers L is same as L of equation (2). From balance of the weight of the water risen up and surface tension, h is described below.

\[
h = \frac{2T \cos \theta}{\rho_w g l}
\]  

[3]

The pressure acting on the risen water is negative pressure. Therefore, this negative pressure acts as power drawing two particles. If this power is set with Fs,

\[
Fs = \rho_w gdh^2 = \frac{d \cdot (2T \cos \theta)^2}{\rho_w g l^2}.
\]  

[4]

Since Fs is in inverse proportion to the square of l from equation (4), when the distance between 2 particles becomes the nearest (lmin), Fs max is shown. From (2) equation,

\[
\frac{L}{\Delta x} = \sqrt{1 + (ka)^2 - 2ka \sin(kx_{0n} - \omega t)} \equiv 1 + \frac{(ka)^2}{2} - ka \sin(kx_{0n} - \omega t)
\]

\[
L_{\text{min}} = \Delta x + \frac{(ka)^2 \Delta x}{2} - ka \Delta x
\]

\[
l_{\text{min}} = L_{\text{min}} - d = \Delta x + \frac{(ka)^2 \Delta x}{2} - ka \Delta x - d
\]

\[
Fs_{\text{max}} = \frac{d \cdot (2T \cos \theta)^2}{\rho_w g (\Delta x + \frac{(ka)^2 \Delta x}{2} - ka \Delta x - d)^2}
\]  

[5]

The above shows that Fs max is shown at wave peak.

Since acceleration is applied to frazils by the action of the wave, it is thought that it is acting as power pulling apart frazils. When we describe acceleration as \( f_i = d^2 L / dt^2 \), from equation (2), it is set to \( f_i = ka \Delta x \cdot gk \cdot \sin(kx_{0n} - \omega t) \). Then, acceleration becomes the maximum at wave peak.

\[
f_{i_{\text{max}}} = ka \cdot k \Delta x \cdot g
\]  

[6]

If power to pull apart is set to Fp here,

\[
Fp_{\text{max}} = m \cdot ka \cdot k \Delta x \cdot g
\]  

[7]

where m is mass of a frazil cube.
Since both of Fs and Fp take maximum at wave peak, it is expected that frazil ice gathers when \( F_{\text{sm}} > F_{\text{pm}} \). Based on this inequality, we perform numerical computation for “ka” term with a variable. The relation of a period and amplitude in case frazil ice can grow up to be initial pancake ice is clarified.

RESULTS
We calculate the value of “ka” which satisfies \( F_{\text{sm}} > F_{\text{pm}} \) and the value of “a (amplitude)” corresponding to it. Parameters are shown in Table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water density ( \rho_w )</td>
<td>1.03 g/cm³</td>
</tr>
<tr>
<td>Ice density ( \rho_i )</td>
<td>0.9 g/cm³</td>
</tr>
<tr>
<td>One side of cube ( d )</td>
<td>0.05 cm</td>
</tr>
<tr>
<td>Interval of calculating points ( \Delta x )</td>
<td>0.1, 0.2, 0.3 cm</td>
</tr>
<tr>
<td>Surface tension ( T )</td>
<td>75.64 dyn/cm</td>
</tr>
<tr>
<td>Angle of contact ( \theta )</td>
<td>8°</td>
</tr>
<tr>
<td>Period</td>
<td>0.4–2.0 s</td>
</tr>
</tbody>
</table>

If we calculate using these parameters, the value of \( F_{\text{sm}} \) become very large compared with \( F_{\text{pm}} \). In connection with it, the value of “a” also becomes very large. As this reason, the method of a setup of surface tension which works on a cube is mentioned. Although it assumes that a form of water which can be pulled up with surface tension is also almost like a rectangular parallelepiped, surface area of a liquid is considered to have become as to be shown in Fig. 7 in practice since surface tension may be made as small as possible. Since it is expected that the value of \( F_{\text{sm}} \) become smaller than the value calculated now from this, we thought of what kind of result is brought, when the value of \( F_{\text{sm}} \) was temporarily set to \( 1/50 \). \( 1/50 \) is arbitrary values. A result is shown in Fig. 8.

Fig. 8 shows the relation of period and amplitude in a boundary region which satisfies \( F_{\text{sm}} > F_{\text{pm}} \). It means that frazils can grow up to be initial pancakes in the area below a line. From this figure, it can be said that period and the amplitude is one of the conditions which frazil ice grows up to be an initial pancake ice. Moreover, since \( F_s \) is the function of the distance between frazils (l) and the value of amplitude differs from by the difference of \( \Delta x \), we can consider that formation of initial pancakes is related also to the density of frazil.

![Figure 7. Schematics of how to surface tension’s work.](image-url)
CONCLUSION

In this study, we concentrate on the process in which frazil ice grows up to an initial pancake. First, we described the locus of movement of frazil ice using the linear theory of the wave motion. This result was in agreement with the observation of the experiment using polypropylene particles. Next, we performed easy modeling about two frazil particles. We mentioned the relation between the formation of initial pancake ice and the wave conditions from balance of the power by the surface tension and acceleration which may affect to frazil ice. We obtained the result that period and amplitude is related to initial pancake formation and also related to the density of frazil ice because Fs is the function of distance of two frazil particles. This will be one reasonable possibility that the conditions of initial pancake formation can be searched for with wave period and amplitude.

However, in this study, since we have formed various assumptions, we need to solve them. The 1st is having modeled frazil ice as a cube. Originally frazil ice is considered to be a ball or an ellipse ball, and it can be expected that the surface tension which acts on it will become smaller than that works on a cube. Although it was only expressed as 1/50 this time, it can be said that it is necessary to stand a strict model. Having made the model only about two frazil particles is mentioned to the 2nd. Since the difference of the water level which can be pulled up with surface tension becomes small when more than two particles exist, it is thought that Fs becomes small. We plan to pursue these influences deeply by future research.

ACKNOWLEDGMENTS

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Figure 8. The relation of period and amplitude for formation of initial pancakes.
support and the professional execution of the test programme in the Research Infrastructure ARCTECLAB.
The Design, Construction, and Observation of Permanently Installed Safety Booms in Ice Covered Waters

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ABSTRACT
Safety booms to warn boaters of fast water currents that leads to dangerous situations upstream and downstream of hydroelectric power plants have been deployed at many generating stations in the world in order to minimise potential accidents resulting from accidental intrusion. In cold regions, where ice is present during the winter season, these booms are installed in early June and removed in October to avoid damage by ice. However, significant delays were encountered in some years for the deployment of these booms due to high river flow, which lasted several days after the start of the summer season when boater’s traffic is seen to increase in these waterways. It is also difficult to decide on the date for the removal of these booms, as the window of opportunity between the desired date of removal and the start of ice formation is sometimes very short. This can make the task of removing the boom difficult to plan ahead, and more than often, the boom is removed much earlier than desired.

The purpose of this paper is to describe the design, construction, and observation of one typical safety boom installed in the summer of 2006 in the Headpond of Bark Lake flow control dam, owned and operated by Ontario Power Generation (OPG), Ontario. This boom was designed to remain in the water year round.
The boom design considerations including the prevailing ice conditions at the site, the historical water discharge and the associated currents, water level fluctuations, and the ice observed during the winter of 2007 is described.

Introduction
The challenges during the design process of the Bark Lake safety boom were that the boom was to be subjected to during the ice freeze up, during the winter and the spring break-up. Some of the design criteria are listed below:

- The boom must remain in the water year-round.
- The boom is to be placed in a relatively large lake where winds and waves are expected to be moderate.
- The water level usually varies by 10 metres, with its highest level during the freeze up and the lowest level during the winter months.
- The slopes of the lake’s shores are steep on the North side but relatively shallow on the South side.
- The lake is “L” shaped.
- When the spring runoff starts, the ice would be at its lowest level. The flow and the associated current velocity start to increase applying significant load on the boom.
- The boom had to be reasonably flexible to resists the ice forces for relatively high currents and fast rising water level.
- To ensure the ice loads remain below the resistance capacity of the boom, a criterion for opening the gates of the Bark Lake Control Dam located about 200 m downstream of the boom was proposed so that the ice forces remain within acceptable limits.
- The boom was in a remote area, thus any failure of the boom components would result in significant effort to repair.

Design Challenges and Improvements
The Bark Lake boom design criteria were unique to previously designed booms. It posed the following challenges that required an extension to existing boom technology:

(a) The boom was to minimize the time when a warning system is not in place. This was considered to be an important feature as for most summers, it is not possible to deploy safety booms until early June due to the high flow in the river and the safety risk to the crew installing the boom. They are also removed earlier than desired if the forecast calls for higher flow in the coming days.

(b) The boom was exposed to water level changes of about 10 m over the winter. As a result, it would be frozen on the shores over most of the winter, and would need to release in the spring when the spring freshet causes an increase in water level. As well, it would have to be capable of dealing with mid-winter thaws that might bring water into the reservoir. Furthermore, all parts of the boom (eg, anchors, cables, etc) had to be designed to accommodate such large displacements.

(c) As the boom was placed in a relatively remote area, and considering the boom will see high loads in conjunction with 10 m water level change during winter in presence of ice, the components that connect the floats to the suspended cables were designed to ensure minimum tension and wear would occur. For example, the attachment points between the floats and the cable were increased to distribute the load on 50% more connecting points.
(d) As the loads on the boom were expected to be high, the floats were made shorter, so the load applied on each float is reduced. This was done to reduce the wear of the hardware in waves during the summer as well.

(e) The design case was established presuming that active management would be employed by the dam operator (OPG) to limit the loads on the boom.

**Description of the boom**
The boom is located on the Madawaska River, upstream of the Bark Lake Dam reservoir (Geographic Coordinates: N45.416815°, W77.789190°). The boom consists of two spans (North and South) anchored on both shores and attached in the centre to a buoy anchored to the shore via a long anchor cable. The boom spans a width of about 262 m. No anchors are placed in the water. Figures 1 and 2 show the location of the boom in Bark Lake Dam Reservoir and the boom layout and its anchor points. The position of the anchors and the length of each of the boom spans were determined based on the lake configuration and the water depth.

![Figure 1. The Location of the Boom.](image1)

![Figure 2. The Reservoir Bathymetry.](image2)

**The Current Distribution on the Lake**
The current distribution in the lake varies in time and in space. It depends on the water level elevation and the river discharge throughout the year. Figure 3 shows the annual discharge curve as well as the corresponding, maximum, minimum and average lake level elevations.

The maximum discharge to be seen by the boom when an ice cover is still present, will occur in early April, and is between 300 and 500 m³/s. The corresponding water level elevations in early April may be 304, 307 or 310 m as shown in Figure 3.

The current velocity for the historic maximum flow of 500 m³/s was calculated for three cross sections taken across the boom layout corresponding to the three possible water level elevations (see Figure 4). They are summarized below:

- For 304 m elevation: Average current velocity = 2.26 m/s
- For 307 m elevation: Average current velocity = 0.64 m/s
- For 310 m elevation: Average current velocity = 0.36 m/s
For a flow of 300 m$^3$/s, the average current velocities are:

- For 304 m elevation:  Average current velocity = 1.36 m/s
- For 307 m elevation:  Average current velocity = 0.38 m/s
- For 310 m elevation:  Average current velocity = 0.22 m/s

A numerical model was also run for the Bark Lake Headpond using SMS software in order to compare to the above current values. The model was run with the following conditions:

1. The maximum discharge under the Water Management Plan (2000-2005) provided by OPG and the corresponding low water level under the Plan, given in the OPG graphs as 225 m$^3$/s and 310 m, respectively.
2. The historic maximum flow of 500 m³/s and a water level of 310 m.
3. The historic maximum flow of 500 m³/s and a water level of 307 m.

The validity of the model runs were checked for continuity. All three simulations met the criterion suggested by the software developer for the continuity calculation to be within 3%. The maximum velocity in all three simulations occurred at the outlet of the reservoir, which is far from the boom site and therefore not necessarily relevant to this analysis.

The mean velocity (across the entire model domain) for each of the three simulations was:

- Simulation #1: Discharge 225 m³/s and 310 m elevation: Mean velocity = 0.25 m/s
- Simulation #2: Discharge 500 m³/s and 310 m elevation: Mean velocity = 0.50 m/s
- Simulation #3: Discharge 500 m³/s and 307 m elevation: Mean velocity = 1.22 m/s

Screen captures showing the velocity distributions for the simulations for 310 m water level elevation are shown in Figures 5 and 6.

![Figure 5. Flow 225 m³/s, velocity 0.25 m/s.](image)
![Figure 6. Flow 500 m³/s, velocity 0.50 m/s.](image)

A screen capture showing the velocity distributions for the simulations for 307 m water level elevation is provided in Figure 7.

The current velocity obtained from the SMS model was not used to calculate the ice forces, as it was the average current velocity over the entire model domain. The velocity is therefore much higher with the SMS model as the one with the cross section taken 200 m upstream from the gates, where the current is very high.

While the SMS results weren’t used directly for boom design, they were helpful for understanding the hydraulic regime.

**The Ice Forces on the Boom**

The ice force on the boom was calculated using a model calibrated using field measurements (Abdelnour et al, 1998). The load on the boom was calculated for a maximum wind velocity of 100-km/hr. The average current velocity calculated for the two flows considered (i.e., 500 and
300 m$^3$/s) and for the three water levels considered (i.e., 304, 307 and 310 m) were used. The boom span width was calculated for each water level elevation. The effective area consists of a triangle with 30 degrees apex angle and the boom width as its base (Abdelnour et al., 1993;1994). The drag forces were then calculated and are related to the effective area, the wind and current velocity. The wind and ice drag coefficients used are for rough ice and were obtained from Michel, 1978.

The maximum load applied on the boom was calculated to be 372 tonnes (see Table 1). This load occurs when there is an ice cover and the water level is at 304 m (the lowest level) and when the maximum discharge is 500 m$^3$/s. This is due to the fact that the current is much higher for this combination (shown previously). However, it is a very unlikely situation, and large boom components would have been required if it were adopted as the design case. For example, the load would be higher than the resistance capacity of a 38 mm (1.5") diameter anchor cable. Larger anchor cable can be used but will be much more costly to acquire and deploy. The calculated load on the 38 mm anchor cable would be 186 tonnes while the maximum working load of the 38 mm cable is 130 tonnes.

<table>
<thead>
<tr>
<th>Span Width (m)</th>
<th>Water Level (m)</th>
<th>Average Depth (m)</th>
<th>River Discharge (m$^3$/s)</th>
<th>Apex Angle (Deg.)</th>
<th>Distance From Boom to Apex (m)</th>
<th>Effective Area (m$^2$)</th>
<th>Wind Speed (km/hr)</th>
<th>Average Current Velocity (m/s)</th>
<th>Force on the Boom (kN)</th>
<th>Line Load on the Boom (kN/m)</th>
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<tbody>
<tr>
<td>123</td>
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<td>1.80</td>
<td>500</td>
<td>30</td>
<td>230</td>
<td>14116</td>
<td>100</td>
<td>2.26</td>
<td>3646</td>
<td>29.6</td>
<td>372</td>
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<tr>
<td>211</td>
<td>307</td>
<td>3.70</td>
<td>500</td>
<td>30</td>
<td>394</td>
<td>41539</td>
<td>100</td>
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<td>30</td>
<td>489</td>
<td>64046</td>
<td>100</td>
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<thead>
<tr>
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<td>360</td>
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</table>
After some evaluation, a current velocity of 1.36 m/s, resulting from a 300 m$^3$/s discharge and corresponding to 304 m water level elevation, was used as the design case. This was adopted in combination with the implementation of active management by the operator (OPG). Therefore, it was important that the operator control the discharge during early April, and when an ice cover is present, to ensure the flow will never surpass 300 m$^3$/s as long as the water level is below 307 m. For open water conditions, the load will be insignificant for all possible levels and should not affect the boom.

The Floats and Span Cables
Ontario Power Generation (OPG) provided a guideline for the design of a safety boom. According these guidelines, the gap between the floats must be less than 2 m, so that boats cannot pass through. Also, for good visibility, the floats must have at least 305 mm (12"") freeboard. The float thickness was 6.4 mm (1/4") to resist the ice forces in the area. In order for the float to have at least 305 mm freeboard, the required diameter for 6.4 m long float (a selected length of 6 m plus 0.2 m for each of the spherical ends) was 559 mm (22"") as presented in Table 2. The overall specific gravity of the pontoon was 0.42 and the calculated freeboard was 324 mm as shown in Figure 8.

As the total width of the boom is 262 m, the boom was split into two spans. Because of the geometric shape of the reservoir, the south span was twice as long as the north span. The north span has 14 floats and the South span has 28 floats for a total of 42 floats. The North span cable was 111 m long and the South span was 222 m long. The span width of the North span was 100 m and the south span was 210m and the total width of the boom was 283 m. The ratio of the cable length to the boom span width is about 18%. This is a relatively large ratio for a boom and was selected to allow flexibility in the system when the ice load is applied.
The south span was divided into two separate cables connected with a junction plate and a small buoy. This was done to ease the boom construction and simplify future repair and maintenance of the boom and the cables. Figure 9 shows the cables layout and arrangement of the floats. In Figure 10, a picture was taken of the boom during the month of March 2007.

**Boom Resistance**
The boom resists the ice forces until the force exceeds the resistance capacity of the floats. This assumes that the ice is not frozen to the boom. This is usually the case in late winter due to exposure of the steel floats to the sun. The maximum resistance of the floats for 559 mm (22”) diameter is 7 kN/m. The line load applied on the boom, presented in Table 3 was calculated from full scale measurements carried out on the Lake Erie Ice boom (Crissman et al, 1995). The line load is directly related to the friction between the ice and the float and the buoyancy force applied by the submerged float.

![Figure 9. The Float Distribution on the North and South spans.](image-url)
The Centre Buoy
A buoy was built to hold the junction plate that connects the two span cables and the 195 m long anchor cable together. The vertical loads applied on the buoy from the cables and the junction plate and the required minimum freeboard of 24” required by the OPG guidelines in open water conditions (for a clear visibility of the warning displayed on the buoy) necessitated a relatively large buoy. The buoy had 1219 mm (48”) diameter and 1778 mm (70”) length. The freeboard was 697 mm in open water conditions. With maximum ice load, the cable would stretch and the load on the buoy will increase significantly. With maximum ice load the freeboard would be 457 mm as shown in Figures 11 and 12.

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Length</th>
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</thead>
<tbody>
<tr>
<td>(in)</td>
<td>(mm)</td>
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<tr>
<td>48</td>
<td>1219</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>End Plate Thickness</td>
</tr>
<tr>
<td>(in)</td>
<td>(mm)</td>
</tr>
<tr>
<td>0.250</td>
<td>6.25</td>
</tr>
<tr>
<td>0.250</td>
<td>6.25</td>
</tr>
<tr>
<td>pipe/ends/bars weight</td>
<td>Maximum Buoyancy</td>
</tr>
<tr>
<td>(lb)</td>
<td>(kg)</td>
</tr>
<tr>
<td>928</td>
<td>453</td>
</tr>
<tr>
<td>4572</td>
<td>2076</td>
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</tbody>
</table>

Figure 11. The buoyancy calculations.

Figure 12. The Buoy

The Anchors
Three anchor points were needed to hold the boom in place. Six anchors were installed, two at each location, two in the centre, two on the North shore and two on the South shore as shown in Figure 13.
In order to avoid unnecessary and costly drilling in the riverbed, which would have required a barge and the complication of its deployment in a relatively remote location, it was decided to position the two centre anchors on shore. The required cable between the buoy junction plate and the anchors was 195 m long as shown in Figures 13 and 14.

The centre anchor cable connects from Anchor point A (anchors A1 and A2) to the junction plate under the centre buoy. In order to have the buoy at the desired location during the open water season as shown in Figure 14, the cable had to be long enough, as it will rest on the riverbed when not in tension as shown in Figure 13. During the winter conditions, and when the ice loads is applied on to the boom, the cable would stretch and extend several meters resulting in the displacement of the central buoy in the downstream direction until the cable is fully stretched. Anchor “A” takes half the load applied on the ice boom. Since the boom was designed for a maximum current velocity of 1.36 m/s and a 300 m³/s discharge with a corresponding water level elevation of 304 m, the load on Anchor A would be 68 tonnes. The load on the other two anchors, north and south would be half, 34 tonnes as presented in Table 4.

![Figure 13. The Boom Layout.](image1) ![Figure 14. The Anchors A1 &A2 and the Cable to the Buoy.](image2)

<table>
<thead>
<tr>
<th>Span Width (m)</th>
<th>Water Level (m)</th>
<th>Average Depth (m)</th>
<th>River Discharge (m³/s)</th>
<th>Force on the Boom (kN)</th>
<th>Line load on the Boom (kN/m)</th>
<th>Load on the Boom (tonnes)</th>
<th>Load on the two Side Anchors (tonnes)</th>
<th>Load on the Center anchor (tonnes)</th>
</tr>
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<tr>
<td>123</td>
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<td>989</td>
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<table>
<thead>
<tr>
<th>Span Width (m)</th>
<th>Water Level (m)</th>
<th>Average Depth (m)</th>
<th>River Discharge (m³/s)</th>
<th>Force on the Boom (kN)</th>
<th>Line load on the Boom (kN/m)</th>
<th>Load on the Boom (tonnes)</th>
<th>Load on the two Side Anchors (tonnes)</th>
<th>Load on the Center anchor (tonnes)</th>
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<tbody>
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<td>45</td>
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<tr>
<td>262</td>
<td>310</td>
<td>5.30</td>
<td>300</td>
<td>360</td>
<td>1.4</td>
<td>37</td>
<td>9</td>
<td>18</td>
</tr>
</tbody>
</table>

**Table 4. Ice Forces on the Anchors.**

The six shore anchors were drilled to a depth of 6 m into the rock. The overburden was about 14 m on average and the maximum was 20 m. The diameter of the hole was 127 mm (5”). Anchors rods were dropped in the hole after welding on to it a 1-inch chain. Grout was used to secure the chain in the holes.

Each of the two chains were connected to a junction plate where the anchor cable or the span cables’ spelter sockets was attached.
Conclusions
After two years of operation, the boom has performed as expected. The boom has remained in the water year-round for the past two years. The water level has varied by 10 metres and the boom rested on the bottom of the lake for a period of at least two months as shown in Figure 15. When the water level rose, the boom slowly lifted and took its open water position for the summer season.

The project has achieved successes in providing river users with a warning system that remained in place until the start of the ice freeze up and was ready again in the spring as soon as the ice had disappeared. This has minimized the window where a warning system is not in place. It is an important feature as for most summers; it is not possible to deploy safety booms until early June due to the high flow in the river and the safety risk to the crew installing the boom. They are also removed earlier than desired if the forecast calls for higher flow in the coming days.

Several booms with similar design criteria were constructed in 2007 and more are under construction in 2008.

Acknowledgments
The authors would like to acknowledge Mr. Peter Kenny of Sullivan and Son for carrying out the boom installation, Mr. Blake Pierce and Louis Adeghe of Ontario Power Generation, who provided environmental data and developed the requirements for this project and Andrew Liddiard, who executed the numerical modelling while employed as Senior Engineer with BMT Fleet Technology.

References

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Ice Entrainment through Submerged Gates

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ABSTRACT – Data obtained in a flume using plastic blocks to simulate ice entrainment through submerged outlet gates were obtained in 1978. The data at that time were presented in a plot based on a Froude number based on exit velocity and depth to top of gate opening. While good discrimination was achieved between entrainment and non-entrainment conditions, the data has since been reanalyzed to present it in a more useful form in terms of the approach Froude number of the flow and the depth to top of gate opening.
1. Background

Back in 1978 the author had a graduate student working at CRREL execute under his supervision a series of tests (approximately 80 with “large” blocks and approximately 80 with “small” blocks plus some with mixture of block sizes) in a fairly large (0.92m x 0.92m) cross section flume where plastic blocks were introduced from upstream and the behavior at a submerged outlet determined. The blocks were plastic of the same specific gravity of ice (0.92) and of two sizes (6.4 mm x 0.037 m x 0.037 m for the small blocks and 6.4 mm x 0.074 m x 0.074 m for the large blocks) to simulate ice and various outlet depths and dimensions were used but all nearly full width of the flume (0.84 m). The main determination was whether or not the blocks were entrained and passed out of the outlet or whether they were retained at the surface upstream of the barrier. The results were reported in a paper presented at the Lulea, Sweden IAHR Ice Symposium in 1978 (Stewart and Ashton, 1978).

At the time (nearly 30 years ago) it was believed that the most important variables were a Froude number based on the outlet velocity and depth of the top of outlet beneath the upstream water surface and the ratio of outlet depth to full depth. Pretty good discrimination between entrainment and non-entrainment was achieved using these variables.

2. Previous Results

The results were reported in terms of a plot of $V_e/(gH_1)^{1/2}$ versus $H_2/H$ where $V_e$ is the exit velocity at the outlet, $g$ is gravity, $H_1$ is the depth from the water surface to the top of the outlet, $H_2$ is the height of the outlet, and $H$ is the full depth upstream of the barrier. See Figure 1. (Note that Figure 1 taken from the original report is somewhat incorrect as the outlet was often submerged beneath the downstream flow; however, that does not change the upstream velocities calculated in this analysis since the outlet velocities were calculated from flume discharge meters.)

It is believed a more applicable examination of the data as applicable to intakes (and in light of some 30 years more experience) would be in terms of the upstream velocity $V$ and depth $H$ rather than $V_e$ and $H_1/H$ in a similar plot. This would enable a more ready examination of the depth of outlet on entrainment of arriving ice pieces. Unfortunately the raw data was not presented in the paper, and particularly values of $H$ and $H_1$ so it is not possible to recover the values of the upstream velocity $V$ from that data. However, in the author’s file archives were found 4 data sheets that have values for each experiment for $V_e/(gH_1)^{1/2}$ and, more importantly, the corresponding value of $H_2/(H_1+H_2)$. With this table of data it is possible to recover the value of $H_1$, then recover the value of $V_e$, then recover the value of the upstream velocity $V$ and from the data plot in the paper (using the known values of $H_2$ ) then recover the value of $H$. The file archives unfortunately were missing the data on measured thicknesses of accumulation upstream of the gate and this lack resulted in some difficulties in interpreting the data as discussed below when dealing with the erosion velocity.
3. Description of Experiments

Various depths of flow were used in the 1978 experiments. Typical upstream depths were in the range of 0.4 to 0.7 meters. The outlets were of three different heights; 0.0762 m, 0.152 m, and 0.229 m. The depths to top of outlet were varied. One of three outcomes of the individual runs was recorded: Entrainment, no entrainment, and the borderline cases where some ice was entrained (< 10 % of the supply) and passed through the outlet. The results as originally presented are shown in Figures 2 and 3 in terms of the exit Froude number \( \frac{V_e}{(gH_1)^{1/2}} \) versus \( \frac{H_2}{H} \) where \( V_e \) is the exit velocity through the gate opening, \( g \) is gravity, \( H_1 \) is the depth beneath the water surface to the top of the opening, \( H_2 \) is the height of gate opening, and \( H \) is the upstream depth of flow.

4. Original Results Presentation

The original results were presented in two plots, one for the small block experiments, and one for the large block experiments. These are presented in Figures 2 and 3 (adapted from Stewart and Ashton, 1978).

In Figures 2 and 3 the solid data points represent experiments in which significant entrainment of blocks through the outlet occurred and the open data points represent experiments in which no entrainment occurred. The red data points represent experiments where there was a notation “less than 10% of blocks” were entrained. The lines discriminating between the entrainment and non-entrainment experiments were determined in the original paper “by eye” and for both sets of experiments was found to be:

\[
\frac{V_e}{(gH_1)^{1/2}} = 0.4 \left( \frac{H_2}{H} \right)^{1/2} \tag{1}
\]

In the Figures 2 and 3 above this line has been replaced by a line determined by a least squares fit of the “< 10%” data points and for the small block experiments was found to be:

\[
\frac{V_e}{(gH_1)^{1/2}} = 0.37 \left( \frac{H_2}{H} \right)^{-0.6} \tag{1a}
\]

And for the large block experiments was found to be:

\[
\frac{V_e}{(gH_1)^{1/2}} = 0.38 \left( \frac{H_2}{H} \right)^{-0.57} \tag{1b}
\]

These are both nearly identical to the discrimination found “by eye.”

In light of experience gained since the original paper was prepared, it is clear that a more useful presentation of the data would be in terms of the upstream Froude number \( \frac{V}{(gH)^{1/2}} \) versus \( H/H_1 \) where \( V \) is the upstream approach velocity (averaged over the full depth), \( g \) is gravity, \( H_1 \) is the depth beneath the water surface to the top of the opening, and \( H \) is the upstream depth of flow. The data has been reworked as described above using the archival data sheets and yield the Figures 4 and 5.

For both data sets the discrimination line in terms of these variables is:
\[
\frac{V}{(g H)^{1/2}} = 0.28 (H_1/ H)^{0.85} \tag{2}
\]

This presentation of the data is considered more useful since in practical cases the upstream Froude number is specified in terms of the flow parameters in the approach channel and the depth to the top of the outlet gates that either precludes entrainment of ice or enables is one of the design objectives.

5. Limitations of the Data

It is tempting to assume that if the conditions of the practical problem are such that they are in the “NO ENTRAINMENT” zones shown in the Figures, that there is no entrainment. In hindsight, however, it is clear that the conditions of the experiments were such that in some experiments there was inadequate supply of blocks, or length of channel upstream, to allow an equilibrium thickness of accumulation to form upstream of the gate structure. More importantly the conditions of the experiments were such that the accumulations were of the “narrow jam” type (discussed below). If the conditions are of the “wide jam” type, a reasonable design approach would be to run a numerical simulation (such as the DynaRICE model of Clarkson University; see Shen et al (2000)), determine the depth of flow beneath the accumulation, and use that dimension for H in calculating the Froude number and the \(H_1/H\) variables. \(H_1\) would be the distance from the bottom of the accumulation to the top of the gate opening. (Note: the DynaRICE simulation does not handle the details of entrainment at submerged gates but can be used to calculate the thickness of an accumulation arrested at a surface barrier.)

The other limitation is due to use of only one block thickness for both the large and small blocks. Presumably the limiting velocities associated with the results presented above could be scaled according to the densimetric block Froude number, or in the ratio \((t/ t_i)^{1/2}\) where \(t\) is the block thickness for the design conditions and \(t_i\) is the block thickness used in the experiments \((0.0064 \text{ m})\). These concepts are explored briefly below.

6. Approach to Data in Terms of Block Froude Number

Another approach to the data is to examine the behavior of individual blocks with the overall concept being that if a single block is unstable, it will be transported downstream. Ashton (1974a) reworked the data of Uzuner and Kennedy (1972) for the critical velocity at which a block of ice arriving at a downstream barrier is entrained into the flow. Subsequently Tatinclaux and Gogus (1981) performed a series of experiments on the critical velocity at which a submerged block is unstable and hence would be entrained into the flow when submerged beneath a downstream ice cover. The experiments and analysis of Tatinclaux and Gogus improved on the results reported earlier by Ashton (1974b) and resulted in the relationship for the block Froude number \(F_b\) (above which a block is unstable) in the form:

\[
F_b = V_b / \left[ g(1 - \rho_i / \rho) t_i \right]^{1/2} = \left[ -2.26 (t_i / L)^2 + 2.14 (t_i / L) + 0.015 \right]^{-1/2} \tag{3}
\]

Where \(V_b\) is the entrainment velocity, \(g\) is gravity, \(\rho_i\) is the density of ice, \(\rho\) is the density of water, \(t_i\) is the block thickness, and \(L\) is the block length. In the present experiments \(t_i / L = 0.173\)
for the small blocks and \( t_i / L = 0.0865 \) for the large blocks which yields critical values of \( F_b \) of 1.77 for the small blocks and 2.32 for the large blocks. It should be noted that the experimental relationship of Tatinclaux and Gogus was for a block initially horizontal beneath a flat downstream ice cover. It is suspected that a block coming to rest beneath an irregular surface could be entrained at either a higher or lower \( F_b \).

Beltaos (1995), in an analysis of submergence and deposition of ice floes encountering an existing cover, found a relationship for the upstream Froude number and the thickness of accumulation \( t_j \) and the porosity \( p \) in the form (Beltaos equation 4.10, p. 115)

\[
F = V / (gH)^{1/2} = [2 \left(1 - \rho_i / \rho\right)(1 - p) t_j / H]^{1/2} \left[1 - t_j / H\right]
\]

Regardless of the porosity, the maximum \( F \) occurs when \( t_j / H = 0.33 \), and this suggests strongly that the maximum thickness that will occur by block entrainment and deposition is about 0.33 \( H \); further the maximum \( F \) is limited to 0.154 \( (1-p)^{1/2} \) where \( \rho_i / \rho = 0.92 \).

7. Comparison with Present Results

The above analyses strongly suggest that examination of the conditions experienced by individual blocks will yield insights into the question of whether or not a block of ice will be entrained into the flow. While the original paper (Stewart and Ashton 1978) reported that the length and thickness of accumulation of blocks upstream of the outlet were measured, that data has not been found. Nevertheless, the block Froude number \( F_b \) was calculated using the upstream velocity \( V \) to calculate \( F_b \) and the results are presented in Figures 6 and 7.

8. Discussion

Figures 6 and 7 give almost identical results. The solid discrimination lines in the figures were drawn in the context of the discussion above of block Froude number dependency. Up to about \( H_1 / H = 0.15 \) the data is inconclusive. This region represents intakes with shallow depth of submergence and should be avoided if the intent is to avoid entrainment into the outlet flow. From \( H_1 / H = 0.15 \) to \( H_1 / H = 0.33 \) the data well discriminates between entrainment and non-entrainment. Since it is believed the limiting depth of accumulation of “narrow jams” is at 1/3 the depth, a horizontal line has been drawn that seems to reasonably discriminate between entrainment and non-entrainment results. There is less certainty in the results as \( H_1 / H \) increases beyond about 0.6 simply because it is doubtful that there was sufficient ice supply in the experiments to be sure that equilibrium conditions were attained. The value of \( F_b \) associated with the horizontal lines is considered to represent a critical “erosion velocity” of the ice pieces. For both large and small block experiments the value depicted is \( F_b = 3.7 \). This is considerably larger than the value that would be predicted based on the data of Tatinclaux and Gogus (1981). It is also noted that since the upstream velocity was used in the calculation of \( F_b \) the actual value associated with the somewhat higher velocity beneath the accumulation would yield even higher values of the threshold \( F_b \).
9. Implications for Design of Intakes to Avoid Passing Ice

Based on the above results, an approach to design of intakes to avoid passing fragmented ice is as follows:

1) Establish whether or not the upstream conditions are such that the “narrow” jam criterion is appropriate. While somewhat simplified, the analysis of Beltaos (1995) suggests a “narrow” jam cannot exist unless B/H is less than about 7. (In the experiments reported here B/H was in the range 1.2 to 3.)
2) With the upstream depth of flow and velocity, calculate the upstream Froude number \(F = V/(gH)^{1/2}\).
3) Calculate the depth to top of gate opening \(H_1\). If \(H_1/H\) is less than about 0.33 use the Figure of Froude vs \(H_1/H\) to determine whether entrainment is likely.
4) If \(H_1/H\) is greater than about 0.33, calculate the block Froude number \(F_B = V/[g(1-\rho_i/\rho)t]^{1/2}\). In calculating \(F_B\) a representative “thin” thickness of block should be used since thinner blocks are more easily eroded from the underside or otherwise transported to depth in the flow. For example ice pieces moving downriver at the time of breakup typically lose as much or more than half their thickness at the time of initial movement.

10. Implications for Design to Pass Ice

Sometimes the objective of a design is not to retain ice but to pass the ice through a gate structure. Tuthill (2008) provided some data on Corps of Engineers navigation dams where the objective is to pass ice through gates. The data associated with ice passage had upstream Froude numbers varying from 0.28 to 0.40 and with \(H_1/H\) values ranging from 0.47 to 0.6. This data set plots well above the discrimination lines in Figures 4 and 5 and, while not enabling better discrimination, is fully in accord with the present results. It is not known what the thicknesses of the ice were in those cases. The approach velocities were in the range of 2 to 3.2 m/s; assuming a typical floe thickness of 0.3 meters results in block Froude numbers \(F_u\) ranging from 4.1 to 6.6 and again are in accord with the present results.

11. Conclusions

Reexamination of data from a series of flume experiments using plastic blocks to represent ice found the results to be more or less consistent with existing theory. The critical “erosion velocity” was found to be somewhat higher than would be expected from the results of idealized experiments on stability of ice blocks beneath a smooth ice cover. The simple result is that, if the conditions are such that if the ice accumulation upstream of the outlet is of the “narrow jam” type, then the depth of the top of the opening should be at least 1/3 of the total upstream depth. If greater than 1/3 then the block Froude number should be calculated and limited to about 3.7 using the upstream velocity. If the conditions are such that a “wide jam” may occur upstream, then the thickness of the accumulation should be calculated using the appropriate theory, and with the thickness of such an accumulation in hand, then apply the above results, with \(H_1\) taken as the depth from bottom of accumulation to top of outlet opening and \(H\) as the depth beneath the accumulation.


References


Figure 1. Definition Sketch
Exit Froude vs $H_2/H$  SMALL BLOCKS

\begin{align*}
\text{Exit Froude} &= \frac{V_e}{(gH_2)^{1/2}} \\
y &= 0.37x - 0.6
\end{align*}

Figure 2. Original presentation of small block data

Exit Froude vs $H_2/H$  LARGE BLOCKS

\begin{align*}
\text{Exit Froude} &= \frac{V_e}{(gH_2)^{1/2}} \\
y &= 0.38x - 0.57
\end{align*}

Figure 3. Original presentation of large block data
Figure 4. Small block data in terms of upstream Froude number

Figure 5. Large block data in terms of upstream Froude number
Figure 6. Block Froude number $V/\left[g(1-\rho_i/\rho)\left(t_i\right)^{1/2}\right]$ vs $H_1/H$ – Small blocks

Figure 7. Froude number $V/\left[g(1-\rho_i/\rho)\left(t_i\right)^{1/2}\right]$ vs $H_1/H$ – Large blocks
Special Session: Numerical simulation in ice engineering
Two-Dimensional Numerical Model for River-Ice Processes Based upon Boundary-Fitted Coordinate Transformation Method

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Abstract: River ice is a kind of natural phenomena in the cold region under specific conditions of meteorology, geomorphology and hydraulics. In order to simulate accurately the complicated boundary conditions and overcome the problems caused by wide gap of scale between length and width, a 2-D river-ice numerical model based upon the boundary-fitted method has been developed, considering the influence of the frazil ice accumulating under ice cover and the shape of freezing fringe of ice cover during the river-ice processing etc. The presented model is capable of determining the velocity field, distribution of water temperature, concentration distribution of the frazil ice, transportation of the floating ice, progression, stability and thaw of ice cover, transportation, accumulation and erosion of ice under ice cover. MacCormack scheme is used to solve numerically the equations. The model is validated by the field observations at Hequ Reach of the Yellow River, and the comparison results show that the 2-D numerical model presented is capable of simulating the river-ice process with high accuracy.
1. Introduction

The river-ice evolution is influenced by river hydraulics, meteorology, thermodynamics, geomorphology, etc. The meander side-bank of the river makes it impossible to use rectangular Cartesian coordinates to deal with complex geometric boundaries accurately. In the recent more than twenty years, researchers have developed many theories to simulate the one-dimensional river-ice models. Generally speaking, the one-dimensional river-ice model is only applicable for straight channel and can only simulate the section-averaged values varied along the river. For natural river, the water depth, velocity and ice cover thickness vary dramatically along the width of river. So it is necessary to consider the characteristic variations along the width of river in the river-ice analysis and numerical simulation.

The transformation method of Boundary-fitted-coordinate (BFC in short word) has been applied widely in many fields, it can be used to solve with high accuracy the complex geometrical boundary and corresponding boundary conditions. As shown in Fig.1, the solution is conducted on the fixed rectangular perpendicular grids.

Figure1. x-y domain and ζ-η domain

In this paper, two-dimensional numerical model of river-ice based upon BFC is developed, including river hydraulics, ice transportation, thermodynamic and freezing, etc.

2. River-ice Two-dimensional Mathematical Model

2.1 River Hydraulics

The two-dimensional shallow water equations in conservational form in terms of distance direction x and width direction y is used as governing equations, with considering resistance under ice cover (Wang, 1999)

\[
\frac{\partial h}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0
\]

\[
\frac{\partial q_x}{\partial t} + \frac{\partial}{\partial x} \left( \frac{q_x^2}{h} + \frac{gh^2}{2} \right) + \frac{\partial}{\partial y} \left( \frac{q_x q_y}{h} \right) = hb_x - gh \frac{\partial}{\partial x} \frac{\partial t}{\partial x} = b_x
\]

\[
\frac{\partial q_y}{\partial t} + \frac{\partial}{\partial x} \left( \frac{q_y^2}{h} + \frac{gh^2}{2} \right) + \frac{\partial}{\partial y} \left( \frac{q_x q_y}{h} \right) = hb_y - gh \frac{\partial}{\partial y} \frac{\partial t}{\partial y} = b_y
\]
in which $\rho$, $\rho_i$=density of water and ice respectively; $h$=water depth; $t_i$=ice-cover thickness; $u$, $v$=velocity component in $x$ and $y$ direction respectively. For ice-free case $b_x = \frac{1}{\rho} \frac{\partial p_x}{\partial x} - g \frac{\partial z_x}{\partial x} + \frac{\tau_{nx} - \tau_{nx}}{\rho h} + F_{hx}$, $b_y = \frac{1}{\rho} \frac{\partial p_y}{\partial y} - g \frac{\partial z_y}{\partial y} + \frac{\tau_{ny} - \tau_{ny}}{\rho h} + F_{hy}$; For with ice-cover case: $b_x = \frac{1}{\rho} \frac{\partial p_x}{\partial x} - g \frac{\partial z_x}{\partial x} + \frac{\tau_{nx} - \tau_{nx}}{\rho h} + F_{hx}$, $b_y = \frac{1}{\rho} \frac{\partial p_y}{\partial y} - g \frac{\partial z_y}{\partial y} + \frac{\tau_{ny} - \tau_{ny}}{\rho h} + F_{hy}$, in which $p_s$=water-surface air pressure; $z_s$=elevation of riverbed; $z = z_s + h$ =elevation of water-surface: $\tau_b = \tau_{bx} + \tau_{by}$ =resistance of riverbed; $\tau_i = \tau_{ix} + \tau_{iy}$ =resistance of ice-cover underside; $\tau_s = \tau_{sx} + \tau_{sy}$ =resistance of ice-cover underside; $\tau_{ws} = \rho_s C_w w_s$=wind* shear stress; in which $\rho_s$=air density; $w_s$=wind speed; $C_D$=coefficient of wind’ shear stress, $\tau_w = |\tau_w| \cos \theta$; $\tau_o = |\tau_o| \sin \theta$, $\theta$=angle between wind velocity and downstream direction of river; $F_{hx} (= f v)$ and $F_{hy} (= - fv)$ =Coriolis’ force respectively; $f (= 2\omega \sin \theta)$=coefficient of Coriolis’ force, $\omega$=earth’s angular velocity.

From the Chezy’ formula, the expression $\tau_s = \rho g \sqrt{V_h} \frac{h^{1/3}}{R^{1/3}}$ can be deduced. For the wide and shallow river $R \approx h$, therefore the resistance at the riverbed can be written as:

$$\tau_{bs} = \rho g n_b^2 u \sqrt{u^2 + v^2} / h^{1/3}, \quad \tau_{by} = \rho g n_b^2 v \sqrt{u^2 + v^2} / h^{1/3} \tag{4}$$

If ice cover presents, the resistance underside of ice-cover and the resistance at the riverbed are respectively:

$$\tau_{ix} + \tau_{bx} = \rho g n_c^2 u \sqrt{u^2 + v^2} / h^{1/3}, \quad \tau_{iy} + \tau_{by} = \rho g n_c^2 v \sqrt{u^2 + v^2} / h^{1/3} \tag{5}$$

where, $n_b$=riverbed’ roughness; $n_c$=composite Manning’s roughness which can be calculated by following formula $\left(\text{Mao, Ma and She et al., 2002}\right)$

$$n_c = [(P_b)^{1/2} + P_i n_i^{1/2})/P]^{2/3} \tag{6}$$

where, $n_i$=roughness of ice cover; $P_b$, $P_i$=wet perimeter of flow and ice cover respectively, $P = P_b + P_i$.

### 2.2 Water Temperature

Two-dimensional unsteady water temperature in terms of conservation of thermal energy:
\[ \frac{\partial}{\partial t} (\rho C_p T) + \frac{\partial}{\partial x} (Q \rho C_p T) + \frac{\partial}{\partial y} (Q \rho C_p T) = \frac{\partial}{\partial x} \left( AE_{\rho} \rho C_p \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left( AE_{\rho} \rho C_p \frac{\partial T}{\partial y} \right) + B \Sigma S \]  

where \( T \)=vertical average water temperature; \( C_p \)=specific heat of; \( A \)=sectional area of flow; \( B \)=water-surface width; \( E_x \), \( E_y \)=turbulent dispersion coefficient in x- and y-direction respectively; \( \Sigma S \)=net heat flux per unit flow surface. For the sake of simplification, assuming \( E_x = E_y = E \) and flow discharge remaining constant, the above equation can be re-written as

\[ \frac{\partial T}{\partial t} + u \frac{\partial T}{\partial x} + v \frac{\partial T}{\partial y} = E \left( \frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} \right) + \frac{B}{\rho C_p A} \Sigma S \]  

2.3 Surface-ice and Suspended Frazil-ice

According to two-layer ice transport theory, ice run can be divided to the surface-ice and the suspended frazil-ice in the ice-free river reach. The equations for surface and suspended ice are respectively (Wu, 2003):

\[ \frac{\partial C_s}{\partial t} + u \frac{\partial C_s}{\partial x} + v \frac{\partial C_s}{\partial y} = -\frac{B}{\rho_i L_i A} \Sigma S + \frac{\alpha}{A} \left( 1 - \frac{V_s}{u_i} \right) C_c \]  

\[ \frac{\partial C_s}{\partial t} + u \frac{\partial C_s}{\partial x} + v \frac{\partial C_s}{\partial y} = -\frac{B}{\rho_i L_i A} \Sigma S - \frac{\alpha}{A} \left( 1 - \frac{V_s}{u_i} \right) C_c \]  

where \( C_s \)=volumetric concentration of surface-ice layer; \( C_c \)=volumetric concentration of suspended frazil-ice; \( L_i \)=latent heat of water fusion; \( \alpha \)=an empirical coefficient quantifying the rate of supply to the surface ice from suspended frazil ice; buoyant velocity of ice particles \( u_i = -0.025(1 + 2.5 / 1130) + 0.005 \),\( \)where \( T \)=average temperature of water-surface; \( V_s = u_s \sqrt{g / [C(0.7C + 6.0)] / 5} \), \( C \)=Chezy’s coefficient.

2.4 Ice-cover Progression

According to the ice coming from upstream and the hydraulic condition, there are three kinds of dynamic ice-cover formation modes, i.e. juxtaposition mode, hydraulic thickening (narrow jam) mode and mechanical thickening (wide jam) mode. From the mass conservation of surface ice at the leading edge, the rate of ice-cover progression is:

\[ V_p = \frac{Q_{ij} - Q_u}{B_u t_s (1 - e_s) - (Q_s - Q_u) / V_{scp}} \]  

\[ \text{[11]} \]
where, $B_o$=top width of open water between border ice; $e_o = e_r + (1 - e_r)$, $e_r$ = overall porosity of ice cover, where $e_r$ = porosity between ice floes, $e_p$ = porosity of an ice floe; $Q_i$ = volumetric rate of ice entrainment under the cover at the leading edge (m$^3$/s); $Q_s$ = volumetric rate of ice discharge in surface ice layers (m$^3$/s); $V_{scp}$ = average velocity of the incoming surface ice. The node-isolation method is applied in computation of the progression distance of each grid point at the leading edge.

When the Froude number is less than that calculated by the following formula, the ice-cover will progress to upstream by juxtaposition mode. By contrast, if the Froude number of flow is greater than the calculated value, the ice coming from upstream will entrain under the ice-cover, resulting in narrow jam, and the thickness of ice-cover can be get from the balance condition.

\[
Fr_c = 2 \sqrt{\frac{(1 - \rho_i / \rho) t_o / d - (1 - t_o / d)}{5 - 3(1 - t_o / d)}}
\]  

[12]

Where, $d$=water depth at the leading edge; $t_o$=thickness of floe. When the Froude number exceeds $Fr_c$ single floe juxtaposition can not be maintained. Then, ice-cover will progress to upstream by hydraulic thickening (narrow jam) mode, inducing the thickness of ice-cover increasing, water level at the leading edge rising and flow at bottom separating. The change of thickness of ice-cover during the period of $\Delta t$ is:

\[
\Delta h = \frac{Q_s t}{B_o \Delta x}
\]

[13]

As the thickness of ice-cover at the leading edge increasing, the Froude number gradually declines. When less than $Fr_c$ the leading edge will progress again. If forces acting on the ice-cover exceed the bank shear, mechanical thickening will be formed, and its balance thickness can be calculated:

\[
\frac{B_v}{\mu C h^2} \left(1 + \frac{\rho_i}{\rho} \right) + \frac{2 \tau_i}{\rho \mu h^2} \left(1 - \frac{\rho_i}{\rho} \right) = \frac{B_o \tau_{bc}}{\rho \mu h^2} + \frac{2 \tau_o}{\rho \mu h^2} + \frac{\rho_i (1 - \rho_i)}{\rho} t^2
\]

[14]

Where, $V_i$ = velocity under the ice-cover; $\mu$ = friction coefficient between floes; $R_o$ = hydraulic radius; $\tau_i$ = cohesion term of the bank shear.

2.5 Ice Transport under Ice-cover

The ice transport capacity is formulated in the following relationship (Shen and Wang, 1995):

\[
\frac{B_o V^2}{\mu C h^2} = \frac{B_o \tau_{bc}}{\rho \mu h^2} + \frac{2 \tau_o}{\rho \mu h^2} + \frac{\rho_i (1 - \rho_i)}{\rho} t^2
\]
\[ \phi = 5.487 (\Theta - \Theta_c)^{1.4} \quad [15] \]

in which, \( \phi = \frac{q_i}{d_n F \sqrt{g d \Delta}} \) = dimensionless ice transport capacity; \( \Theta = \frac{V_{ci}}{F' g d \Delta} \) = dimensionless flow strength; \( \Theta_c \) = dimensionless critical shear stress; \( q_i \) = volumetric ice discharge per width; \( V_{ci} \) = shear velocity on the underside of ice-cover; \( F' \) = fall velocity coefficient; \( d_n \) = nominal diameter of ice particles.

**2.6 Width of Border Ice**

If \( V_s \) satisfies with the following condition border ice will be initially formed:

\[ V_s < \frac{\sum S}{1130 (-1.1 - T)} - \frac{15 w_n}{1130} \quad [16] \]

The border ice will progress in width direction due to floes congregating. The progression depends on stabilization between floes and border ice and calculated as followings:

\[ \Delta W = \frac{14.1 \sum S}{\rho L_i} \left( \frac{u}{V_c} \right)^{-0.93} N^{1.08} \quad [17] \]

where, \( V_c \) = maximum allowance velocity at which a floe can adhere to the existing border ice; \( \Delta W \) = the increased width of border ice during a given period; \( N \) = surface-ice concentration.

**3. Boundary-fitted-coordinate System**

As Fig.2 shows, solution of elliptical partial-differential equations results in corresponding transformation of coordinates \( \xi = \xi(x, y) \), \( \eta = \eta(x, y) \), which satisfy with the Dirichlet boundary condition and the following equations:

\[ \frac{\partial^2 \xi}{\partial x^2} + \frac{\partial^2 \xi}{\partial y^2} = P(\xi, \eta), \quad \frac{\partial^2 \eta}{\partial x^2} + \frac{\partial^2 \eta}{\partial y^2} = Q(\xi, \eta) \quad [18] \]

where \( P \) and \( Q \) are both continuous function of \( \xi, \eta \). By selecting proper functions of \( P \) and \( Q \) the curvilinear grids with various density at \( x-y \) plane can be transformed into uniform rectangular grids at \( \xi - \eta \) plane. The procedure of BFC creation includes arrangement of grid points along boundaries of region D on \( x-y \) plane in terms of requirements. As shown in Fig.1(a), there are \( M \) nodes along boundary \( \Gamma_1 \) and \( \Gamma_3 \), \( N \) nodes along boundary \( \Gamma_2 \) and \( \Gamma_4 \), and the spacing between those nodes can be unequal; Nodes at boundaries of \( D^' \) region are corresponding with
nodes at boundaries of \( D \) region; Numerically solving the following equations by using the central difference scheme:

\[
\begin{align*}
\alpha x_{\xi\xi} - 2\beta x_{\eta\eta} + \gamma x_{\xi\eta} + J^2 (P x_{\xi} + Q x_{\eta}) &= 0 \quad [19] \\
\alpha y_{\xi\xi} - 2\beta y_{\eta\eta} + \gamma y_{\xi\eta} + J^2 (P y_{\xi} + Q y_{\eta}) &= 0 \quad [20]
\end{align*}
\]

in which, \( \alpha = x_{\eta}^2 + y_{\eta}^2 \), \( \beta = x_{\xi} x_{\eta} + y_{\xi} y_{\eta} \), \( \gamma = x_{\xi}^2 + y_{\xi}^2 \), \( J = x_{\xi} y_{\eta} - x_{\eta} y_{\xi} \).

4. Transformation and Computational of Basic Equations

Let \( \xi = \xi \cdot J \), \( \eta = \eta \cdot J \), \( u_{\xi} = u_{\xi_{\eta}} + v_{\xi_{\eta}} \), \( v_{\eta} = u_{\eta_{\eta}} + v_{\eta_{\eta}} \), \( q_{\xi} = q_{\xi_{\eta}} + q_{\xi_{\xi}} \), \( q_{\eta} = q_{\eta_{\xi}} + q_{\eta_{\eta}} \), in which \( u_{\xi} \) and \( u_{\eta} \) are velocity components in \( \xi - \eta \) coordinate respectively.

The governing equations under BFC system can be deduced through coordinate transformation of equations [1]–[3]:

\[
\frac{\partial h}{\partial t} + \frac{1}{J} \frac{\partial q_{\xi}}{\partial \xi} + \frac{1}{J} \frac{\partial q_{\eta}}{\partial \eta} = 0 \quad [21]
\]

\[
\frac{\partial q_{\xi}}{\partial t} + \frac{1}{J} \frac{\partial q_{\xi}}{\partial \xi} \left( \frac{q_{\xi}}{h} + \frac{\xi}{2} \right) + \frac{1}{J} \frac{\partial q_{\eta}}{\partial \eta} \left( \frac{q_{\eta}}{h} + \frac{\eta}{2} \right) = b_x \quad [22]
\]

\[
\frac{\partial q_{\eta}}{\partial t} + \frac{1}{J} \frac{\partial q_{\xi}}{\partial \xi} \left( \frac{q_{\xi}}{h} + \frac{\xi}{2} \right) + \frac{1}{J} \frac{\partial q_{\eta}}{\partial \eta} \left( \frac{q_{\eta}}{h} + \frac{\eta}{2} \right) = b_y \quad [23]
\]

If boundary conditions is generally expressed as \( A \phi + B \partial \eta / \partial n = C \), in which \( A, B, \) and \( C \) are given, \( \partial \eta / \partial n \) is normal derivative at boundary, \( \phi \) is a variable desired, gradient of function \( f \) is \( \nabla f = (f_{\xi_{\eta}} + f_{\eta_{\eta}}) \xi_{\eta} + (f_{\xi_{\xi}} + f_{\eta_{\eta}}) \xi_{\xi} \). Let \( f = \xi, f = \eta \), the transformed forms of boundary conditions can be obtained as:

\[
\frac{\partial \phi}{\partial n} = \frac{1}{\sqrt{q_{11}}} (q_{11} \frac{\partial \phi}{\partial \xi} + q_{12} \frac{\partial \phi}{\partial \eta}), \quad \frac{\partial \phi}{\partial n^{(\alpha)}} = \frac{1}{\sqrt{q_{12}}} (q_{12} \frac{\partial \phi}{\partial \xi} + q_{22} \frac{\partial \phi}{\partial \eta}) \quad [24]
\]

in which, \( q_{11} = \xi_{\eta}^2 + \xi_{\xi}^2 = x_{\xi}^2 + y_{\xi}^2 = \alpha \); \( q_{12} = \xi_{\xi} \xi_{\eta} + \xi_{\eta} \xi_{\eta} = -\beta \); \( q_{22} = \eta_{\xi}^2 + \eta_{\eta}^2 = \gamma \). Similarly, by transformation of equations [8]–[10] the governing equations of water temperature, concentration of surface and frazil ice under BFC can be deduced:

197
\[
\frac{\partial T}{\partial t} + \frac{1}{J} \frac{\partial}{\partial \xi} \left[ u_s T - \frac{E}{J} \left( q_{x,1} \frac{\partial T}{\partial \xi} + q_{x,2} \frac{\partial T}{\partial \eta} \right) \right] + \frac{1}{J} \frac{\partial}{\partial \eta} \left[ v_s T - \frac{E}{J} \left( q_{y,1} \frac{\partial T}{\partial \xi} + q_{y,2} \frac{\partial T}{\partial \eta} \right) \right] = \frac{B}{\rho c_p A} \sum S
\]

\[25\]

\[
\frac{\partial C_s}{\partial t} + \frac{1}{J} \left[ \frac{\partial (u_c C_s)}{\partial \xi} + \frac{\partial (v_c C_s)}{\partial \eta} \right] = - \frac{B}{\rho L_c} S + \frac{\alpha}{A} (1 - \frac{V}{u_i}) C_c
\]

\[26\]

\[
\frac{\partial C_c}{\partial t} + \frac{1}{J} \left[ \frac{\partial (u_c C_c)}{\partial \xi} + \frac{\partial (v_c C_c)}{\partial \eta} \right] = - \frac{B}{\rho L_c} S - \frac{\alpha}{A} (1 - \frac{V}{u_i}) C_c
\]

\[27\]

![Figure 2. Transformation of BFC method](image)

![Figure 3. Discrete scheme](image)

As illustrated in Fig.3, ABCD=control volume, (i, j) =control node, \(\Delta \xi \Delta \eta\) =control area. Equations [21]-[23] and equations [25]-[27] can be expressed in the following vector form:

\[
U_i \frac{1}{J} F_\xi + \frac{1}{J} G_\eta = b
\]

\[28\]

and its discrete form:

\[
U_{i,j}^{n+1} = U_{i,j}^n - \frac{\Delta t}{\Delta \xi \Delta \eta} \left( \frac{1}{J_{AB}} H_{AB} + \frac{1}{J_{BC}} H_{BC} + \frac{1}{J_{CD}} H_{CD} + \frac{1}{J_{DA}} H_{DA} \right) + \Delta t b^n
\]

\[29\]

where \(H_{AB} = F_{AB} \Delta \eta\), \(H_{BC} = G_{BC} \Delta \xi\), \(H_{CD} = -F_{CD} \Delta \eta\), \(H_{DA} = H_{DA} \Delta \xi\). Two-step MacCormack scheme is used to solve the above equations. In order to assure symmetry of computation, stagger arrangement of forward and backward difference is adopted at i- and j-direction control surface. Node variables are staggerly arranged, which is helpful to solve the moving boundary conditions.

5. Model Test

5.1 Introduction about Hequ Section of Yellow River
Field observation (Headquarters, 1993) in the Hequ section of the Yellow River is selected to validate the above numerical model. The sketch of the Hequ reach and its relevant characteristics are given in Fig.4 and Table1. The numerical simulation of freeze-up process is conducted in the reach from Longkou to Yumiao, with length of 58.5km (19 sections in total), and the computations is compared with field observations. Characteristic of each section are shown in Table 1. The period of time simulated is from 8:00 am of Nov.26 to 8:00 am of Nov. 29 in 1986, which is three days. The time-step is 900s.

Table 1. Characteristics of cross sections of Hequ Reach

<table>
<thead>
<tr>
<th>Section</th>
<th>Distance from Longkou (m)</th>
<th>Side-slope coefficient</th>
<th>bottom elevation (m)</th>
<th>bottom width (m)</th>
<th>roughness coefficient</th>
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<td>Longkou</td>
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<td>Yingzhantan</td>
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<td>100</td>
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</tbody>
</table>

5.2 Grid Division

There are many crankle and alluvion in Hequ branch, so boundary-fitted grid is used. The alluvion divide river into several sub-river, and the boundary of adjacent sub-river is composed of the boundary of alluvion and public water boundary. The whole computational area is of multi communicating zones. For every sub-river the structure-type grid is established by using structural-form joining method. During the computational, calculating on the local refined grid and initial grid is conducted separately with same method. First, selecting the area of original grid needed be local refined, then dividing every selected grid into (D1×D1) local refined grid, in
which \( DI = \) grid refined coefficient in \( x \)-direction, \( DJ = \) grid refined coefficient in \( y \)-direction. In this paper, \( DI = DJ = 4 \). The computational area is divided into three segments, i.e. open flow segment, progression segment and ice-cover segment. The grid was re-divided depending on the progressive distance of the leading edge. Let \( x = \) river distance direction, \( y = \) width direction, origin lies on the right bank of Longkou section. Initial BFC grid is divided into \( 1000 \times 20 \). After grid refined, the maximum grid size is 80.5m and the minimum grid size is 2.6m.

### 5.3 Calculation and Result Analysis

The computations of water-level with field observations during the freeze-up period are compared as shown in Fig.5. It is obvious that water level at the leading edge of ice-cover increases as ice-cover advanced. Water-level variation along the width are shown in Fig.5.

#### Figure 5. Variation of water level with distance

(a) Variation of water level (Nov.27)

(b) Variation of water level (Nov.28)

(c) Variation of water level (Nov.29)

#### Figure 6. Water level of cross section changing with \( y \) direction

(a) Water level of Yangmian (Nov.27)

(b) Water level of Hehui (Nov.28)

(c) Water level of Longkou (Nov.29)
Fig. 7 and Fig. 8 give the comparison of the position of leading edge of ice cover between computations and field observations. Obviously, they both agree well, and it can be found that the progression speed of ice cover is not uniform. The main reason may be that floe concentration at leading edge of ice cover is largely affected by air temperature.

![Figure 7. Averaged freezing-fringe position](image1.png)

![Figure 8. Positions of freezing fringe](image2.png)

Fig. 9- Fig. 11 give the comparison of thickness of ice-cover between computations and field observations. From those figures, it can be seen that during ice-cover progression the thickness of ice-cover is not equal. The main reason is that geometric characteristic of each section, incoming ice and flow condition are not the same when ice cover initial formation, and the ice transportation under ice cover make the thickness of downstream ice cover changed. Besides, the result of research show that mechanical thickening mode occur during progression of ice cover, and the reason is that width of the section is much larger than depth at crankle.

![Figure 9. Ice-cover thickness in Shitizi-Yangmian Reach (Nov.27)](image3.png)

![Figure 10. Ice-cover thickness in Quyu-Hehui Reach (Nov.28)](image4.png)

![Figure 11. Ice-cover thickness in Yingzhantan-Longkou Reach (Nov.29)](image5.png)

6. CONCLUSIONS
To accurately simulate the complicated boundary conditions in natural rivers, a two-dimensional numerical model of river-ice process is developed, based upon boundary-fitted coordinate. The MacCormack scheme is used to solve the transformed equations. The presented model is validated by the field observations and satisfactory results are achieved. The study shows that the progression of ice-cover leading edge and the thickness distribution of ice cover are affected by
many factors, and water level at the leading edge of ice-cover increase while the leading the edge is advancing upstream. The 2-D numerical model presented is capable of simulating these characteristics.

Acknowledgments

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References


Finite Element Analysis of Nonlinear Ice-induced Vibration Response of Ocean Structures

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Using finite element method and a mathematical model based on the concept of inertial force, this paper mentioned equations of motion for system. In this paper, based on the results from the numerical calculation, the changes of the ice-induced vibration response of two-dimensional ocean structures have been discussed with different profile artificial boundary, value scope of finite structure region and damping coefficient of structural medium within the linear-elasticity range. Using a hysteresis damping assumptions, the paper has discussed on how to form the damping matrix by hysteresis damping coefficient in the time domain analysis. It displayed a formula of value from damping coefficient conversion frequency.
1. Introduction

The northern ocean waters are cold frozen in China. As the national economy develops, the form and number of ocean structures is an increasing, such as bridges, oil platforms. Anti-ice-induced vibration problem has been paying close attention. Because of the interaction between sea-ice and structures, the sea ice-induced vibration analysis of the structures is more complex. The research is very difficulty.

The ice-induced vibration analysis of ocean structure in ocean engineering design is a very important technical work, especially the ocean engineering on the northern cold coastal waters. Ice-induced vibration analysis on the basis of the results of the ice-induced vibration safety evaluation of the structures display the numerical value of ultimate ice-induced vibration of safety, security operational ice-induced vibration, response spectrum for design and ice-induced vibration time interval. On the one hand, Anti-ice-induced vibration can be designed according to it. On the other hand we can combine the structural characteristics to calculate the structural spectrum (e.g., Gao Zhaojie et al. 1998). The benchmark parameters of anti-ice-vibration design can be provided by it for the various parts of structures.

For general engineering, a simplified one-dimensional plane model calculation will satisfy requirements of ice-induced vibration design of ocean engineering. But for major projects such as bridge and oil platforms, one-dimensional simplified calculation model has been unable to meet the requirements of calculation. Hence it need for two-dimensional finite element calculation. This research and calculation is still in the exploratory stage (e.g., Shi Qingzeng et al. 2001). The ice-induced vibration analysis and design method is adopted by inertial force method and the two-dimensional method of profile excitation in the current. It does not take into account factors of different direction of vibration excitation from sea ice, and the diversity of structural forms. In the actual severe ice vibration process, the structure has come into the nonlinear stage (e.g., Huang Yan et al. 2004). In this paper we consider to use two-dimensional finite element model to explore this problem. We consider the following two main issues:

When the ice-induced vibration of interaction between ice and structures is analyzed by using the mathematical models of inertial force method and finite element method, sea ice and structures are discrete by directly using units. Because the structure is a finite object, it is a relatively simple that the structure be divided into finite element. But sea ice is actually a semi-infinite objects, the analysis can only be calculated on the sea ice within a limited range (e.g., Duan Menglan et al. 2000). Therefore, it must involve the selection of sea-ice extent and installation of artificial boundary. Because most of the existing artificial boundary is based on the sources within the incentive vibration, therefore, ice-induced vibration wave of exogenous vibration, usually the effectiveness of artificial boundary is a concern. This issue include: On the one hand, because the most of the existing artificial boundary is based on the sources within the incentive vibration, therefore, usually the effectiveness of artificial boundary is a matter of concern for ice-induced vibration wave of exogenous vibration. On the other hand, the result of the calculation choosing how much sea ice extent can meet the needs of engineering and how much effect of the ice-induced vibration response by ice on the structures (e.g., Jin Xiaoding et al. 1994).

When we analyze certainty ice vibration response, there are usually two analytical methods. It is time-domain analysis method and frequency domain analysis method. In the time domain
analysis, viscous damping model is often used to calculate the damping of the structural medium (e.g., Huang Yan et al. 2005). In the frequency domain analysis, viscous damping model or hysteresis damping model are often used to calculate the damping of structural medium. In structural dynamics, hysteresis damping model is often used to express the damping characteristics of the structural medium, and to get corresponding hysteresis damping coefficient through the samples test of the structural material (e.g., Shi Qingzeng et al. 2002). The composite modulus be used for the structural medium, we can carry out analysis in the frequency domain. In the actual severe ice vibration process, the structure has come into the nonlinear stage. At this time, we should use time-domain nonlinear analysis method. Therefore, we should research that hysteresis damping coefficient is applied to the time domain analysis.

2. Basic Assumption
In this paper, we will use the inertial force mathematical models and finite element method to present motion equations of the structural system. In the ice-induced vibration numerical analysis of ocean structures, we have made the following assumptions.

2.1 Assumption I
In the process of ice vibration, sea-ice motion is a base level of the sea surface. It makes the structure has a relative motion to the base level.

2.2 Assumption II
In the ice vibration process, if the structural vibration on the base level leads in the vibration of the upper and lower structure on base level. Structural vibration wave from the reverse transfer process of the upper and lower structure will generate a harmonic vibration. So the base level above references should be considered in the selection of the largest relative displacement surface because of the structural medium harmonic vibration lead to.

2.3 Assumption III
As the ice vibration process, the motion of structural medium for the base level will generate the vibration inertial force, then the vibration inertial force transfers into the node force on the various nodes of the discrete structure. This vibration force makes the structure to generate the corresponding reaction (acceleration, displacement, stress and strain).

2.4 Assumption IV
In order to determine the inertial force of the ice vibration, various vibration acceleration records are usually used according to sea ice circumstances of different waters. The simulation of ice vibration wave and artificial ice vibration wave can also be used.

2.5 Assumption V
As a result of the difference of structural shape, such as length, width and high, the structural shape clearly affects the ice vibration response. This effect can be considered in the discrete finite element.

3. Motion Equation of Structural System and Analytical Method
The reaction analysis of the foundation piers of cross-sea bridges and oil platforms in ice vibration is the dynamics parameters of motion displacement, velocity, acceleration, stress and
strain of medium of concrete piers when the undulating waves caused collision between sea-ice and piers (e.g., Ou Jinping et al. 1999). When it is analyzed, we must consider of the dynamic characteristics, geometry size, ice vibration characteristics and boundary conditions of concrete piers.

3.1 The Numerical Analysis Model of Nonlinear Ice Vibration Response of Pier
The recursion schemes method of ice-induced vibration numerical analysis is the gradual integral method in time domain (e.g., Yue Qianjin et al. 2000). We can achieve the ice-induced vibration analysis of piers by Matlab.

3.1.1 The Mechanics Model of Pier Structures
Firstly, we divide a pier into many concrete structural layers. The mechanics model of each layer can be simplified for two-dimensional shear model of layers. The model parameters are made of mechanics parameters of layers. The assumption is used by a discrete-focus mass method for mass of pillars structural layers. Focus mass is \( (m_1, m_2, \ldots, m_n) \). The ice-induced vibration analysis model of structural layers can be expressed as in formula [1].

\[
m_i = \frac{1}{2} \rho h_i, \quad m_i = \frac{1}{2} \rho h_{i-1} + \frac{1}{2} \rho h_i
\]

Where: \( \rho \) _ Mass density of reinforced concrete; \( h \) _ Thickness of structural layers.

The mass is linked by the spring of anti-profile deformation. The spring constant is as in formula [2].

\[
k_i = \frac{G}{h_i} \quad (i=1, 2, \ldots, n)
\]

Where: \( G \) _ The shear modulus of structural layers.

3.1.2 The Motion Equations of Pier Structural Layer
Based on the aforementioned assumptions, under the ice vibration loads, the pier in deep sea will be show the characteristic of linear-elasticity multi-degree of freedom (DOF) system. The motion equation of finite element system at the moment \( t \) is as in formula [3].

\[
M \ddot{x}(t) + C \dot{x}(t) + Kx(t) = -MI \ddot{x}_g(t)
\]

Where: \( M, C \) and \( K \) _ the mass matrix, damping matrix and stiffness matrix of system, all is constant phalanx \( (n \times n) \). ‘n’ is the freedom degree of ice layer system.

\( \ddot{x}(t), \dot{x}(t) \) and \( x(t) \) _ Relative acceleration, velocity, node displacement vector

\( x_g(t) \) _ Outside force acceleration of ice vibration

\( I \) _ Unit vector of excitation vibration
When the ice vibration is respectively along with horizontal vibration of x-axis and vertical vibration of y-axis:

\[ I_x = [1, 0, 1, 0, \ldots, 1, 0]^T \quad I_y = [0, 1, 0, 1, \ldots, 0, 1]^T \]

The Rayleigh damping is used in stiffness matrix system:

\[ C = \alpha M + \beta K \]

Where: \( \alpha, \beta \) - damping constant, following in formula [4]

\[ \alpha + \beta \omega_i^2 = 2 \omega_i \xi_i \quad \alpha + \beta \omega_j^2 = 2 \omega_j \xi_j \]  \[ \text{[4]} \]

Where: \( \omega_i, \omega_j, \xi_i, \xi_j \) - vibration vector respectively

According to vibration type analysis, two kinds of the larger "contributions" angular frequency of the self-vibration and damping ratio corresponding to vibration type is used to calculate value of \( \alpha, \beta \).

### 3.2 The Analytical Recursion Schemes Method of Ice-induced Vibration Response Analysis of Pier Structural Layer

Under the complex external force \((ML\dot{x}(t))\), usually the formula [3] can be solved by gradually numerical integral equation. The recursion schemes of the algorithm are as in [5].

\[ Y_{i+1}(t) = A_i Y_i + L_i F_i \]  \[ \text{[5]} \]

Where: \( Y_i \) - Initial vector \( Y_i = \{x_{i+1}, x_i\}^T \);
\( Y_{i+1} \) - Solution vector; \( A_i \) - Genetic operators; \( F_i \) - Load vector; \( L_i \) - Load operators

The recursion scheme has the single-step method and the two-step method. There are three solution states in the single-step method. It is as in formula [6].

\[ Y_i = \left\{ x_i, \dot{x}_i \right\}^T \quad Y_i = \left\{ x_i, \dot{x}_i \right\}^T \quad Y_i = \left\{ x_i, \ddot{x}_i \right\}^T \]  \[ \text{[6]} \]

There are also three solution states in the two-step method. It is as in formula [7].

\[ Y_i = \left\{ x_{i+1}, x_i \right\}^T \quad Y_i = \left\{ \dot{x}_{i+1}, \dot{x}_i \right\}^T \quad Y_i = \left\{ \ddot{x}_{i+1}, \ddot{x}_i \right\}^T \]  \[ \text{[7]} \]

Based on the assumption of linear-elasticity conditions in a time interval, \( A_i \) and \( L_i \) in formula [5] are the analytical genetic operators respectively and the analytical load operators.
In this paper, the solution state of two-step method is used for calculation. It is

\[ Y_i = \{ x_{i+1}, x_i \}^T \]

In this paper, the single-step solution state is used to reckon the analytical recursion schemes of two-step method. The analytical genetic operator can be divided into four equal parts. It is as in formula [8].

\[ A_i = \begin{bmatrix} A_{11}(i) & A_{12}(i) \\ A_{21}(i) & A_{22}(i) \end{bmatrix} \]

According to the single-step method, we can solve the initial solution of state equation.

\[ Y_i = \{ x_i, x_i \}^T, \quad \dot{Y}(t) = \alpha Y(t) + p(t) \]

\[ Y(t) = e^{\alpha(t-t_i)}Y_i + \int_{t_i}^t e^{\alpha(t-\tau)} p(\tau) d\tau \]

\[ Y_{i+1} = L_i \Delta f_i = [\alpha^{-1} A_i - \alpha^{-1}] \begin{bmatrix} 0 \\ M^{-1} \end{bmatrix} \frac{\Delta f_i}{\Delta t_i} \]

\( \Delta f_i \) _The difference of time interval of load input. It is \( \Delta f_i = f_{i+1} - f_i \), \( \Delta t_i = t_{i+1} - t_i \).

We can get the formula [9].

\[
\begin{aligned}
\dot{x}_{i+1} &= A_{11}(i) \ddot{x}_i + A_{12}(i) \dot{x}_i + [I - A_{11}(i)K^{-1}] \frac{\Delta f_i}{\Delta t_i} \\
\ddot{x}_{i+1} &= A_{21}(i) \dot{x}_i + A_{22}(i) \ddot{x}_i + A_{12}(i)M^{-1} \frac{\Delta f_i}{\Delta t_i} \\
x_{i+2} &= A_{11}(i+1) \ddot{x}_{i+1} + A_{12}(i+1) \dot{x}_{i+1} + [I - A_{11}(i+1)K^{-1}] \frac{\Delta f_{i+1}}{\Delta t_{i+1}} \\
\ddot{x}_{i+2} &= A_{21}(i+1) \dot{x}_{i+1} + A_{22}(i+1) \ddot{x}_{i+1} + A_{12}(i+1)M^{-1} \frac{\Delta f_{i+1}}{\Delta t_{i+1}}
\end{aligned}
\]

We can eliminate \( \ddot{x}_i \) and \( \ddot{x}_{i+1} \). We can get the analytical recursion schemes of two-step method.

\[
\begin{aligned}
\ddot{x}_{i+2} &= A_{11}'(i+1) \ddot{x}_{i+1} + A_{12}'(i+1) \dot{x}_{i+1} + A_{12}(i+1)M^{-1} \left\{ \frac{\Delta f_{i+1}}{\Delta t_{i+1}} - \frac{\Delta f_i}{\Delta t_i} \right\} \\
A_{11}'(i) &= A_{21}(i+1)A_{11}(i)A_{21}'(i) + A_{22}(i+1) \\
A_{12}'(i) &= A_{21}(i+1)[A_{12}(i) - A_{11}(i)A_{21}'(i)A_{22}(i)]
\end{aligned}
\]
4. The Impact of Profile Artificial Boundary in Ice Vibration Response Analysis of Structures

When the finite element method is used in the analysis, the semi-infinite domain structures must be intercept, and make into a finite domain. Result of this calculation is the actual ice response approximate solution. Such as the level structural layer of the same thickness, we can get its analytical solution through the wave equation. For the same structural layer, we can analyze the accuracy of approximate solution in the finite element method through comparing the error between the fluctuation solutions and finite element solution.

For the finite element solution, the slenderness ratio \( l/d \) of structural layer may be different values. By comparing the magnification coefficient of the structural layer response and fluctuation solution, the relative error of finite element solution can be expressed as in formula [10].

\[
\frac{\beta_v - \beta_v^*}{\beta_v^*} \times 100\% = e
\]

[10]

In the analysis we use dimensionless parameters to describe the impact factor of the relative error.

Dimensionless frequency: \( \bar{\omega} = \omega c_s / d \)

\( \xi \) _ Hysteresis damping coefficient of structural layer medium

\( \omega \) _ Angular frequency

\( d \) _ Structural layer thickness

\( c_s \) _ Shear wave velocity

In the calculation, we choose \( l/d = 8 \) and \( l/d = 20 \) finite element solution. \( \xi = 0.1 \). Free boundary is used to finite element artificial boundary. When \( l/d = 8 \), the results show that has a significant difference between the finite element solution and fluctuation solution. With \( l/d \) increase, finite element solution will converge to the fluctuation solution. When \( l/d = 20 \), the same result will be obtained by the finite element solution and fluctuation solution. The absolute error peak of reaction amplification factor of structural layer between the finite element solution and fluctuation solution display on the point that frequency is \( \omega = \pi / 2 \). It is fundamental frequency of rigid structural layer in half-space.

The actual ice-vibration wave can be made by the superposition of simple harmonic vibration of multi-frequency components. When the excitation frequency equal to the structural fundamental frequency, it is \( \omega = \pi / 2 \), amplification coefficient of structural reaction is the largest. It shows that the excitation frequency near the structural fundamental frequency play a major control role for the ice-induced vibration response.

In fact, ice-vibration wave is a complex spreading process. The spreading process of ice-vibration wave in structure causes the motion of structural layer. Therefore, the fluctuation difference between two points of the structural layer surface is not only related to structural layer, but also with the spreading state of ice-vibration wave in structure. Through the
assumption of the forward spreading of ice-vibration wave in structure, the numerical analysis method is used to analyze ice-induced vibration response of structure. The result of numerical analysis shows that at the same time of a horizontal motion of structural layer surface, it will have a greater vertical motion. The horizontal component and the vertical component of finite element solution is the same as the convergence speed. For a determinate damping coefficient, the major control factor is also the slenderness ratio \((l/d)\).

5. The Damping Matrix Analysis in Time Domain Analysis of Structural Layer

In the certainty ice-induced vibration response analysis of the structural layer, it usually has two method, they are time-domain analysis and frequency-domain analysis. In time-domain analysis, the viscous damping assumption is usually used. The damping force and the velocity of particle motion are proportional. However, structure has entered nonlinear stage in the actual process of severe ice-vibration. At this time, the calculation method of time-domain nonlinear analysis should be used. Therefore, in the time-domain analysis of nonlinear ice-vibration response of structural layer, it relates to the problem how to apply the viscous damping coefficient obtained in test to form the damping matrix. When the viscous damping model is used in structural layer, the relationship of damping matrix and stiffness matrix in the motion equation of finite domain may express for the formula \([11]\).

\[
[C] = \frac{2\xi}{\omega}[K] = \frac{\xi}{\pi f}[K]
\]

\(\omega, f\) _ Compelling vibration frequency of simple harmonic vibration load

When the simple harmonic vibration response is analyzed in the time domain, the damping matrix and compelling vibration frequency is relationship of inverse ratio. When the ice-vibration response is solved in the time domain, we use the hysteresis damping coefficient to express the viscous damping matrix ‘\([C]\)’, ‘\(\omega\)’ and ‘\(f\)’ is not compelling vibration frequency. We call it conversion frequency of damping coefficient ‘\(\omega_c\)’ and ‘\(f_c\)’. It seriously affects the rationality of ice-vibration calculation of structural layer. Therefore, when the hysteresis damping is used to analyze ice-vibration response of structural layer in time domain, the reasonable value of conversion frequency of damping coefficient will be key issue.

When the actual ice-vibration wave is analyzed, we commonly use the analytical method of sine wave to obtain the change curve of the maximum error of absolute acceleration on the structural layer vertex with the conversion frequency of damping coefficient. We can obtain the optimum fitting frequency ‘\(f_c\)’ of ice-vibration wave when the conversion frequency error equal to zero.

The mean value can be calculate for each structural layer according to the following formula \([12]\).

\[
\overline{f_c / f_i} = \frac{1}{n} \sum f_c / f_i , \quad \overline{f_c / f_k} = \frac{1}{n} \sum f_c / f_k
\]
Where: \( f_e / f_1 \) is exponential decay change with increases of fundamental frequency. \( f_e / f_a \) is basically a linear relationship with the fundamental frequency. Through regression analysis, we can get a fitting formula [13].

\[
\frac{f_{c1}}{f_1} = 1.0 + 2.4 \exp \left( -\frac{f_1 - 0.2}{0.16} \right) + 2.0 \exp \left( -\frac{f_1 - 0.2}{0.94} \right) \quad (0.2 \leq f_1 \leq 4.5)
\]

\[
\frac{f_{c2}}{f_a} = 0.26 + 0.2 f_1 \quad (0.2 \leq f_1 \leq 4.5)
\]  

\( f_1 \) Fundamental frequency of structural layer
\( f_{c1}, f_{c2} \) The forecast value of damping coefficient conversion frequency

According to the analysis results of the sine wave, if damping coefficient conversion frequency is higher than the optimum fitting frequency, the maximum of damping coefficient conversion frequency can be used as the final damping coefficient conversion frequency.

\[ f_c = \max (f_{c1}, f_{c2}) \]

6. The Method of Finite Element Mesh Partition

In the analysis of fluctuation issue, the discrete-continuous object will lead two adverse effects. They are low-pass effect and dispersion effect. The results showed that element size should meet the following relationship when the finite element mesh is plot along the spreading direction of wave.

\[ l \leq \left( \frac{1}{\pi} - \frac{1}{8} \right) \lambda_{\text{min}} \]

Where: \( \lambda_{\text{min}} \) Highest frequency in analysis process; \( c_{\text{smi}} \) Least shear velocity in alternating process.

The results showed that result of finite element solution will be different from the results of analytical solution with the increase of frequency. If the error of finite element solution is less than 5%, the finite element mesh is bigger, the smaller the value of the highest frequency. When the excitation frequency equals to the fundamental frequency of the structure, the response of structural layer is the largest. According to the results of calculation,

\[ l \leq \lambda_{\text{min}} / 8 \]

It can meet the actual needs of ice-induced vibration response.

7. Conclusions

In this paper, we used the inertial force mathematical models and finite element method to present motion equations of the structural system through basic assumptions, and mentioned the numerical analysis model of nonlinear ice vibration response of the pier, and described analytical recursion schemes method of the ice-induced vibration response analysis of pier structural layer.
The ice-induced vibration response of ocean structures was analyzed by Matlab. The following issues have been solved.

The two-dimensional finite element model has been used to explore the ice-induced vibration response of ocean structures. It has involved the selection of sea-ice extent and installation of artificial boundary. The horizontal component and the vertical component of finite element solution is the same as the convergence speed. For a determinate damping coefficient, the major control factor was the slenderness ratio \(l/d\).

The time-domain nonlinear analysis method was used in the ice-induced vibration response of ocean structures. We have studied that hysteresis damping coefficient is applied to the time domain analysis. According to the analysis results of the sine wave, if damping coefficient conversion frequency is higher than the optimum fitting frequency, the maximum of damping coefficient conversion frequency can be used as the final damping coefficient conversion frequency.

The method of finite element mesh partition has proposed. According to the results of calculation: \(l \leq \lambda_{min} / 8\). It can meet the actual needs of ice-induced vibration response.

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Numerical Implementation and Benchmark of Ice-Hull Interaction Model for Ship Manoeuvring Simulations

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Abstract
The first question about the performance of a ship operating in ice is usually about the speed-power relationship moving ahead in the specified ice conditions. With advanced physical model test technology and the increasing scope and reliability of full-scale data on reference ships, this question may be answered with considerable confidence at the design stage. Attention has turned in recent years to assessment of the performance of ships undertaking turns and more complex maneuvers in ice. Ability to predict turning performance is important in ship navigation, and it is essential as the basis for numerical models in marine simulators used for operator training and operations planning.

This paper reports on the development of a new physically based ice-hull interaction (IHI) model, developed at the NRC Institute for Ocean Technology. This model will serve as the key ice component for ship real-time simulators in ice. The model calculates forces generated by increments in an arbitrary prescribed ship motion. The model incorporates multi-failure ice modes and hydrodynamic effects, and tracks the development of the broken channel. The theoretical basis for the model has been described in a previous paper. This paper presents the model’s numerical implementation and benchmarking based on ship model tests in ice.
Two series of physical model experiments carried out at NRC-IOT provided the benchmarking data. The reference ships are the Terry Fox icebreaker and CCGS R-class icebreaker. The model tests included resistance measurements, constant radius turns and sinusoidal manoeuvres using a captive model mounted on a Planar Motion Mechanism (PMM). The operating conditions included open water, level ice and pre-sawn ice. Ship motions and ice loads during the manoeuvres were measured in tests.

The PMM physical model experiments were realistically simulated using IHI model software. The calculated ice forces on the hull, the ice contact during the manoeuvre and the width and shape of the channel formed were compared with physical measurements. The extensive experimental data sets enable verification of details of the mathematical model, as well as providing insight into the ship-ice interaction processes.

1. Introduction

Precise manoeuvring of a ship in ice is necessary in confined passageways and in the presence of navigation hazards. Navigation simulators, training simulators and autopilot systems are valuable tools to achieve precise control of a ship in a particular set of ice conditions. A new ice-hull interaction (IHI) model for the real-time simulations of ship manoeuvring in level ice was developed in NRC/IOT. The model will be integrated into the CMS training simulator as an ice force module in the original numerical framework. The model adopted an analytical approach with numerical implementation and was built on a detailed mechanical analysis of the hull-ice interaction in level ice. It considers the distributions of the breaking force, buoyancy force and clearing force using the corresponding theories. The adoption of the analytical approach yielded a short calculation time, which made the model suitable for real-time simulations. Since the forces were calculated at each new increment of any prescribed motion, the resulting simulation had the capacity to respond to arbitrary control inputs and hence arbitrary manoeuvres in ice.

The conception and the corresponding theories of the model were introduced in Lau et al (2004). This paper focuses on its numerical implementation and benchmarking based on the ship model test data obtained at IOT.

A stand-alone numerical software using MatLab language was developed, which provided an independent numerical platform for developing and benchmarking the ice-hull interaction model. The model was benchmarked against measurements from two PMM ship model test series carried out at IOT, Terry Fox model tests (Deradjji and van Thiel, 2004; Lau, 2006) and R-Class model tests (Hoffmann, 1998). The comparison and analysis of physical tests and simulation results also provided insight into the ship-ice interaction processes and clues for further refinement of the model.

2. Brief description of model theories

The model estimates the ice forces needed for simulating the ship steady manoeuvring in ice in time-domain. It neglects the high frequency ice force fluctuation and integrates ice force over a large time interval consisted of at least a few ice broken cycles to arise at an average local ice resistance.
The total ice forces on the hull are calculated by the vectorially sum of the contributions from three independent force components, breaking force, buoyancy force and clearing force, which respectively represent the corresponding physical phenomena during the ship breaking ice process.

\[ X_{\text{ice}} = X_{\text{break}} + X_{\text{buoy}} + X_{\text{clear}} \]  
\[ Y_{\text{ice}} = Y_{\text{break}} + Y_{\text{buoy}} + Y_{\text{clear}} \]  
\[ N_{\text{ice}} = N_{\text{break}} + N_{\text{buoy}} + N_{\text{clear}} \]

Where \( X, Y \) and \( N \) are the surge force, sway force and yaw moment respectively, and the subscripts “ice, break, buoy, and clear” refer to the total ice force, the ice breaking contribution, the ice buoyancy contribution and the ice clearing contribution respectively. The ice breaking forces components are mainly dependent on ice thickness, ice flexural strength, ice crushing strength, ice shear strength, ice elastic modulus, hull frame angles at waterline and ship velocity. The ice clearing force components are mainly dependent on ice thickness, hull wet surface and ship velocity. The buoyancy force component is mainly dependent on the ice-water density difference, hull wet surface and ice thickness.

The whole hull was divided into many segments and the ice forces on each segment were calculated based on the ice-hull contact area and ship motions. The global ice forces on the whole hull were obtained through vectorially adding these forces and moments. Figure 1 shows a sketch of global force and yaw moment calculation in IHI model.

### 2.1 Breaking force calculation

When the ship turns, more parts of the hull may contact the unbroken level ice. In the model, the ice-hull contact area was calculated based on the ice edge and ship motion and the channel was tracked in time through a simple house-keeping method. A multi-failure model considering the bending, crushing and shear failures was used to check the ice failure at the waterline of the hull from stem to stern and the failure mode that required the minimum failure force was selected. A cusp ice crack pattern consistent with a 3-D plate theory governed the flexural failure and the channel formation. The average size of the broken ice pieces depends on ship speed, ice thickness and its mechanical properties with reference to Varsta (1983), Enkvist (1972) and Lau et al (1999).

### 2.2 Clearing force calculation

The IHI model calculates the clearing force component by considering the force imposed by the motion of an ensemble of ice pieces rotating and sliding along the submerged surface of the hull. The clearing force component includes viscous drag and inherent buoyancy for the rotating ice floes, forces caused by wave pressure and ventilation of the rotating ice floes, and inertial forces due to ice acceleration. The IHI model adopts an energy method to calculate the force imposed by the motion of the ice mass during the ice floe turning process. The force due to ventilation, the static pressure and bow wave on the ice piece turning at water surface, is estimated according to Enkvist (1972) and Kotras et al. (1983).
2.3 Buoyancy Force Calculation
The buoyancy component represents the lifting force by the submerged ice pieces due to the density difference between the ice and water. A simplified flat-plate model representing the underwater surface of the ship hull was used for buoyancy force calculation as shown in Figure 3. The model estimates the ice volume on each wetted surface of the flat-plate model in time domain.

3. Numerical Implementation of the Model
The developed ice-hull integration model software simulates the ice forces on the hull due to user-specified ship motions. The software allows direct inputs of ship geometry, prescribed ship motions, ice mechanical properties, and the initial ice edge geometry, and compute ice loads on the hull, ice-hull contact area, and channel configuration. The IHI model software is designed as a Visual Calculation Program (VCP). During the calculation process, the users can instantaneously watch the simulation process and check the simulation results, such as ship’s motion, ice channel and calculated surge force, sway force and yaw moment on the hull imposed by the ice. It is important for the users to visualize the physical process of the ice-hull interaction and for the developer to refine the model. The software has flexibility and refinement spaces for future development of the IHI model. It also has a friendly data exchange connection, which makes it easy to be implemented into other numerical frameworks as an interior module. The whole IHI code consists of three layers, Parameters Input Layer (PIL), Core Calculation Layer (CCL) and Results Output Layer (ROL). Figure 2 shows the software structure for the IHI model and the associated main m-files.

Additional details of the model theories and numerical implementation can be found in Liu et al. (2006, 2007a, 2007b).

4. Benchmark of the Model
IOT has achieved a good correlation between the model test and sea trial results (Spencer and Jones, 2001; Jones and Lau, 2006). The data from the captive model tests can be directly used for calibrating and benchmarking the model. Two captive ship model tests, Terry Fox model tests and CCG R-class model tests, which were tested at IOT using a Planar Motion Mechanism (PMM), were selected in this paper. The tests runs were simulated using IHI model software and the benchmarking was carried out through comparing the test measurements and the simulation results.

4.1 Description of the Ship Models
IOT model 417 is a 1:21.8 scaled model of the Canadian Coast Guard icebreaker, M.V. Terry Fox, outfitted with a rudder. The rudder angle was kept at zero for all ice tests. Series of model tests using the Model 417 were carried out in IOT (Derradji and Van Thiel, 2004; Lau, 2006). The model’s initial condition is: Draft, 0.376m; Trim, 0.0m; Displacement, 682.5kg. Table 1 provides the segmented water line width and the flare angles of the Terry Fox Model of each segment. Figure 4 shows Terry Fox’s water line profile represented in the IHI Model.
IOT model 491A is a 1:20 scaled model of the Canadian Coast Guard (CCG) R-Class Icebreaker outfitted with twin propellers and a single rudder at centerline. The model’s initial condition is (Hoffmann, 1998): Forward Perpendicular Draft, 0.338m; After Perpendicular Draft; 0.362m; Draft in Mid-ship, 0.35m; Trim, 0.024m; Displacement, 965kg. Table 2 provides the segmented water line width and the flare angles of the R-Class model of each segment. Figure 5 shows R-Class Model water line profile represented in IHI Model.

4.2 Test Conditions
Besides the resistance runs and constant radius manoeuvres, the sinusoidal runs were also selected for the benchmark in order to showcase the model’s ability in simulating the ship’s arbitrary manoeuvres in ice. In the Terry Fox model tests, one type of ice thickness, 40mm, was used and the model ship’s velocities ranged from 0.02m to 0.6m. The ice flexural, shear and compressive strengths were 31.5 kPa, 44.2 kPa and 130 kPa respectively. In the R-Class model tests, two types of the ice thickness, 30 mm and 50 mm, were used in tests and the model’s tangential velocity was kept at 0.6m/s. The ice flexural, shear and compressive strengths were 20.0 kPa, 28.0 kPa and 82.5 kPa respectively. In all above captive model tests, the pivot point was fixed at the mass centre of the model. The rudder angle was kept at zero degree.

4.3 Channel Comparison
A satisfactory simulation of the geometry of the broken channel is important, as the intact ice edge interacts with the ship hull leading to interaction load (Lau et al, 2004). The hull, due to the flexural bending failure at the ice-hull contact area, breaks the unbroken ice sheet and the ice cusps are continuously created with the ship’s motions. Some the cusps of ice reach the hull bottom and leave the hull eventually and others may be pushed to the sides of the ship. A channel is cleared behind the icebreaker. The ice properties and ship’s velocities directly affect the size of ice pieces broken by the hull. The drift angle and constant radius directly affect the positions of the ice-hull contact and the shape of the ice edge the ship leaves behind. The different channel width reflects a different ice-hull contact condition that, in turn, determines the ice force distribution along the hull surface and affects the global ice forces imposed on the ship.

Figure 6 shows the channel created by the Terry Fox model in a constant radius run with 0.4 m/s tangential velocity and 10 m turning radius. Figure 7 shows the channel simulated by the IHI model for the same test condition. The model calculated the average channel resulting in a smooth channel edge as shown in Figure 7. The actual ice channel edges observed during tests were irregular; therefore, trend lines were used to fit the measurements, i.e., the red curves in Figure 6, and the average width between two trend lines was used for the comparison. Figure 8 showed a comparison between the predicted channel widths and the measurements (Lau, 2006) as a function of turning radius for the Terry Fox model tests.

The comparison showed that the calculated channel widths agreed well with the measurements, i.e., they fell within 10% of each other; therefore, the simulated channel width was reasonable and acceptable. The 50 meters radius data agreed better than that of the 10 meters radius data. It might be due to the different sample size of channel width measurements, as more data points taken for the 50m runs reduced the uncertainty of the width estimate.
4.4 Ice Force Comparison

4.4.1 Resistance Tests
Figure 9 shows a comparison of the measured resistance force to its predictions for the Terry Fox resistance tests in level ice and pre-sawn ice. Again, they agree well with each other with a relative discrepancy smaller than 20% for most data points. The discrepancy may be due, in part, to uncertainty and non-uniformity of ice properties, i.e., ice thickness, ice density, failure strength, friction coefficients, etc, over the entire ice sheet.

Except for the data point corresponding to 0.6m/s ship speed, which seems not follow the data trend, the predictions are lower than measurements. This may be caused by the neglect of ice crushing at the stem, secondary cracks on some big ice cusps and frictional forces during ice sliding process on the wet surface of the hull. For the pre-sawn ice run at 0.1 m/s model velocity, a relatively bigger discrepancy can be observed. The discrepancy may also be caused by the model idealization and simplifications of the problem treatment, i.e., the simple flat-plate representation of the model hull for buoyancy calculation; the idealized ice breaking process and clearing process, ice piece pattern and ice piece size, etc.

It should also be noted that the model is based on low to median ship speed. At high speed, the submersion process of ice pieces is very complicated and an independent resistance component may not exist (Kamarainen, 1994).

4.4.2 Constant Radius Tests
Figure 10 shows a comparison of the measured moment to its predictions for the Terry-Fox constant radius tests in 40mm level ice. The comparison shows the predictions were within 20% of the corresponding measurements for the 10 meters runs. For the 50 meters runs, the data spread is relatively large and only four data points are available for comparison; nevertheless, the predictions are within the spread of the measured data.

Shi (2002) reported a drift angle of 5.6º existed for the R-Class constant radius runs. Therefore, a 5.6º drift angle was prescribed in the simulations of the R-Class constant radius runs. Figure 11 shows a comparison of the measured moment to its predictions for the R-Class runs with various constant radii that were tested in 30 mm and 50mm level ice. The comparison showed that the model predicted fairly well the yaw moment and its dependency on the turning radius. The comparison showed that the discrepancy between measurements and predictions is within 15% for the 30 mm thick ice tests and 35% for the 50 mm thick ice tests. The simulation results are larger than the corresponding test measurements, which may be caused by a larger than actual drift angle than was prescribed in the simulations, which may affect the final comparison.

4.4.3 Sinusoidal Tests
Figure 12 is the measured yaw moment measured during sinusoidal run with the R-Class model in the 30mm level ice, which were taken from Shi (2002). The runs correspond to 0.6 m/s tangential velocity100 seconds period, zero drift angle, and with a yaw rate ranging from −0.05
to 0.05 rad/s. Considering that the yaw rate kept changing in order to keep the centerline of the model always inline with its path during the sinusoidal runs, it is more practical to compare the trend line of the measured data to that of the predictions. To compute the predicted moments, a drift angle of 0.5° was prescribed according to the measurement reported by Shi (2002).

Figure 13 shows the comparison of the sinusoidal runs obtained in the 30 mm ice. Figure 14 and 15 respectively show the comparisons of the measured yaw moments and sway forces to those predicted for the 50 mm runs. The comparisons showed that the trend lines of the measured data agree fairly well to those from predictions. The simulated yaw moment vs. yaw rate curves was roughly a straight line, which was expected theoretically. The yaw moment vs. yaw rate curves and the sway force vs. yaw rate curve all offset from the origin, which may be caused by the drift angle that results in sway velocity.

It is more difficult to maintain an exactly steady sinusoidal motion in ice than in open water, and this affected the final comparison results.

5.0 Conclusions
A new hull-ice interaction model for real-time simulation of ship navigation in level ice was presented. The numerical implementation of the model was benchmarked using data from two model test series; Terry Fox model tests and R-Class model tests, conducted at IOT using an PMM.

The benchmarking results showed the ice-hull interaction model predicted fairly well the measurements obtained from model tests with good accuracy, despite significant model idealization. The benchmarking also showed the model’s capability in simulating different ship’s manoeuvres in ice. It could favourably predict the global ice forces on the hull with good numerical efficiency and universality that are essential to marine simulation.

The comparison also showed that the drift angle played an important role in influencing the global ice loads on the hull; therefore, more attentions should be paid to the accurate control or measurement of drift angle in future tests.

Acknowledgement
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Reference


Shi Y., 2002, “Model Test Data Analysis of Ship Maneuverability in Ice”, Master Degree Thesis, Faculty of Engineering and Applied Science, Memorial University of Newfoundland, St. John’s, Canada.


Figure 1. Global force and yaw moment calculation in IHI model

Figure 2. Software structure of IHI Model program with main m-files

Figure 3. Sketch of a flat-plate model for buoyancy force calculation

Figure 4. Terry Fox Model water line profile represented in the IHI Model
Table 1. Geometries of Terry Fox Model represented in IHI model

<table>
<thead>
<tr>
<th>Location (m)</th>
<th>Half WL Width (m)</th>
<th>Flare angle (°)</th>
<th>Area (m²)</th>
</tr>
</thead>
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<td>0.000f</td>
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<td>0.0969</td>
</tr>
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</table>

1 The transverse plane was measured at interval forwards of the aft perpendicular (AP).
2 The equivalent area of the hull side surface under the waterline till the bottom at each section as shown in Figure 3.

Table 2. Geometries of IOT R-Class model represented in IHI model

<table>
<thead>
<tr>
<th>Location (m)</th>
<th>Half WL Width (m)</th>
<th>Flare angle (°)</th>
<th>Area (m²)</th>
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</thead>
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<td>0.0150</td>
</tr>
</tbody>
</table>

1 The transverse plane was measured at interval forwards of the aft perpendicular (AP).
2 The equivalent area of the hull side surface under the waterline till the bottom at each section as shown in Figure 3.
Figure 9. Comparison of measured and simulated resistance for Terry Fox Model in 40 mm - 31.5 kPa ice at test speeds ranging from 0.1 to 0.6 m/s

Figure 10. Yaw moment comparison for Terry Fox Model with 10 and 50 meters radius turning in 40mm-31.5kPa ice

Figure 11. Yaw moment comparison of IOT R-class Model constant radius runs in 30 mm-20 kPa and 50 mm-20 kPa level ice

Figure 12. Regression test results of R-Class model sinusoidal run in 30 mm - 20 kPa level ice (Shi, 2002)

Figure 13. Yaw moments comparison of R-Class Model sinusoidal run in 30 mm - 20 kPa flexural strength ice.

Figure 14. Yaw moments comparison of R-Class Model sinusoidal run in 50 mm - 20 kPa flexural strength ice
Figure 15. Sway force comparison of R-Class Model sinusoidal run in 50 mm - 20 kPa level ice
Static and Dynamic Interaction of Floating Wedge-Shaped Ice Beams and Sloping Structures

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Abstract

When level ice interacts with a sloping structure, or when a ship advances in level ice, the ice sheet may begin to fail by forming cracks in the radial direction. These radial cracks will be lengthened and increased in number until a circumferential crack is formed and consequently the ice sheet reaches its ultimate capacity. After the formation of the radial cracks the ice sheet can no longer be modelled as a continuum, instead it is common to use the model of adjacent wedges. This paper reviews the state-of-the-art in modelling the ultimate failure of an ice sheet using the model of adjacent wedge-shaped beams.

In this paper, both the static and dynamic problems are formulated for a floating wedge-shaped beam interacting with a sloping structure. For the dynamic interaction, the results of the elastohydrodynamic approach are compared with the model of Winkler foundation combined with added mass and hydrodynamic damping. The comparison shows that the elastohydrodynamic model is more reliable than the Winkler approach. The breaking lengths of the ice wedges are also investigated and it is concluded that the breaking lengths increase with increasing ice thickness and/or axial compression in the ice; while increasing the drift acceleration will always decrease the breaking lengths. The static results match the results of the elastohydrodynamic solution for small ice drift accelerations. The calculations are performed using the commercial finite element program “Comsol Multiphysics”.
1. Introduction

Accurate predictions of ice actions are vital in order to optimise the design of structures in the Arctic regions. A good understanding of the ice-structure interaction process will help establishing reliable models to estimate the ice forces. When level ice drifts against a fixed structure, or when a ship advances in level ice, the ice forces will increase until the ice sheet fails and hence the forces exerted on the structure drop. The failure of the ice sheet may occur in different modes, namely crushing, bending, buckling or mixed mode where two or more of the failure modes are active at the same time. The ice properties, the structure width and the relative drift velocity control the mode of failure of the ice sheet. In the case of an ice sheet is being pushed at moderate speeds against a sloping structure, the bending failure of the ice sheet will dominate over the other modes of failure. Based on this hypothesis, the theory of an elastic plate resting on an elastic foundation “Winkler foundation” has been used to model the interaction between level ice and sloping structures. It is important at this point to highlight the influence of the relative drift velocity on the accuracy of the above mentioned model. A relatively low velocity between the ice and structure causes the behaviour of the ice to be inelastic rather than elastic. High speed interaction will make the accuracy of the Winkler foundation model questionable, since such foundations account only for the restoring forces from the water.

Hertz (1884) was the first to solve the static problem of an infinite floating plate with a vertical concentrated load at its origin in terms of infinite series. Later, Hetenyi (1946) and Wyman (1950) presented solutions to the same problem in terms of Bessel functions. The semi-infinite plate on elastic foundation with a vertical static load at the free edge was solved by Nevel (1965) and Kerr and Kwak (1993). All the aforementioned solutions which treat ice as a continuum are useful to predict the initiation of cracks in ice simply by comparing the stress in the ice sheet with a failure criterion, i.e. here the flexural strength of ice. On the other hand, these solutions can not predict the number and the extent of the cracks and hence they fail to estimate the ultimate failure of the ice sheet. One promising approach to overcome the limitation of the elastic plate theory is to implement the fracture mechanics concepts where the energy input from the external load is balanced by the energy dissipated in deforming the ice and creating the cracks. Recently fracture mechanics approaches have been used by several researchers to study the cracks initiation and propagation in ice. Nevertheless the state-of-the-art in studying the ultimate failure of an ice sheet is still based on the observations from experiments and full-scale, which suggest predefining the cracking patterns into radial and circumferential cracks. According to these observations, the ice begins to fail by forming cracks in the radial direction starting from the contact point. These radial cracks are due to the tension at the bottom of the ice plate. Increasing the forces applied on the ice will increase the number of the radial cracks and lengthen them until a circumferential crack is formed, see Nevel (1972).

The continuum approach using the elastic plate theory will not be able to model the circumferential cracks because the plate is already broken by the radial cracks. In order to predict these circumferential cracks, Nevel (1958) proposed replacing the plate by adjacent wedge-shaped beams resting on an elastic foundation. The first static solution of a wedge beam on elastic foundation under a vertical load at the apex was presented by Hetenyi (1946) in terms of simple functions. The solution of Hetenyi, however, diverges near the apex which makes it useless for the problems of ice-structure interaction. Nevel (1961) published a power series
solution for a wedge with vertical static distributed load and his solution converges over the whole domain. In addition to the vertical force, there exists horizontal force acting on the wedge. This horizontal force will modify the stress in the wedge and it may cause the wedge to fail in buckling. Nevel (1979) solved the static problem of a wedge beam with horizontal and vertical loads analytically. He presented the solution in integral forms and was able to study the bending and buckling failure of the ice wedge. Later Nevel (1992) simplified the integral solution and republished it in terms of infinite series. In his new solution, Nevel neglected the effects of the horizontal force on the bending of the ice wedge and considered it only when calculating the flexural stress. The results of the buckling analysis from Nevel were confirmed by a numerical analysis conducted by Sodhi (1979). Sodhi used the finite element method (FEM) and considered a radial stress field for the in-plan stresses in the ice sheet.

Kerr (1978) provided approximate expressions for the static buckling forces of a semi-infinite wedge on elastic foundation. Li and Bazant (1994) examined the use of beam theory to solve the ice wedge problem. They used the finite difference method (FDM) together with the plate theory and finally concluded that the results of beam theory are sufficiently accurate for wedge angles up to $\pi/4$. Määttänen and Hoikkanen (1990) used the FEM to solve the static problem of wedge-shaped beam subjected to axial compression and distributed transverse load by discretizing the beam into finite elements and assuming a constant axial load along each element. McKenna and Spencer (1993) adopted the theory of beams on Winkler foundation to study the dynamics of the ice wedge using the FEM. They derived the mass and stiffness matrices for a wedge-shaped beam element and they assumed constant added mass and hydrodynamic drag coefficients when solving the dynamic equation of motion. Dempsey and Zhao (1993) investigated the validity of using added mass together with Winkler foundation for solving the dynamic problems of a floating ice sheet and they concluded that this approach can not model the dynamic response accurately because the added mass varies with time and space. Dempsey et al. (1999) presented an elastohydrodynamic approach to study the dynamic problem of floating ice beams where the ice is modelled as an elastic beam and the water is modelled as a potential flow. This paper studies the static and dynamic interaction between sloping structures and floating wedge-shaped ice beams. First the problem of dynamic interaction is formulated according to the elastohydrodynamic model. Second the model of Winkler foundation combined with added mass and hydrodynamic damping is discussed and finally the static problem is presented. The commercial finite element program “Comsol Multiphysics” is used to solve the interaction problem. The results are shown, discussed and finally conclusions are drawn.

2. Model Description
Figure 1 shows a wedge-shaped ice beam of thickness, $h$, and angle, $\theta$, floating on water of constant depth, $d$, and drifting against an upward sloping structure with slope angle, $\alpha$.

a) Elastohydrodynamic model
According to the elastohydrodynamic model, the governing differential equation of the floating wedge in Figure 1 is

$$\rho_i h \frac{d^2w}{dt^2} + \frac{1}{b} \frac{d^2}{dx^2} \left[ EI \frac{d^2w}{dx^2} \right] + \frac{H(t)}{b} \frac{d^2w}{dx^2} - p_i(x,t) = q(x,t) \quad (w = 0 \text{ at } t = 0) \quad [1]$$
Figure 1. A floating ice wedge drifts against an upward sloping structure.

Where \( w \) is the transverse deflection of the beam, \( x \) is the space coordinate along the beam, \( t \) is the time, \( \rho_i \) is the density of ice, \( E \) is the modulus of elasticity, \( I \) is the moment of inertia \((I = b \cdot h^3/12)\), \( b \) is the width of the beam \((b = b_0 \cdot x)\), \( b_0 \) is the width of the beam at 1 m from the apex, \( H \) is the in-plane compressive force, \( p_i \) is the pressure on the bottom surface of the wedge due to the hydrodynamic reaction from the water and \( q \) is the external applied pressure.

Assuming irrotational flow, the motion of the water is governed by potential theory where the velocity vector is expressed as the gradient of the velocity potential, \( \phi(x,z,t) \), see Nevel (1970) and Fox and Chung (2002). The irrotational flow is also continuous (the water is incompressible) which means that Laplace equation must be satisfied in the water domain

\[
\nabla^2 \phi = \frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \quad -d < z < 0 \quad -\infty < x < \infty
\]

Equation 2 is a second order partial differential equation that has the following boundary conditions:

\[
\begin{align*}
\frac{\partial \phi}{\partial z} &= 0 \quad z = -d, \quad -\infty < x < \infty \quad \text{(at the sea bed)} \\
\frac{\partial \phi}{\partial z} &= \frac{\partial w}{\partial z} \quad z = 0, \quad 0 < x < \infty \quad \text{(at the ice-water interface)} \\
\frac{\partial^2 \phi}{\partial t^2} + g \frac{\partial \phi}{\partial z} &= 0 \quad z = 0, \quad -\infty < x < 0 \quad \text{(at the water free surface)} \\
p_i + \rho_w g w + \rho_w \frac{\partial \phi}{\partial t} &= 0 \quad z = 0, \quad -\infty < x < \infty \quad \text{(linearized Bernoulli equation)} \\
\phi &= 0 \quad -d < z < 0, \quad x = \pm \infty
\end{align*}
\]
where \( \rho_w \) is the density of water and \( g \) is the acceleration of gravity. If the ice sheet is drifting with a constant acceleration, \( a \), against the sloping structure, the transverse deflection of the wedge at the contact point will be

\[
w(0,t) = 0.5 \cdot a \cdot t^2 \cdot \tan(\alpha)
\]  

[4]

Solving equation 1 and 2 simultaneously and satisfying their boundary conditions in 3 and 4 will couple the physics of the elastic beam with that of the hydrodynamic foundation and this elastohydrodynamic solution is hoped to predict accurately the dynamic response of the floating beam. Here it is important to mention that the deflection of the ice before the breakup is typically much less than the ice thickness and therefore the partial emergence of the ice sheet prior the fracture is not considered in the present model.

**b) Dynamic model using Winkler foundation and added mass**

Several researchers used the model of elastic beam on Winkler foundation combined with added mass and hydrodynamic damping in order to study the dynamics of floating ice. In this case, equation 1 is replaced by

\[
\rho \frac{d}{dt} \left( 1 + c_a \right) \frac{d^2 w}{dt^2} + \rho w c_w \left\{ \frac{dw}{dt} \frac{dw}{dt} + \frac{1}{b} \frac{d^2}{dx^2} \left[ \frac{E I}{E} \frac{d^2 w}{dx^2} \right] + \frac{H(t)}{b} \frac{d^2 w}{dx^2} + \rho w g w = q(x,t) \right. 
\]  

[5]

where \( c_a \) and \( c_w \) are the added mass and hydrodynamic damping coefficients, respectively. Here, the hydrodynamic damping is mainly due to the viscosity in the boundary layer and therefore a linear damping term would be appropriate. However, a quadratic term is used in equation 5 following the presentation of McKenna and Spencer (1993) and the influence of this on the numerical results was found to be minor. The boundary conditions of equation 5 are the same as those of equation 1 and the solving techniques are pretty much the same. However, solving Laplace equation is not needed here in order to calculate the pressure under the ice wedge.

**c) Static model**

For the static and/or quasi-static interaction, the inertia and damping effects diminish and the model can be simplified as shown in Figure 2.

![Figure 2. Wedge-shaped ice beam on elastic foundation (static interaction).](image)

The governing differential equation becomes
\[
\frac{1}{b} \frac{d^2}{dx^2} \left[ E I \frac{d^2w}{dx^2} \right] + \frac{H(t)}{b} \frac{d^2w}{dx^2} + \rho_w g w = q(x,t) \]  

[6]

The static solution is not influenced by the rate of the deflection and therefore the boundary conditions can be applied simply as external static forces, i.e. \( P \) is a vertical force and \( H \) is a horizontal force, as shown in Figure 2.

In the following, the commercial finite element program “Comsol Multiphysics” is used in order to solve the static and dynamic problems of an ice wedge. The use of “Comsol Multiphysics” makes it possible to solve a two dimensional elasticity problem when modelling an ice wedge instead of solving the simple Euler-Bernoulli beam equation, i.e. equations 1, 5 and 6. Consequently, the ice wedge is modelled in the present simulation as plane stress where the Poisson's ratio is set equal to zero. Table 1 summarises the properties of this numerical model. In Table 1, the geometrical properties of the wedge and properties of the ice are chosen similar to those used by McKenna and Spencer (1993).

### Table 1. The properties of the numerical model.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>Water free surface</td>
<td>300 m ((−300 ≤ x &lt; 0 \text{ m}))</td>
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<tr>
<td>Water depths ((d))</td>
<td>120 m ((0 &lt; z &lt; 120 \text{ m}))</td>
</tr>
<tr>
<td>Wedge length ((L))</td>
<td>300 m ((0 ≤ x ≤ 300 \text{ m}))</td>
</tr>
<tr>
<td>Wedge angle ((\theta))</td>
<td>45° ((b_o = 2.0))</td>
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<tr>
<td>Wedge truncated distance ((x_{\text{truncated}}))</td>
<td>2.5 m</td>
</tr>
<tr>
<td>Wedge thicknesses ((h))</td>
<td>((0.3, 1.0) \text{ m})</td>
</tr>
<tr>
<td>Ice flexural strength</td>
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</tr>
<tr>
<td>Ice modulus of elasticity ((E))</td>
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</tr>
<tr>
<td>Ice density ((\rho_i))</td>
<td>900 kg/m(^3)</td>
</tr>
<tr>
<td>Water density ((\rho_w))</td>
<td>1025 kg/m(^3)</td>
</tr>
<tr>
<td>Structure slope from the horizontal ((\alpha))</td>
<td>60°</td>
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<tr>
<td>Drift accelerations ((a))</td>
<td>((0.001, 0.01, 0.1, 0.5, 0.7, 1.0) \text{ m/s}^2)</td>
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<td>Added mass coefficient for Winkler-dynamic model ((c_a))</td>
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<td>Hydrodynamic damping coefficient for Winkler-dynamic model ((c_w))</td>
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<td>Wedge boundary conditions</td>
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<tr>
<td></td>
<td>At the root ((x = 300 \text{ m})): Fixed</td>
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</tbody>
</table>
3. Results and Discussion

Figure 3 shows the hydrodynamic reaction forces under the ice wedge at two different drift accelerations namely 0.001 and 0.5 m/s². The wedge thickness is 1 m and the hydrodynamic reaction forces are calculated using the elastohydrodynamic and the Winkler-dynamic models.

**Figure 3.** The hydrodynamic reaction forces at the breakup under the 1 m thick ice wedge drifting with the constant accelerations a) 0.001 and b) 0.5 m/s² (the results are according to the elastohydrodynamic and to the Winkler-dynamic models).

From Figure 3 it is evident that the results of the elastohydrodynamic and the Winkler-dynamic models are very similar at small drift accelerations. However, they diverge considerably at high accelerations. Figure 4 presents the flexural stress at the breakup for the same wedge discussed above. From the figure one sees that the elastohydrodynamic solution reaches the fracture slightly faster than the Winkler foundation solution. The breaking length of the wedge measured from the sloping structure is 7.76 m at small acceleration (0.001 m/s²). At high acceleration (0.5 m/s²), the breaking length is 4.5 m according to the elastohydrodynamic solution and 6.26 m according to the Winkler approach.

**Figure 4.** The flexural stress at the breakup of the 1 m thick ice wedge drifting with the constant accelerations a) 0.001 and b) 0.5 m/s² (the results are according to the elastohydrodynamic and to the Winkler-dynamic models).
The results of the Winkler model presented in Figures 3 and 4 may differ quite much if other values of the added mass were used. The Winkler-dynamic model could be improved by using an added mass coefficient that varies as a function of the wedge width. But such model will still be unable to consider the time variation of the added mass. All this indicates that the elastohydrodynamic approach is more reliable and the results obtained using Winkler foundation and added mass should be treated with some caution.

In order to investigate more closely the effects of drift acceleration on the breaking lengths of the above wedge, the elastohydrodynamic approach is used and the accelerations are varied from 0.001 to 1.0 m/s². The results of the flexural stress at the breakup are illustrated in Figure 5 and they show clearly that the breaking lengths reduce as the drift acceleration increases.

![Figure 5](image)

**Figure 5.** The flexural stress at the breakup of the 1 m thick ice wedge drifting with the constant accelerations 0.001, 0.01, 0.1, 0.5, 0.7 and 1.0 m/s² (the results are according to the elastohydrodynamic).

After subsequent breaking of ice, rubbles are created and typically accumulated in front of the sloping structures. The rubble accumulation causes the ice wedges to push through the rubbles during the interaction with the structure. As a result, the wedge will be subjected to axial compression from the rubble in addition to the horizontal and vertical forces from the structure. The effect of this axial compression on the breaking lengths of the wedge is examined in this paper. An axial force of 22.5 MN is used as an example and the results of the breaking lengths as a function of the drift acceleration are shown in Figure 6.

Figure 6 highlights also the effects of ice thickness on the breaking lengths by presenting the breaking lengths of a 30 cm thick wedge as a function of the drift accelerations. The information obtained from Figure 6 suggests that increasing the ice thickness and/or the axial compression in...
ice will increase the breaking lengths; in the mean time increasing the drift acceleration will always decrease the breaking lengths.

Figure 6. The breaking lengths of several wedges as a function of the ice drift accelerations.

Figure 7 shows the results of the static analysis. The analysis included three wedges. The first is 0.3 m thick, the second is 1 m thick and the third is 1 m thick and subjected to axial force of 22.5MN. The vertical static forces that caused failure in these wedges are 33.36 kN, 261.7 kN and 320.2 kN, respectively. The breaking lengths are 4.0 m, 7.76 m and 13.51 m, respectively. Here, it is interesting to note that these static breaking lengths match well with the dynamic breaking lengths calculated from the elastohydrodynamic model for small drift accelerations.

Figure 7. The static flexural stress of several wedges.
4. Conclusions
This paper looked at the static and dynamic interaction of floating wedge-shaped ice beams and sloping structures. The ice is assumed to fail in bending and any transition in the failure mode is not considered. The most important finding are summarised as follows

- The elastohydrodynamic approach is reliable when modelling the dynamic interaction of ice wedge and sloping structures.
- The Finite Element Method provides a powerful tool to solve such elastohydrodynamic model.
- The results obtained using Winkler foundation and added mass should be treated with some caution.
- The breaking lengths increase by increasing the ice thickness and/or the axial compression in ice; meanwhile increasing the drift acceleration will always decrease the breaking lengths.
- The static solution matches well with the elastohydrodynamic solution at small ice drifts accelerations.

A thorough parametric study is needed in order to understand more the influence of the different parameters on the interaction process. In addition to the parameters introduced in this paper, several other parameters should also be included in the study such as the water depth, constant drift velocities, vertical distributed loads, and the area of the distributed load …etc.

5. References


Finite Element Analysis of Fluid-Ice Interaction during Ice Bending

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This study deals with a numerical simulation of the dynamic bending behavior of a floating ice-sheet subjected to the dynamic force by using ABAQUS/Explicit. The ice is considered as a homogeneous elastic material. An incompressible and inviscid of the flow is assumed. The fluid model is assumed to be a simple hydrodynamic model which is obtained by the equation of state material model which accounts for volumetric strength of hydrodynamic material. The pressure in the material is described as the function of density and bulk modulus. 2D and 3D fluid-structure interaction calculations are carried out. Infinite and semi-infinite floating ice-sheet is subjected to rapid loading at the ice edge. The displacement and stress distribution results acquired from FE calculation are compared with the analytical solutions. The FE results agree well with the analytical ones. Further, the influence of the dynamic response due to a rapid forcing and wedge angle is discussed. The large effect of free surface of water and the bending behavior depending on the ice wedge angles are demonstrated.
1. Introduction
The ice loads exercised on a ship advancing into a field of a level ice represent an important factor for the ship design and the navigation in arctic areas. When a ship collides with a floating ice-sheet, the floating ice is subjected to a rapid loading at its edge, and then fails by local crushing and bending. The vertical force caused by the inertia of ice-sheet and the dynamic pressure of water in a bending mode increase quicker than in a quasi-static case when the indentation proceeds. The dynamic response induced by the interaction between fluid and structure is more significant than the static response. This study deals with the 3-dimensional dynamic response of an ice-sheet leading to the bending failure.

Several researchers applied analytical methods related to a dynamic response due to an edge force on an ice-sheet or an incident wave, etc (Fox and Chung 2002 and 1998, Fox 2002, Fox and Squire 1994, Zhao and Dempsey 1996). These approaches are suitable to understand the details of an interaction between floating ice and sea water, as well as to achieve precise solutions effectively. Mathematical models have become more sophisticated and have thus been applied to complex sea-ice conditions, e.g. crack problems of ice, inhomogeneity of ice, and so on. However, analytical approaches are limited to 1D or 2D conditions, or related to a 3D simple shape of an ice. Therefore, they have not been applied to 3D dynamic response with various wedge angles of ice such as icebreaking patterns between ships and level ice. On the other hand, a 3D numerical model of the icebreaking process of a ship advancing in level ice has been developed by Valanto 2001. The ice breaking mechanism of each ice breaking cycle, i.e. the resistance of a floating ice and surrounding water, the cusps breaking, the ventilation and the ice crushing, computed by the combination of the numerical model and the simple formulations. The numerical solutions show agreement with the experimental and measured data. The solutions focused on one breaking cycle of ice sheet, which is related to one cycle icebreaking process in an unbroken level ice field. In order to estimate the repeated icebreaking force on ships advancing in level ice, dynamic interaction of various ice edge geometries in an unbroken and broken ice field needs to be considered, especially because of the complex ice breaking pattern created by a forward moving ship.

In this paper, we focus on the dynamic response in a bending failure of various wedge angles of a sheet ice. Numerical calculations are carried out by the ABAQUS/Explicit which is a commercial finite element (FE) analysis code which turns out to be well suited for calculating the interaction of fluid-structure, e.g. Stenius, Rosen, and Kuttenkeuler 2006 and Bereznitski 2001. Computed results are compared with the analytical results of 2D or 3D infinite or finite floating plate which have been obtained by Fox, et al. 2002 and 1998, Fox 2002, Fox et al. 1994 and Zhao et al. 1996. FE results show the good agreement with analytical ones. The applicability of the FE model to the calculations of fluid-structure interaction in a floating ice-sheet is confirmed. Moreover, the remarkable dynamic effect of free surface of water is presented. The ice breaking force and the ice breaking pattern are affected by the wedge angle. Thus the influence of a dynamic response due to wedge angle is presented.

2. FE calculation for fluid-structural interaction
FE calculations of a fluid-structure interaction are conducted by the computer code ABAQUS/Explicit 6.6. The interaction analysis is carried out by using the equation of state model for fluid and the contact algorithm between the fluid FE mesh and the structural FE mesh.
A detailed description of the fluid-structure interaction modeling with ABAQUS/Explicit 6.6 is found in the ABAQUS Manual.

To model an incompressible inviscid flow in ABAQUS/Explicit 6.6, a linear $U_s-U_p$ equation of state model which can derive an incompressible viscous flow governed by the Navier-Stokes equation of motion is used. Moreover, a small amount of shear viscosity to suppress shear modes that could tangle the mesh is also defined. The water is treated as a simple hydrodynamic material model. This model provides zero shear strength and a bulk response given by

$$P = \rho_0 c_0^2 \eta = \rho_0 c_0^2 \left(1 - \frac{\rho_0}{\rho}\right) = K \left(1 - \frac{\rho_0}{\rho}\right)$$

where, $P$ is a pressure, $\rho_0$ is a reference density, $\eta$ is a nominal volumetric strain, $c_0$ is a particle velocity, $\rho$ is a density and $K$ is a bulk modulus. In this study, $\rho_0 = 1000$ kg/m$^3$ and $K = 2.25$ GPa. The shear viscosity is chosen to be $1.5 \times 10^{-5}$Pa sec.

The fluid and structure are meshed as two distinct bodies. Two bodies are interacting by the contact algorithm at the interfacing surface. In this study, the penalty contact algorithm with frictionless contact in a tangential component is used. The contact algorithm is defined between the top surface of the water and the bottom surface of the ice-sheet.

### 3. Computational model

Figure 1 shows the model geometry and the examples of FE meshes for a semi-infinite floating ice. The model consists of a water tank filled with water and an ice-sheet. An ideal computational area has to been defined as an infinite area, which indicates an infinite water domain and an infinite ice domain in the horizontal plane. Therefore, the relative large computational domain for water and ice is arranged in order to reduce the effect of the finite model size. The bottom and side boundaries of water are fixed in a normal direction of the tank’s walls. The ice-sheet boundaries attached to the tank’s walls are fixed in the same manner as a water domain as illustrated by Figure 1. The loaded edge of the ice-sheet has a free boundary condition. The water and ice-sheet are modeled with the 8-noded linear bricks element (C3D8R) for the 3D analysis and 4-noded bilinear plane strain quadrilateral element (CPE4R) for the 2D analysis. FE models are meshed with fine elements adjacent to the loading point. Since large deformation in water domain is anticipated, adaptive mesh domain in the water is defined to maintain the mesh quality during the simulations. The water and ice-sheet is subjected to a gravity force. In the following analyses, Young’s modulus of ice $E$ is $5.4$ GN/m$^2$ and Poisson’s ratio $\nu$ is $0.3$. The density of ice $\rho_i$ and water $\rho_w$ are $900$ kg/m$^3$ and $1000$ kg/m$^3$, respectively. The Bulk modulus of water $K$ is $2.25$ GPa. The ice thickness is $1$ m. Different types of load conditions are used in the simulation. Each load condition is shown in the description of each calculation condition.

### 4. Comparison of FEA and analytical approaches

Figure 2 illustrates a 2D model for an infinite floating ice with a line loading (Figure 2 (A)), the analysis condition of FEM (Figure 2 (B)) and the time history of force at the center line of ice-sheet (Figure 2 (C)). The length and the thickness of the ice domain are $300$ m and $1$ m,
respectively. The length and the depth of the water domain are 300 m and 50 m, respectively. The minimum element size of the initial FE mesh is 1.0 m adjacent to a loading point ($x=0$) in the ice-sheet. The number of elements of the ice-sheet is 200, and that of the water is 1000. An identical mesh of the following 3D FE calculation in the x-z plane is used. The line load with the maximum magnitude of 1.0 MN/m is applied on the ice-sheet as shown in Figure 2. Figure 3 (A) demonstrates the displacement $d_z$ on the top surface of the ice-sheet. The magnitude of the displacement adjacent to the loading point of FE results each time (0.5, 1.0, 1.5 and 2.0 sec) is larger than for the two analytical results (Fox et al. 2002 and Zhao et al. 1996). However, the displacement obtained by the FEM approximately agrees with the analytical results. Two analytical results are in agreement with each other. Figure 3 (B) shows the stress $\sigma_x$ on the top surface of the ice-sheet. The magnitude of maximum stress and stress distribution after reaching the maximum point of FE results at each time (0.5, 1.0, 1.5 and 2.0 sec) is smaller than the analytical results (Zhao et al. 1996). However, the locations of the maximum stress which is located at 20 m at 0.5 sec are approximately coincident with one another.

Figure 4 illustrates a 3D infinite floating ice with a point loading, the analysis condition of FEA and the time history of force at the center of the ice. The length, width and the thickness of the ice are 300 m, 300 m and 1 m, respectively. The length, width and the depth of the water domain are 300 m, 300 m and 50 m, respectively. The minimum element size of an initial FE meshed is 0.5 m in the ice-sheet adjacent to the loading point ($x=0, y=0$). The number of elements in the ice-sheet is of 40000, and that of the water is of 200000. The slope and constant after 0.1 sec point load with the magnitude 1.0 MN is loaded on the ice-sheet as shown in Figure 4. Figure 5 (A) and Figure 5 (B) point up the displacement $d_z$ and the stress $\sigma_x$ on the top surface along the centerline of the ice-sheet, respectively. The displacement obtained by FEM shows good agreement with analytical results (Fox et al. 2002). The comparison of the stress distribution of FEM and analytical results shows a similar tendency as the one observed in 2D infinite floating ice with line loading (See Figure 3 (B) and other mentioned above). Also, the stress distribution obtained by FEM agrees with the analytical results (Fox et al. 2002).

A 2D semi-finite floating ice with line loading is analyzed. The 2D semi-infinite floating model is identical with 2D infinite floating ice except for the ice-sheet size. The ice size is half size of the 2D infinite model, and ice-sheet is arranged at the left side ($0< x < 150$) of the water tank. The length and the thickness of the ice domain are 200 m and 1 m, respectively. The length and the depth of the water domain are 400 m and 50 m, respectively. The minimum element size of the initial FE meshes is 0.8 m adjacent to a loading point ($x=0$) in the ice-sheet. The edge of the ice-sheet is loaded with the line load of the maximum magnitude of 0.1 MN/m. Figure 6 (A) and Figure 6 (B) present the displacement $d_z$ and the stress $\sigma_x$ on the top surface of the ice-sheet for FE and analytical results (Fox et al. 2002), respectively. The displacement obtained by FEM until 0.4s agrees well with the analytical results. The disagreement of displacement between FEM and analytical results is gradually increasing as the time is progressing, especially, as the ice edge displacement becomes quite large. The stress distribution of FEM shows larger distributions than the analytical ones at all time steps. On the other hand, FEM stress distribution of 2D and 3D infinite floating ice is smaller than analytical ones (see Figures 3 and 5). The fluid flow behavior of free surface is considered the main cause of this disagreement. However, the location of the maximum stress at each time step of FEA, where the flexural failure seems to occur, shows quite good agreement with the analytical results.
In order to investigate the effect of dynamic response of floating ice, static stress distributions are shown at the same time as the dynamic stress in Figures 3, 5 and 6. The magnitude and the location of the static maximum stress are different from that of the dynamic stress. In the 2D infinite ice, dynamic stress distribution is larger than static one after 0.5s (see Figure 3). In the case of 3D infinite ice and 2D semi-finite ice (see Figures 5 and 6), the magnitude and the locations of the static maximum stress are larger than the dynamic maximum stress until around 0.5s. However, after 0.5s, the static peak stress and the location are smaller than for dynamic ones. Therefore, the comparison between the static solution and the dynamic results indicates that the dynamic effects of water underneath floating ice are quite important.

FE simulations of three kinds of conditions for a floating ice-sheet have been carried out as benchmark calculations. As a result of the comparison with the analytical results, FE fluid and structural simulation by ABAQUS/Explicit are capable of estimating the nature of the floating ice-sheet from the results above.

5. Response of semi-infinite wedge ice subjected to rapid loading

The influence of a free surface of water is investigated by comparing the free surface model shown in Figure 6 with a non-free surface model. The non-free surface model defines the boundary at $x=0$ to prevent a water flow into the $x<0$ region. The ice edge in which rapid load is defined has free boundary, but water is fixed at $x=0$. Figure 7 presents the comparison of displacement $d_z$ and the stress $\sigma_x$ with FEA results for a free surface and non-free surface model, respectively. The displacement and the stress distribution of non-free surface model are smaller than the free-surface results. These results naturally follow the common sense in the breaking ice phenomena, as if there is no free surface, the water underneath the ice-sheet provides more pressure to support the ice-sheet. Moreover, these results indicate that the force required to break the ice-sheet is larger when there is no free surface of sea.

The semi-infinite wedge was also analyzed with various values of the wedge angle. 12 types of wedge angles from 30 degrees to 180 degrees are used. A rapid and concentrated force at the ice edge induces a strong stress concentration and a large FE mesh deformation, and FE mesh is wrapped. A pressure load is adapted instead of a concentrated force. The pressure load is proportional to time as shown in Figure 8 (A). The example result of displacement and stress distribution of the wedge floating ice is shown in Figure 8 (B).

The effect of wedge angle is investigated. The ice thickness is 1.0 m, and loading condition is pressure load which is proportional to the time and the total force is $2.0\pi MN/s$ as shown in Figure 8 (A). And, wedge angle is also defined Figure 9 presents the displacement in the $z$-direction $d_z$ at the edge line (left figure in Figure 9) and the center line (right figure in Figure 9) of the ice-sheet at 0.3s. Figure 10 demonstrates the stress in the radius direction $\sigma_r$ at the edge line (left) and the center line (right) of the ice-sheet at 0.3s. The magnitude of the displacement and stress of the small wedge ice is larger than the large wedge ice. In the cases of the small wedge ice (30, 45, 60 degrees), the displacement of both lines agree with each other. The stress of small wedge ice at the center line is larger than at the edge line. On the other hand, in the case of large wedge ice at the edge side, the displacement adjacent to the loading point ($x=0$) shows flexural angle equals zero, which is the nature of the infinite floating ice as shown in Figures
3(A) and 4(A). Similarly, the stress distribution at the edge side of the large wedge ice shows a strong compressive stress concentration at the loading point, which is similar to the stress distributions of the infinite floating ice as shown in Figures 3(B) and 4(B). The stress distributions at the center line of each wedge angle indicate the similarities to semi-infinite ice plate, which shows a zero magnitude at the ice edge (x=0).

The relationships between wedge angle and the maximum stress, and the location of maximum stress are shown in Figure 11. The maximum stress and the maximum stress point in radius direction from the ice edge (x=0) at 0.1 and 0.3 sec are plotted. The left figure refers to the maximum stress, whereas the right figure shows the radius coordinate of maximum stress point. As the wedge angle increase, the maximum stress decrease almost exponentially. If the bending stress $\sigma_r$ is defined as the bending failure criterion and this magnitude assumes to be 1.0 MN/m$^2$, the flexural failure occurs within 0.1s in the wedge ice which is smaller than 80 degree wedge angle. In all wedge angles, the flexural failures occur after 0.3s. The maximum stress point is proportional to the wedge angle at the edge line. It is also almost constant at the center line in the case of wedge angle smaller than 130 degrees. In the small wedge, the maximum stress point of the edge line is almost same as the one of the center line. On the other hand, in the large wedge angle, the maximum stress point of the edge line is larger than the center lines. Assuming the breaking point is defined at the maximum stress point, this relationship between wedge angles and the location of maximum stress implies that ice of broken shape changes from a circle shape to an elliptical shape as the wedge angle increases.

6. Conclusions

In order to verify the applicability of fluid-structural interacting calculation of FE method, FE results are compared with analytical ones. Moreover, influences of dynamic response due to a rapid forcing, wedge angle and ice thickness are investigated. The main results of this paper include the following:
1. The fluid-structural interacting analysis by FE method is capable of estimating the bending behavior of the floating ice-sheet.
2. The dynamic effect of water pressure underneath the ice-sheet and free surface of water is remarkable.
3. The comparison between the static solution and the dynamic results indicates that the dynamic effects of water underneath floating ice are quite important.
4. The force required to break the ice-sheet in the non-free surface condition is larger than free surface condition.
5. Bending behavior of large wedge ice shows more complexity. The stress and displacement of edge lines show similar distributions of the infinite floating ice, but the distribution of the center line is evident of a semi-infinite behavior.
6. Wedge angles have an important effect on the maximum stress and on the location of maximum stress point.

References


![Figure 1](image)

**Figure 1.** Model geometry and FE mesh for semi-finite floating ice. (A) Model geometry of wedge type of a semi-infinite floating ice. (B) FE mesh of a 180 degrees wedge ice.
Figure 2. 2D infinite floating ice with line step load. (A) Schematic for 2D infinite floating ice with line load. (B) Analysis condition of FEM; number in parentheses corresponds to the number of FE element. (C) Time history of force at the center of ice-sheet with line loading.

Figure 3. Displacement (right) and stress (left) of 2D infinite plane ice model (t=0.5, 1.0, 1.5, 2.0 sec). (○:0.52[sec], □:1.00[sec], Δ:1.52[sec], ×:2.00[sec]=FEM, -Zhao and Dempsey (1996), ● static solution)
Figure 4. 3D infinite floating ice with point load. (A) Schematic for 3D infinite floating ice with line load. (B) Analysis condition of FEM; number in parentheses corresponds to the number of FE element. (C) Time history of force at the center of ice-sheet with spot loading.

Figure 5. Displacement (left) and stress (right) of 3D infinite plane ice model (t=0.2, 0.4, 0.6, 0.8, 1.0 sec). (□:0.20[sec], △:0.41[sec], ◯:0.61[sec], ○:0.80[sec], ×:1.00[sec]=FEM, -Fox and Chung (2002), ● static solution)
Figure 6. Displacement (left) and stress (right) of 2D semi-infinite floating ice (t=0.2, 0.4, 0.6, 0.8, 1.0 sec). (□:0.20[sec], Δ:0.41[sec], □:0.61[sec], ○:0.80[sec], ×:1.00[sec]=FEM, -Fox and Chung (2002), ● static solution)

Figure 7. Comparison of displacement (left) and stress (right) for free and non-free surface floating ice. (t=0.1, 0.2, 0.3, 0.4, 0.5 sec)
Figure 8. Boundary condition at ice edge, and displacement and Mises stress of wedge floating ice.

Figure 9. Displacement of wedge angle = 30, 45, 60, 80, 90, 100, 110, 120, 135, 150, 170, 180 degrees at 0.3 sec. (left side: edge line, right side: center line).
**Figure 10.** Stress of wedge angle = 30, 45, 60, 80, 90, 100, 110, 120, 135, 150, 170, 180 degrees at 0.3 sec. (left side: edge line, right side: center line).

**Figure 11.** The relationship between edge angle and the maximum stress, the maximum stress point in radius direction (left side: maximum stress in radius direction, right side: location of maximum stress in radius direction).
Numerical Simulation of Ice Conditions on the Nelson River

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The lower Nelson River is located in northern Manitoba, Canada. There are three of the largest generating stations in Manitoba Hydro’s system on this portion of the river, with a combined installed capacity of approximately 3500 MW. Currently, the station situated furthest downstream is the Limestone generating station. The characteristics of the flow regime and the nature of the river bed geometry together provide ideal conditions for very dynamic ice formations to form downstream of this station each winter. Specifically, very large areas of open water give ample time for large quantities of active frazil, anchor ice, and surface ice to form and evolve. The financial and operational implications of these river ice processes have been well documented and are on the order of a million dollars annually for Manitoba Hydro.

The focus of this paper will be on the application of the CRISSP2D model to simulate freeze-up ice conditions on a 35 km section of the Nelson River extending from the Limestone generating station at the upstream end of the model to approximately 5 km downstream of the proposed Conawapa generating station ‘B’ Axis. Model formulations and the freeze-up ice processes modeled will be presented. The numerical model will then be calibrated for open water conditions and thermal ice simulations for a chosen time period will be run. The simulated surface ice conditions, specifically the surface ice concentrations and border ice extents will be compared to field observations obtained for the same time.
1.0 Introduction
The Nelson River is located in northern Manitoba, Canada and is Manitoba’s largest and most “powerful” river draining more than one million square kilometers. Manitoba Hydro currently has three hydroelectric generating stations in the lower reach of the river with approximately 3500 MW of total installed capacity. Currently, the station located furthest downstream is the Limestone generating station but a fourth station, Conawapa, is in the planning stages and would have an installed capacity of 1485 MW. The characteristics of the flow regime and the nature of the river bed geometry together provide ideal conditions for very dynamic ice formations to form downstream of the Limestone station each winter and have been the subject of numerical model studies in the past (Malenchak et al., 2006; Liu et al., 2004). Specifically, very large areas of open water give ample time for large quantities of active frazil, anchor ice, and surface ice to form and evolve throughout the reach. Often, an ice cover will be initiated in the lower portions of the river and advance upstream towards the existing stations due to the accumulation of surface and suspended ice. The financial and operational implications of these river ice processes have been well documented and are on the order of a million dollars annually for Manitoba Hydro.

The extent of the numerical model created is shown below in Figure 1 along with the bed elevation contours. Between the current and proposed generating stations two sections of rapids exist that are of interest because of their significant impact on the ice processes in this section of the river each winter. At Sundance Rapids, a bed elevation drop of 2.5 m occurs over a length of 400 m and at the longer of the two rapid sections, Lower Limestone Rapids, bed elevation relief of 11 m is realized along a 3 km section of the river. The relative locations of these rapid sections are identified as well as the generating station axes. The model describes about 35 km of the lower Nelson River which empties into Hudson Bay about 90 km downstream of the model.

2.0 Numerical Model
CRISSP2D is a two-dimensional numerical model that couples hydrodynamic calculations with numerous thermal-ice and dynamic-ice sub-models to simulate many complex river ice freeze-up and break-up processes. The hydrodynamic sub-model is capable of simulating transitional flows and solves the two-dimensional, depth-averaged, unsteady flow equations using an explicit finite-element implementation of the streamline upwind Petrov Galerkin concept (Liu and Shen, 2003). The Galerkin finite-element method is employed to solve the transport of thermal energy equations for the ice-water mixture (Liu and Shen, 2005) and includes the solution of both water temperature \( T_w \) and suspended frazil concentration \( C_v \) as related to advection and diffusion. Mass exchanges of suspended frazil ice particles are considered to occur at both the water and bed surfaces and are modeled using a two-dimensional extension of the formulations developed for the RICEN model (Shen et al., 1995). The mass exchange of frazil ice at the water surface affects the surface ice concentration while the mass exchange at the bed surface will initiate and add to the accumulation of anchor ice on the bed material. The above calculations are combined with a Lagrangian solution of the conservation of thermal energy equations for the ice-water mixture to allow for the simulation of the supercooling process.
It has been well documented that two distinct processes govern the growth of border ice at a given location and time. The first, usually considered as static border ice growth, occurs rather quickly and is the rapid growth of border ice towards the center of the river over the calm, slower moving portions of the river near the banks. Empirical criteria necessary for static border ice growth were developed by Matousek (1984) according to field observations on the River Ore, Czechoslovakia. Calculations using these criteria are performed on the finite element nodes and if the following conditions are met, the node is considered to be static border ice; 1) the water surface temperature must be less than a critical value ($t_{cr}$), 2) local water velocity must be less than a critical value ($v_{sl}$), 3) the buoyant velocity of a frazil particle at the water surface ($v_b$) must be greater than the vertical turbulent fluctuations ($v_z$), and 4) new border ice nodes be connected to either existing border ice or river bank nodes.

The second border ice process considered is a dynamic border ice growth mechanism and is due to the accretion of floating frazil ice to either the river bank or existing border ice. This process is modeled according to the formulation developed by Michel et al. (1982) which is based on field observations on the Ste. Anne River. The dimensionless relationship for the rate of lateral growth of border ice requires that the area concentration of surface ice floes ($C_a$) be greater than 0.1 and that the flow velocity at the location of growth be less than a critical value ($v_{dyn}$). Typically, the critical velocity limiting the dynamic growth of border ice is significantly greater than the critical value used in the static border ice calculations. As well, the implementation of Michel’s theory in CRISSP2D requires that $C_a$ be greater than 0 rather than the 0.1 limit discussed elsewhere (Michel et al, 1982).

![Figure 1. Numerical model extents and water level gauge locations.](image)
3.0 Hydrodynamic Calibration

The model described above was first calibrated for open water conditions using measured water levels at the six gauges located throughout the reach. The relative locations and the names of each of the gauges can be found in Figure 1 above. A one week simulation was run using data from August 11, 2005 @ 12:00 am to August 18, 2005 @ 12:00 am. The measured flow through the Limestone Generating Station was used to set the upstream flow boundary condition and a seventh gauge (05UH769) was used to set the downstream level boundary condition. This time period was chosen because good data was available from all the gauges and it was during the open water season prior to the winter that we intended on conducting our thermal ice simulations. It was also determined that due to the morphology of this section of the Nelson River and the relatively short growing season at this northern location, the effects of summer vegetation growth on the calibrated bed roughness values would be negligible.

Comparisons between the measured and modeled water levels at two of the gauges used for calibration were selected to be shown in Figure 2 below. These gauges were 05UH740, which is located just upstream of Sundance Rapids along the north side of the river and 05UH702, which is located about 400 m upstream of the Conawapa ‘B’ axis along the south side of the river. From the figure below it is quite clear that the measured and modeled water levels at these two locations match quite well. For gauge 05UH740, the average difference between the measured and modeled values is -0.015 m with a maximum deviation of 0.250 m for the one week simulation. Likewise at gauge 05UH702, the average difference is 0.020 m with a maximum deviation of -0.200 m.

![Figure 2](image-url). Comparison of measured and modeled water levels at 05UH740 (left) and 05UH702 (right).

The above calibration resulted in a range of values being specified for the Manning’s bed roughness coefficient, \( n \). The \( n \) values used in the model ranged from a minimum of 0.025 in the tailrace section just downstream of the Limestone Generating Station to a maximum value of 0.0535 in the Sundance Rapids area. The large value specified in the area of Sundance Rapids is
required to capture the amount of head loss experienced at this location. Similar results to those illustrated in Figure 2 were obtained for the other four gauges used in the open water calibration. As well, a second data set for later in this open water season (September, 2005) was used to verify the above calibration and was also successful with similar results.

Figures 3 and 4 below were plotted at hour 74 of the calibration simulation. The flow at this time was approximately 6250 cms. From the two-dimensional velocity contours shown in Figure 3, the spatial distribution and range of water velocities throughout the study reach are clear. The slowest flowing section of the reach is just downstream of Sundance Rapids with an average velocity of 1.26 m/s which coincides with a Froude number of 0.15 in this area. In contrast, velocities reach as high as 5.70 m/s in the Lower Limestone Rapids area, which correspond to supercritical Froude numbers as high as 1.51 in isolated areas. A longitudinal profile along the thalweg of the river was generated and is shown in Figure 4. From this plot, the relief along the various sections of the river is clear along with the different water surface slopes that correspond to each of these sections. The range and spatial distribution of depth and velocity throughout this section of the river will be important considerations when simulating the surface ice conditions below.

**Figure 3.** Two-dimensional velocity contours for open water calibration, Q = 6250 cms.
4.0 Thermal Ice Simulations

Manitoba Hydro has monitored this reach extensively since 1997 and has conducted a winter observation program at various times throughout the ice season depending on the variability of the ice conditions at that time. Chosen for the initial simulation of surface ice conditions below was a time period surrounding March 3, 2006. This time period was chosen because a detailed monitoring report was available at this time and the combination of the flow and weather conditions for the 2005-06 winter suppressed the formation of ice cover in the lower sections of the Nelson River all the way to Hudson Bay.

At the upstream end of the model, the flow through the Limestone Station was used along with a constant inflow water temperature of 0.02 °C. This temperature has been measured in the tailrace channel and considered valid provided an ice cover exists on the forebay of the station. The absence of an ice cover downstream of this reach allows for the downstream level boundary to be specified using an open water rating curve for that location. Simulated ice conditions will be compared to photographs taken between 12:00 and 14:00 hrs on March 3, 2006. The simulations will be started a sufficient time prior to this to allow for the dynamic surface ice processes to reach a stable condition.

4.1 Water Temperature and Frazil Concentration

The contours of frazil concentration and water temperature for March 3, 2006 @ 12:00 hrs are shown in Figure 5. Quality observed data for both the water temperature and frazil concentration are not yet available but the ability to obtain reasonable results is still important. An important parameter related to this portion of the simulation includes the specification of a linearized heat flux coefficient ($h_{wa}$) for the air-water interface. A value of $h_{wa} = 20$ W/(m$^2$°C) is used for this simulation and is consistent with the value used in previous studies on portions of this same river (Malenchak et al, 2006). From the figure it can been seen that the location of maximum
supercooling is near the Sundance Rapids area and the increasing frazil concentration downstream helps to bring the water temperature closer to 0°C for much of the reach. Slightly higher amounts of supercooling can be found at the Lower Limestone Rapids area as well as the area just upstream of the Conawapa ‘B’ axis. This coincides with areas of elevated water velocity as well. Elevated frazil concentrations are found just downstream of these two areas, which is an expected result as again the water-ice mixture moves toward a state of thermal equilibrium. Some level of supercooling exists throughout much of this reach at least in part due to the suppression of a competent ice cover and continued heat transfer away from the water. The active frazil concentrations that exist because of this show a potential for anchor ice growth throughout the reach contingent on two factors: 1) turbulent mixing is high enough to bring suspended frazil down to the bed surface, and 2) bed particles are stable enough to withstand the buoyant and drag forces of the attached frazil particles. It has been observed throughout the monitoring program that anchor ice accumulations do exist at numerous locations throughout this reach often in the rapid sections where bedrock outcrops provide the competent bed material for the anchor ice to accumulate.

![Figure 5](image-url)

**Figure 5.** Contours of water temperature (left) and frazil concentration (right), Q = 4600 cms.

### 4.2 Border Ice Extents

As mentioned above, border ice grows outward from the banks of the river by two distinct mechanisms, both of which are considered within this numerical model. Border Ice extents calculated by the numerical model are shown below in Figure 6 with specific areas highlighted for discussion and comparison purposes. Because of the dynamic nature of the accretion of frazil ice pans to areas of existing border ice, the simulation length was extended to ensure that a stable extent of border ice was obtained. As mentioned in the discussion of the model formulation, three critical parameters are required to be set when simulating border ice conditions in CRISP2D: a critical surface water temperature (t\_c) of -0.25°C, a critical velocity for static border ice formation (v\_st) of 0.4 m/s, and a critical velocity for the dynamic border ice formation (v\_dyn) of 1.2 m/s were found to be applicable to this reach and good results were obtained from the simulations.
Figure 6. Simulated border ice extents on March 3, 2006 – 12:00 hrs.

In the figures below, photographs obtained from the monitoring report dated March 3, 2006 will be presented to compare the simulated border ice extents above to those observed at this time. The numbered areas on the photographs correspond to the same areas identified above in Figure 6. Areas 1 and 2 are identified in Figure 7 below and the border ice in area 1 does match up quite well with observed extents and it should be noted that the island is included in the simulated border ice locations as it was a part of the finite element mesh. If one is to look at the simulated border ice found in area 2, it does not match up exactly with the large amount of snow covered area seen in Figure 7. This comparison though is actually much more favorable than it looks at first glance because much of the snow covered area identified in area 2 of Figure 7 was left out of the finite element mesh because of the high elevations at these locations. The simulated extent of the border ice combined with the relative location of the mesh boundary does provide a relatively favorable comparison with observed data.

Border ice areas identified as 3 and 4 are highlighted in Figure 8. The simulated extents of the border ice in this area on both sides of the river correspond quite closely to the observed extents seen in the photograph. Again, the small island visible in area 4 has been included in the finite element mesh and is tagged as border ice in Figure 6.
Figure 7. Observed border ice locations near Lower Limestone Rapids.

Figure 8. Observed border ice locations between Lower Limestone Rapids and the Conawapa ‘B’ axis.
A final comparison of border ice extents can be shown for the area identified as area 5 in Figure 6 as well as for the area just upstream of the Conawapa ‘B’ axis along the north shore of the river. Because of the angle of the photograph shown in Figure 9, it is tough to identify exactly the extent of the border ice found in area 5. It can be concluded though that the border ice in this area seems to fill much of the bay and extends to the edge of the main flow area. The simulated location of border ice in this area was found to be very much the same (Figure 6). It can also be seen from Figure 9 that border ice does exist along the north bank of the river at the Conawapa ‘B’ axis and upstream of this location. Small amounts of border ice are simulated at these locations in Figure 6 but the extents here are underestimated by the model under the current configuration.

![Image](image-url)

**Figure 9.** Observed border ice extents near the Conawapa ‘B’ axis.

### 4.3 Surface Ice Concentration ($C_a$)

The simulated surface ice concentrations were compared to observed estimates of the surface frazil ice coverage documented within the report. These estimates were made by experienced field personnel with extensive knowledge of this reach of the river and were expressed as an approximate areal percentage. It was stated in the report that for the section of the river extending from 4.5 km upstream of the Lower Limestone Rapids to 8.5 km downstream of the rapids, surface frazil ice covered approximately 25% of the open water area. It was also reported that the surface frazil ice concentration increased to about 30-40% in the reach downstream of the Conawapa ‘B’ axis. The simulated surface ice concentrations are shown in Figure 10. It can be seen that these simulated concentrations match up reasonably well with the estimated observed values if regionally averaged values are considered. The transport of suspended frazil ice between the suspended and surface layers is controlled largely by two parameters in
In order to obtain the presented concentrations, the empirical constants $\alpha$ and $\beta$ were set to be 0.01 and 0.25 respectively. With $\alpha$ controlling the rate at which suspended frazil adds to the surface ice concentration at the surface and $\beta$ regulating the amount of frazil contained in the existing floes that gets entrained back into the water column. It should be noted that much of the surface frazil ice mentioned above is not visible in many of the photographs (Figures 7–9) largely due to the cloud cover conditions that day, the resolution of the photograph and the angle the photo was taken.

**Figure 10.** Surface ice concentration ($C_a$) contours on March 3, 2006 – 12:00 hrs.
5.0 Conclusions
A CRISSP2D model was hydrodynamically calibrated for a section of the lower Nelson River extending approximately 35 km from the Limestone generating station to about 5 km downstream of the proposed Conawapa generating station ‘B’ axis. The ability of the model to simulate various freeze-up ice processes was then evaluated. This included water temperature, suspended frazil concentration, surface ice concentration, and the extent and location of border ice growth. When possible, simulated results were compared to observed field conditions that were collected by Manitoba Hydro for their winter monitoring program. In general, good comparisons were made between the observed and simulated conditions for most sections of the reach. Further refinement of the model formulations that currently exist and validation of the chosen model parameters will be a part of the continuing research program.

6.0 Acknowledgements
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7.0 References


Thermodynamic Modelling to Test the Potential for Anchor Ice Growth in post-construction conditions on the Nelson River

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There are several locations on the Nelson River in northern Manitoba that are susceptible to the growth of anchor ice. Two of these locations are of particular interest to Manitoba Hydro due to their close proximity to the existing Limestone Generating Station and the future Conawapa Generating Station. Anchor ice growth at these locations can result in elevated tailwater levels over the winter period, directly impacting the winter generation capacity of each station. The ability to predict the potential for anchor ice formation at these two locations, once the Conawapa station has been constructed, is an important aspect in the new station’s planning process.

This paper outlines the results of an investigation which used CRISSP2D’s two-dimensional thermodynamic ice model at these two locations. CRISSP2D was applied to simulate the river’s post-Conawapa thermal ice regime under winter conditions. This information was then used to provide guidance to planners on possible anchor ice growth/effects at these two locations. The first model extended approximately 4.5 km from the existing Limestone GS to a location downstream of Sundance Rapids which is approximately 26 km upstream of the Conawapa axis. Currently, anchor ice forms across Sundance Rapids every winter, but will be flooded by an additional 4.5m of water once the Conawapa station is constructed. It has been assumed that once impounded, anchor ice will no longer form. This study will attempt to confirm these assumptions. The second model was located in the proposed Conawapa tailrace channel. Anchor ice has been observed to form on a shoal approximately 1100 m downstream of the station axis. This study will help identify the potential for anchor ice to continue to form once the generating station is constructed and operational.
1.0 Introduction

The Nelson River is Manitoba’s largest and most powerful river, draining more than one million square kilometres. Manitoba Hydro, a provincially owned utility company, has three hydroelectric generating stations in the lower reaches of the river with approximately 3500 MW of installed capacity. A fourth generating station, Conawapa, is in the planning stages and would have a capacity of 1485 MW.

There are several locations on the Nelson River that are susceptible to the growth of anchor ice. Two of these locations are of particular interest to Manitoba Hydro due to their close proximity to the existing Limestone Generating Station and the future Conawapa Generating Station (Figure 1). Anchor ice growth at these locations can result in elevated tailwater levels over the winter period, directly impacting the winter generation capacity of each station (existing and proposed).

The ability to predict the potential for anchor ice formation at these two locations, once the Conawapa plant has been constructed, is an important aspect in the station’s planning process. Although numerical models exist that can simulate most river ice formation processes such as frazil ice generation, border ice growth and ice cover advancement, the theory surrounding the formation and evolution of anchor ice is still evolving. CRISSP2D simulates the formation and evolution of anchor ice according to the theory presented in Shen et al. (1995).

Figure 1. Location of study area.
This paper outlines the results of an investigation which used CRISSP2D’s two-dimensional thermodynamic model at these two locations. The model was applied to simulate the river’s post-Conawapa thermal ice regime under winter conditions along this river reach. This information was then used to provide guidance to planners and operators on possible anchor ice growth/effects at these two locations.

The first model extends approximately 4.5 km from the existing Limestone GS to a location downstream of Sundance Rapids which is approximately 26 km upstream of the Conawapa axis. Currently, anchor ice forms across Sundance Rapids every winter, but this area will be flooded by an additional 4.5 m of water once the Conawapa station is constructed. It has been assumed that once impounded, anchor ice will no longer form. This study will attempt to confirm these assumptions.

The second model is located in the proposed Conawapa tailrace channel. Anchor ice has been observed to form approximately 1100 m downstream of the station axis. This study will help identify the potential for anchor ice to continue to form once the generating station is constructed and is operating.

2.0 CRISSP-2D Numerical Model

CRISSP2D is a two-dimensional numerical model that couples hydrodynamic calculations with numerous thermal-ice and dynamic-ice sub-models to simulate many complex river ice freeze-up and break-up processes. The hydrodynamic sub-model is capable of simulating transitional flows and solves the two-dimensional, depth-averaged, unsteady flow equations using an explicit finite-element implementation of the streamline upwind Petrov Galerkin concept (Liu and Shen, 2003). The Galerkin finite-element method is employed to solve the transport of thermal energy equations for the ice-water mixture (Liu and Shen, 2005) and includes the solution of both water temperature ($T_w$) and suspended frazil concentration ($C_v$) as related to advection and diffusion. Mass exchanges of suspended frazil ice particles are considered to occur at both the water and bed surfaces and are modelled using a two-dimensional extension of the formulations developed for the RICEN model (Shen et al., 1995). The mass exchange of frazil ice at the water surface affects the surface ice concentration while the mass exchange at the bed surface will initiate and add to the accumulation of anchor ice on the bed material. The above calculations are combined with a Lagrangian solution of the conservation of thermal energy equations for the ice-water mixture to allow for the simulation of the supercooling process.

It has been well documented that two distinct processes govern the growth of border ice at a given location and time. The first, usually considered as static border ice growth, occurs rather quickly and is the rapid growth of border ice towards the center of the river over the calm, slower moving portions of the river near the banks. Empirical criteria necessary for static border ice growth were developed by Matousek (1984) according to field observations on the River Ore, Czechoslovakia. Calculations using these criteria are performed on the finite element nodes and if the following conditions are met, the node is considered to be static border ice; 1) the water surface temperature must be less than a critical value ($t_{cr}$), 2) local water velocity must be less than a critical value ($v_{st}$), 3) the buoyant velocity of a frazil particle at the water surface ($v_b$) must
be greater than the vertical turbulent fluctuations \((v_z')\), and 4) new border ice nodes must be connected to either existing border ice or river bank nodes.

The second border ice process is a dynamic border ice growth mechanism and is due to the accretion of floating frazil ice to either the river bank or existing border ice. Although CRISSP2D is capable of modelling this process, its contribution to the intended results was not significant and therefore this process was not modelled in the simulations for this paper.

3.0 Study Area 1: Sundance Rapids

3.1 Existing Conditions

The Limestone Generating Station is currently the largest station in Manitoba Hydro’s system with an installed capacity of 1330 MW. About 3 km downstream of the station is Sundance Rapids, a short set of rapids with approximately 2.5 m of head loss. A combination of the shallow flow depths across the rapids, the daily cycling nature of discharges, and the extreme cold air temperatures (daily average temperature between -20°C and -30°C during the winter) leads to the formation of anchor ice across 80 to 90% of the river width as shown in Figure 2. This restriction produces a 1-2 m increase in tailwater levels at the generating station resulting in energy losses of more than $2 000 000 annually.

The proposed Conawapa Generating Station, located approximately 30 km downstream from the Limestone GS, will result in a 4.5 m increase in water level at Sundance Rapids. Figure 3 illustrates the water surface profiles before and after construction of Conawapa during open water conditions. Planners have assumed that the anchor ice across Sundance Rapids will no longer form once Conawapa has been constructed. This paper tests that assumption using the CRISSP2D model. Due to the limitations of current anchor ice theory, the results from this study will be used more as a qualitative assessment of post-Conawapa ice conditions than a precise prediction.

3.2 Model Set-up

3.2.1 Open Water Calibration

A CRISSP2D model was set up from the tailrace of the Limestone GS to approximately 1.7 km downstream of Sundance Rapids. The model was calibrated under open water conditions using the dynamic hourly discharges from the generating station, water level gauges in the generating station tailrace, water level gauges immediately upstream and downstream of the rapids, and a rating curve at the downstream boundary of the model. A more detailed discussion of the open water calibration can be found in Malenchak et al (2006).

3.2.2 Modelling Parameters

For each simulation, the downstream boundary was set at 56.7 m, which was the assumed full supply level for the Conawapa GS forebay. The incoming water temperature for each simulation
was set at a constant value of 0.02°C. This temperature has been confirmed with limited field measurements of existing conditions.

**Figure 2.** Anchor ice formation across the Nelson River at Sundance Rapids in February, 2008.

![Anchor ice formation across the Nelson River at Sundance Rapids in February, 2008.](image)

**Figure 3.** Existing and post-Conawapa water surface profiles.

![Existing and post-Conawapa water surface profiles.](image)

Although the hydrodynamic model was calibrated using existing open water conditions, the thermal-ice simulations were run under significantly different conditions (*ie.* 4.5 m additional water depth under post-Conawapa conditions). Therefore, as a proxy for calibrating the border ice formation processes under post-Conawapa conditions, a series of simulations were run using
different critical border ice parameter values for the critical water surface temperatures and velocities. Table 1 shows the combinations of border ice parameters that were used. These parameters were chosen as a combination of values used in previous studies and conservative estimates of the parameter values. The sensitivity of the model results to these parameter values will be illustrated in the results section below. For each set of critical temperature and critical velocity, three simulations were carried out using different historical flow and temperature patterns. It was assumed that the operation of the Limestone GS would be similar under post-Conawapa conditions; therefore three years of historical discharge patterns (Year 1, Year 2, Year 3) from the Limestone GS, shown in Figure 4, were used as the inflow boundary for the model. Meteorological conditions obtained for the corresponding years were also used in the simulations.

**Table 1.** Critical parameters for border ice growth.

<table>
<thead>
<tr>
<th>Set</th>
<th>Critical Border Ice Velocity (m/s)</th>
<th>Critical Water Surface Temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Set 1 (Base Case)</td>
<td>0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Set 2</td>
<td>0.17</td>
<td>-0.5</td>
</tr>
<tr>
<td>Set 3</td>
<td>0.09</td>
<td>-0.5</td>
</tr>
<tr>
<td>Set 4</td>
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<td>-1.1</td>
</tr>
<tr>
<td>Set 5</td>
<td>0.25</td>
<td>-0.8</td>
</tr>
</tbody>
</table>

**Figure 4.** Sample of the upstream inflow boundary conditions.
3.3 Results
Although the different border ice parameters affected the freeze-up dates, the discharge at the
time of freeze-up had a significant effect as well. The discharges for Year 1 and Year 2 were
both relatively high with up to 3000 m\(^3\)/s of daily cycling and as a result, the freeze-up dates
were quite similar for both years as shown in Table 2. The discharges for Year 3 were
significantly less with only 1300 m\(^3\)/s of daily cycling. These reduced discharges caused the
river to freeze-up almost two weeks earlier. The correlation between the freeze-up date and the
air temperature was not as strong as that observed with the station discharge provided the
overnight air temperature was more than a few degrees below zero.

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Freeze-up Dates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year 1</td>
<td>Nov. 11-20</td>
</tr>
<tr>
<td>Year 2</td>
<td>Nov. 10-24</td>
</tr>
<tr>
<td>Year 3</td>
<td>Oct. 27-Nov. 02</td>
</tr>
</tbody>
</table>

The purpose of this modelling was not to predict the exact date of freeze-up, but rather the
potential for anchor ice to form across Sundance Rapids under post-Conawapa conditions.
Although the freeze-up dates vary for each combination of tested border ice parameters,
discharges and meteorological conditions, the modelling indicates that the river should freeze
over under post-Conawapa conditions, limiting the potential for anchor ice formation.

In each simulation, the model showed that during the open water periods, before the ice cover
formed, anchor ice was initiated. But once the ice cover formed, the growth of anchor ice
stopped and the existing anchor ice started to rapidly decay. In all simulations, there were no
significant increases in tailwater levels at the Limestone GS during winter conditions except that
which would be normally expected for stable ice cover conditions.

4.0 Study Area 2: Downstream of the Proposed Conawapa Generating Station

4.1 Existing Conditions
Due to the steepness of this reach, typical ice covers in this study area form from the continued
collapse/consolidation of surface ice floes. Under normal winter conditions, large volumes of ice
(up to 10 m thick) cover the study area starting in mid-December. During years of extremely
high river discharges, the ice cover formation in the reach may be delayed or not form at all as
the reach stays open all the way to Hudson Bay. Under these conditions, anchor ice has been
observed to form in the area that will be part of the tailrace channel for the proposed Conawapa
GS (Figure 5).

Although the location of observed anchor ice will be excavated as part of the Conawapa tailrace
channel, planners need to know if it may still be possible for anchor ice to form in either the
tailrace, or in the area immediately downstream of the tailrace, which would cause head losses
(and energy losses) similar to the existing Sundance Rapids problem.
4.2 Model Set-up

4.2.1 Open Water Calibration

A CRISSP2D hydrodynamic model was set up which extended from 3 km upstream of the proposed Conawapa GS to approximately 4.25 km downstream of the axis. The model was then calibrated against a previously calibrated HEC-RAS model that has been used for engineering and environmental studies at Manitoba Hydro. After calibration, the bathymetry file was modified to include the excavated tailrace channel of the proposed station. The entire model was then truncated so that the upstream boundary coincided with the axis of the station.

4.2.2 Modelling Parameters

Once the model was calibrated for open water conditions, two full seasons of historical discharges and meteorological data were modelled. One season was characterized by high discharges, little cycling, and warmer air temperatures (relative to the second season). The other season had average discharges, daily cycling, and cooler air temperatures. For each simulation, the downstream water level boundary was set using an open water rating curve developed for that location. The incoming water temperature for each simulation was also set at a constant 0.02°C, which is consistent with the Limestone station simulations above.

Figure 5. Aerial photo with proposed structures and photos of anchor ice.
4.3 Results
Examination of the water temperature distribution, specifically the thermocline, indicated there may still be the potential for anchor ice to form in the excavated channel. With warmer air temperatures and higher discharges, the thermocline is located outside the excavated tailrace channel. This would indicate that the potential for anchor ice to initiate in the channel is quite low under these conditions. However, under cooler air temperatures and lower discharges, the thermocline exists within the excavated channel, which indicates that there is a greater potential for anchor ice initiation in the tailrace channel. Even though anchor ice may be initiated at these locations, the ability of the bed material to sustain large thicknesses of anchor ice must be considered. Anchor ice is subjected to considerable drag and buoyancy forces, and without competent bed material (as that found in the Sundance Rapids area), significant thicknesses of anchor ice cannot be realized and the effect on the flow conditions would be minimal. In addition, when the thermocline is located near the end of the channel, minimal amounts of supercooling and suspended frazil concentrations are found in the channel and in the area immediately downstream of the channel which would further reduce the potential for significant amounts of anchor ice to form in this area. A final consideration would be whether the level of turbulence downstream of the tailrace channel would be significant enough during post-construction conditions to bring suspended frazil concentration into contact with the bed material. A detailed winter season calibration of the model and the incorporation of a vertical frazil ice distribution would be required to provide a definitive answer to this question, both of which are currently not available at this time.

5.0 Conclusions
Using the CRISSP2D model, the assumption that anchor ice will no longer form across Sundance Rapids under post-Conawapa conditions was tested. Although the anchor ice modelling capabilities of the model do not allow for a definitive answer to the question, the results provided additional confidence that an ice cover will form early in the winter season, preventing significant growth of anchor ice across the rapids. The modelling downstream of the proposed Conawapa GS indicated that for certain conditions, there may still be the potential for anchor ice initiation based on the water temperature and suspended frazil ice concentration contours alone. Other characteristics of the channel and flow conditions will need to be considered when assessing whether or not anchor ice will be able to accumulate to levels capable of impacting the flow conditions by a significant amount.
6.0 References


A Numerical Model Study on Ice Boom in a Lake-Harbor System

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Lake Notoro near the coast of Okhotsk Sea is connected to the Sea by the Notoro Fish Harbor. The Lake has salinity similar to that of the Okhotsk Sea. It is ideal for aquaculture of scallops, oysters and other marine products. During the winter, ice floes drift into the Lake through the channel connecting to the Sea which causes damage to the aquaculture facilities. An ice boom has been proposed to be installed near the entrance of the Lake to prevent the ice moving into the Lake. In this paper, a numerical model study on the dynamic transport of ice, the effect of the boom, and the ice load on the boom spans are presented. The numerical model is a coupled hydrodynamic and ice dynamic model. The ice dynamics component uses a Lagrangian discrete parcel method based on smoothed particle hydrodynamics. The hydrodynamics component uses a finite element method for shallow water hydrodynamics. Results for steady and unsteady simulations for various wind, ice, and tidal conditions are presented.
1. Introduction

Lake Notoro is located on the northeast coast of Hokkaido, Japan. It is connected to the Okhotsk Sea by the Notoro Fish Harbor (Figs. 1 & 2). The Lake has salinity similar to that of the Okhotsk Sea. It is ideal for aquaculture of scallops, oysters and other marine products. In winters, ice floes drift into the Lake through the channel connecting the lake to the sea which causes damage to the aquaculture facilities. An ice boom has been proposed to be installed near the entrance of the Lake to prevent ice moving into the Lake, which can also cause significant salinity changes. A numerical model study on the dynamic transport of ice, the effectiveness of the boom, and the ice load on the boom spans, is presented.

Figure 1. Lake Notoro and Hokkaido, Japan

1.1. Mathematical model

The DynaRICE model is a two-dimensional ice dynamics model for analyzing dynamic transport of surface ice in rivers and lakes (Shen et al. 2000). The model simulates the coupled dynamics of ice motion and water flow, including the flow through and under the ice rubble. It has been successfully applied to river and sea ice studies, e.g. Shen et al. (1997), Liu and Shen (1998), Wang, et al. (2000), and Shen and Liu (2003). In the model, hydrodynamics is simulated by solving depth-integrated two-dimensional hydrodynamic equations for shallow water flows
including the effects of surface ice. Flow in the surface ice layer is included, so that flow between floes and the flow through ice accumulations are considered. The hydrodynamic component of the model has recently been improved using a finite element method based on the streamline upwind Petrov–Galerkin (SUPG) concept (Liu and Shen 2003). This method is capable of simulating trans-critical flows with wet-dry bed conditions.

**Figure 2.** Lake entrance area, ice condition and proposed ice boom site

The ice dynamic equations consider all the external and internal forces. These forces include water drag, gravity force, internal resistance of ice, and bed friction on grounded ice. Viscoelastic-plastic (VEP) constitutive law was used to formulate internal ice resistance, which is governed by the material behavior of ice rubble (Ji et al. 2004). The pressure term is formulated using a modified form of the Coulomb law. A Lagrangian discrete-parcel method (DPM) is used to simulate the dynamics of the surface ice transport. This method is derived from smoothed particle hydrodynamics (SPH) (Monaghan 1992). The general idea of the DPM method is to describe the surface ice rubbles in the form of sufficiently large number of individual parcels carrying mass, momentum and energy (Shen et al. 2000). The partial slip boundary condition for ice friction along solid boundaries is treated using the method of images. The bed friction on ice movement when ice grounding occurs is also considered.

**1.2. Simulation conditions**

The bathymetry of the entire lake and sea shore (HTRCCR 2006) is shown in Figure 3. Figure 4 shows the domain and finite element mesh of the model, which covers the northern part of the Lake including both the channel between breakwaters and the proposed ice boom. The total length of the model in the North-South direction is about 2 km. Some limited current and water level data were available. Discharge was measured inside the harbor and the water level was measured on the sea side and in the middle of Lake Notoro. All these data were measured during a two-day period in August 2002. No ice condition or meteorological data were available. In all simulations, ice was supplied throughout the simulation period from the sea side boundary. The ice supply was assumed to have a concentration of 0.3 and a thickness of 0.2 m. The ice roughness coefficient is expressed in terms of Manning’s roughness coefficient, ranging from a minimum value of \( n_{\text{min}} = 0.02 \) for single layer ice to a maximum value of \( n_{\text{max}} = 0.06 \). The ice
roughness increases linearly with thickness from the minimum value for single layer ice to its maximum value.

An ice boom is designed to control the ice entering the Lake. The ice boom is modeled as a barrier with 15 spans. The middle spans (No. 6 to 10) have a length of 28 m each. Each of the spans on the west and east sections (No. 1 to 5 and No. 11 to 15) have a length of 24 m. To simulate the boom effects, a number of parameters must be specified. The model can simulate booms with defined draft depth. However, since the design draft is not yet decided, the present simulations assume the boom is of the floating pontoon type. The parameter responsible for surface ice submergence at the boom is a Froude number defined by the local depth-averaged flow velocity and depth. If the limiting Froude number is exceeded, ice will overturn and transport to downstream. The limiting Froude number for overturning was set as 0.09. When the ice accumulation against the boom grows, its thickness may be limited by the erosion of the ice on the underside of the accumulation. In the simulations, the erosion velocity limit was set at 1.5 m/s. An additional mechanism for ice to pass the boom is boom submergence. If the ice load exceeded the critical value of cable tension the boom will submerge and ice will spill over the boom. For all simulations cases, a high critical value of boom line load of 10 KN was assumed. This will allow for the determination of the required design load of the boom spans.

2. **Constant inflow simulations**

The constant inflow discharge was estimated based on the few measured velocity data available. This inflow discharge from the sea was estimated to be 1200 m$^3$/s. The water level in the lake was assumed to be constant at 0.8 m above the sea level, based on the field data (HTRCCR 2006). A steady ice load condition was achieved after a period of about 12 hours. Simulated ice and current distributions at the end of simulation period are presented in Figures 5.

Wind measurements were made over a period of 5 years at Notoro Harbor. The average value of maximum monthly wind velocity for the five years from 1999 to 2004 was 16.2 m/s. The highest wind velocity observed during this period was 28.7 m/s. The general direction of the maximum measured wind was from the North (from the sea towards the lake). The simulation was run with a conservative condition assuming a wind velocity of 25 m/s with a 12º orientation (0º means wind blowing from North), which is along the channel between the breakwaters of the Notoro

**Figure 3.** Bathymetry of Lake Notoro

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![Bathymetry of Lake Notoro](image)
Harbor and normal to the boom. Simulation results with this wind condition at the end of the 72 hour simulation period are presented in Figure 6.

Figure 4. Model domain and finite element mesh

Figures 5 and 6 showed that the bathymetry and current in the breakwater channel caused ice to move towards the Southwest in the channel. However, wind drag acting from the North East caused more ice turning toward the East in the channel. After the ice passes the narrow part of the channel and enters the lake, the ice transport is significantly affected by the wind. The wind driven current from the east affects the ice transport, especially beyond the boom. For both the cases with and without wind, the water level change over the 2 km distance between the harbor entrance on the sea side and the downstream lake side boundary of the simulation domain is less than 0.2 m (i.e. a water surface slope of less than 0.0001). Results showed that the plane geometry of the breakwaters, i.e. the narrow exit of the harbor at about 800 m from the entrance, caused slowing and accumulation of ice rubble. However, ice did not jam in the narrow section, but rather moved into the lake until finally stopped by the ice boom. The ice thickness profiles showed that the ice accumulations are thicker in the harbor than in the lake.
Figure 5. Ice thickness and under ice current velocity at hour 72, no wind effect.

Figure 6. Ice thickness and under ice current velocity at hour 72, with wind effect.

Figure 7 shows comparisons of longitudinal ice thickness and water surface profiles as well as along the boom. Because of the relatively low water velocity, which is less than 1.0 m/s, ice did not pass the boom by underturning or erosion from the bottom of the accumulation. Since a high value of limiting boom line load was used, ice load on the boom did not exceed the 10 KN limit in either case and almost all of the ice remained behind the boom. A small portion of the ice passed the boom by flowing around the openings on its East and West ends. Figure 8 compares the distribution of ice force on the boom spans for both the conditions with and without wind. The force acting on East and middle boom spans in the ‘with wind’ case is higher than in the case without the wind effect. This is the effect of additional drag force from the wind. In the case with no wind effect, a maximum force of 1.73 kN/m was observed at span No. 5. In the case with wind effects, ice reaches the boom earlier, but took a little longer to reach the maximum boom load. The peak load of 1.94 kN/m, which is about 15% higher than that of the case without wind, occurred at span No. 5. This showed that the main mechanism affecting ice movement is the
water drag. The wind drag plays a less significant role on ice transport in comparison to the water drag.

Figure 7. Comparison of variations of ice thickness, water level, and under ice velocity: a) longitudinally and b) along the boom

Figure 8. Effect of wind on force distribution along the boom at hour 72, when the force reached its maximum

Figure 9. Water level measured at Okhotsk sea and Lake Notoro and discharge measured in Notoro Fish Harbor.

3. Simulations with tidal effect
The tidal level was measured over a period of 48 hours (HTRCCR 2006). The data shown in Figure 9 indicated that the tide has a period of 24 hours, and the amplitude is about 1.26 m. During this same period, the water discharge was measured inside the harbor at the narrow section of the channel, and the water level was measured at the center of the Lake Notoro (see Figure 3). All these data were used as the basis for setting up boundary conditions for the simulation.
Since the spatial distribution of water discharge at the lake side boundary of the model domain shown in Figure 4 was not available, it was necessary to run a simulation with an extended domain, as shown in Figure 10, to determine this boundary condition. For the extended domain simulation, the boundary conditions used were: measured water level on the lake side boundary, and measured water discharge on the sea side boundary at the narrow section of the harbor. Since the distribution of the water discharge was not available, the stream-tube method (Shen and Ackermann 1980) was used to determine the correct boundary discharge distribution here. The simulation result provided the flow distribution at the lake side open boundary. This together with the observed water level data on the seaside boundary as shown in Figure 11 were used for ice simulation in the model domain given in Figure 4. The simulated boundary condition data in Figure 11 were repeated to extend the time period to generate 432 hours or 18 days of boundary data for the ice transport simulation. To examine the wind effect, the same wind condition as in the constant water discharge case was used.

Figure 10. Extended model domain for initial simulation; line A-A represents lake side open boundary for proper model.

The simulation results showed a significant effect of tide on ice transport and accumulation. The water level change along the longitudinal axis of the domain varied between 0.2 m and 1.3 m. Periodic changes of the water level and discharge caused oscillatory movement of the ice into and out of the domain. The accumulated ice cover is thinner than in the constant inflow case but thicker ice accumulation along the narrow part of the channel still exist. Higher tidal level raised the water depth on both the east and west ends of the boom. This caused more ice flow around the boom and into the lake, and reduced the load on boom spans on the east and west ends.

Figure 12 shows the boom loads during the first 10 days of the simulation period for both cases with and without wind. After an initial period of about 100 hours the force acting on the boom reaches a relatively stable periodic mode, modified slightly by ice motion irregularities. Forces
are acting on the boom only during the 12 hour high tide periods when the water and ice are flowing toward the boom. Figure 13 shows the comparison of the maximum load on the boom spans for with and without wind conditions.

**Figure 11.** Boundary conditions during each 24 hour cycle of simulation.

**Figure 12.** Forces on the boom spans during the first 10 days of the simulation: a) without wind effect; b) with wind effect

**Figure 13.** Effect of wind on the maximum load on the boom spans under tidal condition.
4. Conclusions

This study was performed to evaluate the efficiency of the ice boom designed for Lake Notoro to protect the Lake water quality for aquaculture. Simulations were made using the ice dynamics model DynaRICE to analyze the ice load on an ice boom to be designed to control sea ice flow into Lake Notoro. Four different scenarios, including wind and tidal effects, were examined. Simulation results showed that ice will be retained by the boom with some bleeding around the west and east ends of the boom. In addition to the retention capability, ice load acting on the boom was evaluated. The ice load distributions along the boom for all four conditions simulated are presented in Figure 14. This figure shows that wind can change the distribution of the boom load and increase the maximum load. The peak loads for the cases with tidal effect are much higher than the cases where tidal effects were neglected. This shows that, in analyzing the design condition, the tidal effect should be included due to the higher peak load.

![Comparison of ice load distribution along the boom for all simulated cases](image)

**Figure 14.** Comparison of ice load distribution along the boom for all simulated cases

References


Modeling Grounded Ice Jams using the Ice Jam Force Balance Equation

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It is well known that wide river ice jams can become grounded, especially near the downstream end, or toe, of the jam. The geometry of wide rivers jams has been successfully modeled by solving an ice jam force balance equation assuming that the ice floes comprising the jam act as a granular material. However, to date, the grounded portions of wide river jams have been explicitly excluded from the ice jam models. This exclusion means that stresses acting through the ice in the toe region and flow through the grounded portion are both neglected. This neglect comes with a price: ad hoc assumptions regarding the source of jam stability in the toe, as well as “non-intuitive” and confusing numerical strategies to prevent the ice from contacting the bed during solution of the ice jam force balance equation. In this presentation the obstacles to modeling grounded portions of ice jams are addressed. The fundamental obstacle is that the vertical stress distribution employed by current models explicitly requires that the ice float at hydrostatic equilibrium, a condition not found in grounded portions of jams. A generalized vertical stress distribution is presented that covers the possible range ice submergence. Water flow through the grounded portion is modeled assuming high Reynolds number porous flow and using flow parameters based on field and laboratory observations. The ice jam force balance is modified to include the generalized vertical stress distribution and the resistance of the bed and the drag of the flow through the grounded portion. The one-dimensional steady flow equation is modified to include flow through the jam as well as under-jam flow and open water. The force equation is then closely coupled with the flow equation and the two are alternately solved to arrive at the final stable ice jam geometry. Procedures for dealing with irregular cross sections in the grounded portions and other practical difficulties are suggested.
River ice break-up and ice jam formation
Spatial and Temporal Patterns of Break-Up and Ice Jam Flooding in the Mackenzie Delta, inferred from historical hydrometric data and remotely sensed imagery

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The Mackenzie Delta is covered in freshwater lakes that provide habitat for a myriad of species. The hydrology and ecology of these delta lakes are dominated by cryospheric processes, specifically snowmelt induced spring break-up ice jams, which typically produce the largest hydrologic event of the year. In light of limited current understanding of break-up patterns and processes in the delta, the objectives of this research are to explore the consistency of intra-delta water level variations occurring during extreme floods in the delta, determine the recurrence of these floods, and consider the influence of flood severity on the timing and duration of break-up. To complete the analysis, a break-up chronology for the period from 1972 to 2006 is assembled using a series of event-based hydrometric variables for 14 Water Survey of Canada (WSC) hydrometric stations in the delta. Historical observations and remote sensing products are used to inform and constrain the variables. Analysis of backwater level and discharge at the Mackenzie River at Arctic Red River (MARR) station and the associated spatial distribution of peak water levels in the delta are used to explain the physical rational of spatial patterns, while a return period assessment is conducted to determine the recurrence of peak water levels. Temporal patterns of extreme events are investigated through a comparison of timing and duration to averages determined for the delta. The results of the analysis highlight years of extensive delta flooding, and within the subset of larger flood years, two types of events are identified: ice-dominated events, with high peak water levels at MARR associated with high levels in the mid-delta, and discharge-driven events, with extensive high water levels in the mid and outer delta despite lower upstream peak water levels. Temporally, the break-up initiation during ice (discharge) driven events occurs earlier (later) than the delta average; no trend is detected with respect to break-up duration. These findings represent the first stage of continued investigation into the hydroclimatic controls on extreme hydrological events in the Mackenzie Delta.
1. Introduction
The Mackenzie Delta, located in Western Arctic Canada is the largest cold-regions delta in North America. Over 49,000 lakes cover the deltaic plain (Emmerton, 2007), creating a unique aquatic ecosystem that is a haven for fish, waterfowl, and aquatic mammals. The hydrologic and ecological processes in the Mackenzie Delta are intimately tied to cryospheric processes. A nival regime hydrograph is observed for the flow of the Mackenzie River entering the delta at Arctic Red River (Woo and Thorne, 2003), therefore runoff is dominated by high spring flows from snowmelt in the Mackenzie Basin followed by low summer and winter flows (Church, 1974). Given the northern flow direction of the Mackenzie, the spring freshet progresses downstream with the seasonal advance of warm weather, and the flood wave often encounters an intact and resistant ice cover resulting in the formation of ice jams and flooding (Rouse et al., 1997). Thus, of particular importance for this break-up dominated reach (de Rham, 2008a) are spring break-up ice jams occurring concurrently with the spring freshet, which typically produce the largest hydrologic event of the year and the flooding of delta lakes (Marsh and Hey, 1989).

Records and observations of break-up and ice jam flooding in the Mackenzie Delta have highlighted certain aspects of the spatial and temporal variation of ice jams and flooding. Break-up patterns and processes in the delta are complex due to the influence of both the Mackenzie and Peel Rivers and numerous interconnecting channel-lake systems (Mackay, 1963a). Literature suggests that the central and eastern sections of the delta are controlled by the Mackenzie River, while the southwestern region of the delta is controlled by the Peel and western mountains rivers (Bigras, 1988), however diversions of backwater (Henoch, 1960, Kriwoken, 1983) and Mackenzie ice (Brown, 1957, Mackay, 1963b) into the Aklavik and Peel Channels have been reported. These are often associated with ice jams, which have been observed to consistently form between Point Separation and Horseshoe Bend (Brown, 1957, Bigras, 1988). Lower levee heights in the northern delta have been observed to cause extensive flooding in this region even in years of overmature break-up (Terroux et al., 1981, Bigras, 1988), while it is postulated that dynamic break-up and ice jamming are required to cause flooding in the southern delta (Bigras, 1988, 1990). This is confirmed by studies into the flooding hydrology of delta lakes, as the number of high elevation lakes, having sill elevations greater than the one-year return period of the spring peak water level (Marsh and Hey, 1989), decreases in a northward progression through the delta (Marsh and Hey, 1991). Related to break-up timing, Henoch (1960) reported that break-up on the Peel River proceeded Mackenzie River break-up by 10 days, while Mackay (1963b) calculated an average difference of 8 days for forty-seven years of record between 1895 and 1960. Marsh and Hey (1989) found that the timing of peak water level in the delta is remarkably consistent, occurring on average on June 3 at Inuvik. This same date was reported for the annual peak flow of the Mackenzie River at Arctic Red River (Woo and Thorne, 2003).

Despite these observations, no comprehensive assessment of the spatial and temporal variation of peak water level during the spring break-up period has been undertaken. Accordingly, the objectives of this research are: (1) to explore the consistency of intra-delta water level variations occurring during ice-affected extreme floods in the delta, (2) determine the recurrence of ice-induced extreme water levels in the delta, and (3) consider the influence of flood severity on the timing and duration of break-up in the delta.
2. Study Area

The Mackenzie Delta is located where the Mackenzie River empties into the Beaufort Sea at 69°N. It covers an area of 12,000 km², with elevation variations of only 3 to 4 metres over the 200 km length, extending from Point Separation in the south, just north of the confluence of the Mackenzie and Arctic Red Rivers, to Shallow Bay in the northwest and Kittigazuit Bay in the northeast (Figure 1). Channels, lakes and wetlands cover the floodplain, underlain by permafrost (MacKay, 1963a). The average annual (1971-2000) temperature recorded for Inuvik is –8.8°C, with precipitation averaging ~250mm annually (www.climate.weatheroffice.ec.gc.ca, 2008). The Mackenzie River provides a large majority of the discharge passing through the delta, contributing $2.85 \times 10^{11}$ m³/year on average annually, while Peel River discharge represents $2.14 \times 10^{10}$ m³/year on average (www.wsc.ec.gc.ca, 2008).

![Figure 1. The Mackenzie Delta, with locations of Water Survey of Canada hydrometric stations.](image)

3. Data and Methodology

The data used to assemble the 1972-2006 break-up chronology in the Mackenzie Delta were obtained from a variety of sources. The peak water level during break-up ($H_m$) and its timing ($t_{H_m}$), in addition to the timing of the initiation of break-up ($t_{H_b}$) and the ‘Last B’ date for 14 hydrometric stations in the delta, form the basis of the chronology (Table 1). These were determined from Water Survey of Canada (WSC) archives, which for each station optimally include: pen recorder charts during the break-up period, yearly station analyses, annual water level tables, discharge measurement tables, and hydrometric survey notes, in addition to the station description, gauge and benchmark history, and stage-discharge curves. Historical observations, air photography spanning the period from 1973 to 1983 available from the National Water Research Institute, and a series of Landsat, AVHRR and MODIS images dating from 1973...
to 2006 were used to confirm and constrain the timing of initial ice movement (pertaining to \( tH_b \)) and ice free status (pertaining to the ‘Last B’ date) for the various locations in the delta.

**Table 1.** Percentage of \( H_m \), \( tH_m \), \( tH_b \) and ‘Last B’ records extracted in the period from 1972 to 2006 for the 14 Water Survey of Canada hydrometric stations in the Mackenzie Delta.

<table>
<thead>
<tr>
<th>ID</th>
<th>Station</th>
<th>( %H_m )</th>
<th>( %tH_m )</th>
<th>( %tH_b )</th>
<th>‘Last B’</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>Mackenzie River at Arctic Red River*</td>
<td>80</td>
<td>77</td>
<td>91</td>
<td>94</td>
</tr>
<tr>
<td>R2</td>
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<td>74</td>
<td>86</td>
<td>97</td>
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<tr>
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<td>100</td>
<td>89</td>
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<tr>
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<td>23</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>Peel River at Frog Creek</td>
<td>17</td>
<td>14</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>M1</td>
<td>East Channel at Inuvik*</td>
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<td>94</td>
<td>97</td>
<td>94</td>
</tr>
<tr>
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<td>46</td>
<td>49</td>
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<tr>
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<tr>
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<tr>
<td>N6</td>
<td>Napoiak Channel above Shallow Bay</td>
<td>26</td>
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<td></td>
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</tr>
</tbody>
</table>

* stations with published discharge values

The \( H_m \) magnitude and timing correspond to the maximum instantaneous stage during break-up, available from the water level charts and published when it is the maximum stage for the year. In years when the gauge is damaged or destroyed during break-up, reliable high water marks published by WSC or mean daily water levels, when available, can be used as the \( H_m \). Comparisons of peak water levels across hydrometric stations were undertaken using normalized water levels, in order to account for variations despite site-specific benchmarks at each station. For a given year (i) the normalized peak water level is calculated as follows:

\[
H_{m \text{, norm} \text{, } i} = \frac{|H_{m \text{, } i} - H_{m \text{, avg}}|}{\sigma}
\]  

Thus, yearly spring peak water levels are compared based on their number of standard deviations (\( \sigma \)) from the mean for the station. Herein, all references of peak water level refer to normalized values. When aggregated for the delta, these peak water levels were averaged yearly, with equal weighting for all stations with records in a given year. Recurrence analysis was also performed for the peak water levels at each station using the Weibull Method, where the probability of exceedance for a given year (i) is calculated using:

\[
P_i = \frac{m_i}{(n + 1)}
\]  

Where \( P \) is the probability of exceedance, \( n \) is the number years with recorded peak water level, and \( m \) is the rank of the peak water level. The inverse of this value represents the return period of the particular peak water level.

The probable timing of \( H_b \) can be determined, following guidelines outlined by Beltaos (1990), by considering both the latest time in which a continuous ice cover can be assumed and the
earliest time at which broken ice effects are apparent on the stage hydrograph. As shown in Figure 2, this is generally fixed as the first significant spike in water level after the stage begins to rise from its winter low, since the steep rise in water level and subsequent peaks are indicative of breaking or broken ice effects. When possible, observations available from site visits, break-up monitoring, air photographs, and satellite images were used to improve interpretation of ice conditions. This was particularly useful for years and stations in which the initiation of break-up was not recorded or not striking, perhaps due to a more thermally driven event or flow diversion of backwater into adjacent channels.

**Figure 2.** Schematic of water level recording chart during the break-up period (de Rham, 2008b adapted from Beltaos, 1990).

The ‘Last B’ date refers to the end of ice-affected flow conditions at discharge stations. The ‘B’ designation is used by WSC technicians to identify the period of the year when water levels are influenced by ice, and in which stage-discharge relationships cannot be used. While this measure is not reproducible using water level charts, as it is based on broader (and potentially more subjective) understanding of any particular station, the ‘Last B’ date can provide a measure of

**Figure 3.** Comparison of ‘Last B’ dates at East Channel at Inuvik hydrometric station (solid triangles) and estimates of the first ice-free day in the delta from air photographs and satellite images (open circles).

The ‘Last B’ date refers to the end of ice-affected flow conditions at discharge stations. The ‘B’ designation is used by WSC technicians to identify the period of the year when water levels are influenced by ice, and in which stage-discharge relationships cannot be used. While this measure is not reproducible using water level charts, as it is based on broader (and potentially more subjective) understanding of any particular station, the ‘Last B’ date can provide a measure of
the end of the spring break-up period for that particular river reach. Of stations in the mid- and outer delta, discharge is published only at the East Channel at Inuvik station. To extend this measure to the outer delta, the suite of air photographs and satellite images were used to estimate the first ice-free day in the delta. Given a fairly strong relationship between the ‘Last B’ date at Inuvik and this visual estimate (Figure 3), the former was used to represent the end of the break-up period in the delta.

For additional analysis, the availability of published discharge values and stage-discharge curves at the Mackenzie River at Arctic Red River (MARR) station allowed the extraction of maximum break-up discharge and the calculation of the backwater level caused by ice for each year. Backwater level is the term used to describe the difference between ice-affected stage and the stage for an equivalent discharge in open-water conditions.

4. Results and Discussion

Peak break-up water levels
The averages of normalized spring peak water levels for the Mackenzie Delta for the period from 1972 to 2006, shown in Figure 4, highlight the years with the highest peak water levels. In this analysis, 1982, 1992, and 2006 stand out as years with the greatest flood severity, which is confirmed by recorded observations (Kriwoken, 1983, Marsh et al., 1993, George Lennie, pers. comm.). Equally, 1983, 1985, 1986, 1989, 1991, 1997 and 2005 also have higher than average water levels for some regions of the delta. The results also reveal a clear decadal trend of alternating severity, with higher (lower) water levels recorded for the 1980s (1970s, 1990s).

![Figure 4](image)

Figure 4. Average normalized peak water level in the Mackenzie Delta for the period from 1972 to 2006 (with bars depicting one standard deviation).

Table 2 shows the peak water levels for extreme and large years at stations with a record length exceeding 15 years. For the extreme years a majority of the stations have levels near or exceeding 1, although differences are apparent. In 1992 levels are not remarkable on the Mackenzie or Arctic Red Rivers, while levels on the Peel River and at all stations in the delta are considerably higher than average. In contrast, 2006-levels are among the highest recorded for the Mackenzie and Arctic Red Rivers and on the Peel Channel at Aklavik, with lower levels...
experienced in the outer delta, although flooding may still be widespread in this region for these events given the low levee heights. For large years, one or several stations in a given region or channel have higher than normal water levels, however the trend of higher water level does not apply to all stations.

Table 2. Normalized peak water levels for extreme and large years at stations with record lengths exceeding 15 years. Values greater than 1 (2) are highlighted in medium (dark) grey.

<table>
<thead>
<tr>
<th>Year</th>
<th>Mackenzie River at Arctic Red River (R1)</th>
<th>Arctic Red River near the Mouth (R2)</th>
<th>Peel River above Fort McPherson (R3)</th>
<th>East Channel at Inuvik (M1)</th>
<th>Middle Channel below Raymond Channel (M2)</th>
<th>Peel Channel above Aklavik (M3)</th>
<th>East Channel above Kittigazuit Bay (N1)</th>
<th>Middle Channel at Tununuk Point (N2)</th>
<th>Reindeer Channel at Ellice Island (N5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1980</td>
<td>-0.93</td>
<td>-0.37</td>
<td>2.35</td>
<td>-0.10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1982*</td>
<td>1.87</td>
<td>1.36</td>
<td>2.11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1983</td>
<td>0.43</td>
<td>0.96</td>
<td>1.00</td>
<td>-1.00</td>
<td>-1.20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1985</td>
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<td>0.85</td>
<td>1.27</td>
<td>0.50</td>
<td>0.62</td>
<td>1.12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1986</td>
<td>0.79</td>
<td>-1.05</td>
<td>0.53</td>
<td>1.27</td>
<td>0.89</td>
<td>1.35</td>
<td>1.19</td>
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</tr>
<tr>
<td>1989</td>
<td>2.01</td>
<td>0.96</td>
<td>0.58</td>
<td>0.84</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1991</td>
<td>0.08</td>
<td>2.34</td>
<td>1.51</td>
<td>-0.12</td>
<td>-0.06</td>
<td>0.35</td>
<td>0.78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1992*</td>
<td>0.71</td>
<td>0.25</td>
<td>1.39</td>
<td>1.72</td>
<td>1.30</td>
<td>2.13</td>
<td>1.40</td>
<td>1.82</td>
<td>2.60</td>
</tr>
<tr>
<td>1997</td>
<td>1.57</td>
<td>-0.15</td>
<td>0.58</td>
<td>1.18</td>
<td>0.11</td>
<td>0.47</td>
<td>0.40</td>
<td>0.32</td>
<td></td>
</tr>
<tr>
<td>2005</td>
<td>1.34</td>
<td>-0.15</td>
<td>-1.00</td>
<td>0.68</td>
<td>0.28</td>
<td>0.19</td>
<td>1.08</td>
<td>0.64</td>
<td>0.85</td>
</tr>
<tr>
<td>2006*</td>
<td>2.70</td>
<td>2.18</td>
<td>0.97</td>
<td>1.74</td>
<td>1.21</td>
<td>2.15</td>
<td>0.89</td>
<td>0.31</td>
<td></td>
</tr>
</tbody>
</table>

*denotes years with the greatest overall average normalized peak water level

A comparison of the delta average peak water level to the peak water level, backwater effect and discharge at MARR (Figure 5) provides some insight into the processes likely driving the observed high water patterns. MARR is the entry point station to the delta, delivering the dominant hydrologic signal from the Mackenzie Basin; here it is also shown to fairly accurately represent delta average peak water levels, particularly for years with higher (lower) than average levels. With the availability of backwater and maximum break-up discharge data at this site, allowing the influence of ice and discharge to be isolated, a set of patterns emerges complementary to those above noted. In years when the peak water level at MARR exceeds the delta average (1989, 1997, 2005, and 2006), the backwater effect shows a comparable deviation above the average, indicating that ice effects (such as ice jams) are a dominant driver of increased water levels. Conversely in years when the delta average is comparable or larger than the value at MARR (1986, 1992), the backwater effect is generally less pronounced while discharge levels are well above average, indicative of a more discharge-driven event. Not included in this latter group is 1991, which exhibited no extreme backwater or discharge levels, however this high water event was confined to the Peel and Arctic Red Rivers, and did not translate into any noteworthy intra-delta flooding (Table 2). The above patterns are particularly striking as they suggest that the spatial expression of peak water levels in the delta are ultimately controlled by the dominant hydroclimatic driver (discharge/ice) during the spring break-up period.
Figure 5. Comparison of average normalized peak water level for the delta with normalized peak water level, backwater level and discharge at the Mackenzie River at Arctic Red River hydrometric station. The symbol D (I) identifies discharge (ice) dominated years.

Recurrence analysis
The recurrence of ice-induced peak water levels, in the form of a return period assessment for various locations in the delta and the delta average, is shown in Figure 6. Following the approach of de Rham (2008a) no extrapolation was undertaken beyond the existing historical record, as peak break-up water levels are influenced by different ice effects in any given year, in addition to the constraints imposed by the height and morphology of channel banks, thus preventing accurate estimation of the likelihood or the occurrence of more extreme peak water levels. This limits the return period assessment to the temporal coverage available for each station. The results of the analysis indicate that water levels are comparable for low frequency events, while considerable divergence occurs for return periods above 10 years (probability of exceedance of 10%). Events with return periods ranging from 10 to 20 years have levels between 1 and 2, while the highest levels, between 2 and 3 have return periods of approximately 30 years. Of particular note, the most recent event, in 2006, resulted in the highest recorded water level for the delta, occurring at the MARR station with a return period of 29 years.

Figure 6. Return period plot for delta stations with record lengths exceeding 15 years, including the delta average.
Temporal trends of extreme events

The average timing of break-up initiation \((t_{Hb})\), and peak water level occurrence \((t_{Hm})\) for delta stations are shown in Figure 7, with the average Julian dates also provided for these and the ‘Last B’ dates at discharge stations. Concurrent with historical records, the initiation of break-up begins earliest on the Peel River, followed by the Mackenzie River at Arctic Red River, although the average delay of 3 days is shorter than was reported by Mackay (1963b) and Henoch (1960). This could be the result of different definitions for break-up, as outlined by Mackay (1963a), or due to changes in the timing of break-up over the last century. The southwestern portion of the delta, represented by the Peel Channel at Aklavik station, precedes the central and eastern delta by one day on average. Stations in the outer delta begin breaking-up 5 to 7 days later, although the measure of break-up initiation may be less physically meaningful for these stations given the low slope and extensive channel connections in the outer delta, such that the first ice movements often do not correspond to clear water level rises on the water level charts. Peak water levels also occur earliest on the Peel River on average, but rise in Middle Channel earlier than both the western and eastern delta, as was observed by Marsh *et al.* (1993).

![Figure 7](image)

**Figure 7.** a) Map of \(H_b\) timing for hydrometric stations in the delta, b) Map of \(H_m\) timing for hydrometric stations in the delta, and c) Table with \(t_{Hb}\), \(t_{Hm}\) and ‘Last B’ timing.

Given that dynamic break-up conditions result in the most persistent ice jams, capable of producing large-scale flooding (Prowse and Beltaos, 2002), and these are produced when a large spring flood wave propagates downstream before any significant decay of the ice cover has occurred (Gray and Prowse, 1993), a natural hypothesis would be of earlier break-up for the most extreme events in the delta. However, no such trend emerges when the timing of break-up in extreme years is compared to the average. Break-up initiation occurs 2.5 days later than average at MARR in 1982 and 1992, and 7.5 days earlier in 2006; similarly on the Peel River initiation occurs 6 and 8 days later in 1982 and 1992 respectively, and 2 days earlier in 2006. A trend does emerge, however when the time series of extreme events is separated into the above delineated categories of ice and discharge driven events. In the ice driven years, 1989, 1997, 2005 and 2006 both break-up initiation and peak water level occurs earlier or close to the
average on both the Mackenzie and Peel Rivers (e.g. 0.5, -1.5, -7.5, and -7.5 days from the average respectively for tHb at MARR), while the reverse is true for the discharge driven years, 1986 and 1992 (e.g. 7.5 and 2.5 days from the average respectively for tHb at MARR).

The average duration of break-up in the delta, from the earliest initiation of break-up (most commonly on the Peel River, but also sometimes on the Mackenzie River) to the ‘Last B’ date at Inuvik is 19.4 days, with a maximum duration of 42 days occurring in 1994, and a minimum of 9 days in 1974. Focusing on extreme events, two alternative hypotheses regarding the comparable duration can be formulated. During dynamic events, the driving force of discharge associated with the steep rising limb of the spring hydrograph fractures and moves the ice cover forming ice jams (Gray and Prowse, 1993). Waves generated by the release of these ice jams have the energy to dislodge and set in motion considerable lengths of ice cover (Beltoos, 2007), thus continuing the cycle downstream. Given that the evolution of break-up is sustained in this way, a shorter duration might be expected for these dynamic events, compared to thermal events in which the ice cover essentially disintegrates in place. Alternatively, if dynamic events produce persistent and long lasting ice jams due to a strong competent ice cover then a longer duration would be expected. A standard linear regression exploring the relationship between break-up duration and delta average peak water level did not reveal a significant trend ($r^2=0.04$). The consideration of ice and discharge driven events was equally unrevealing; while both discharge driven events had shorter than average durations, of the ice driven events 2006 and 1989 had longer than average durations, but 2005 and 1997 had shorter durations (Figure 8). The absence of a clear trend suggests that the timing of break-up initiation and the progression of break-up through the delta, influenced by numerous other hydroclimatic factors, also play an important role in controlling the duration of break-up in any given year.

![Figure 8. Comparison of break-up duration and delta average normalized peak water level for 1972-2006. Duration refers to the period from the earliest delta tHb to Inuvik ‘Last B’ date.](image)

5. Conclusions
The above analysis is the first comprehensive work chronicling spatial and temporal patterns of spring break-up in the Mackenzie Delta. The results highlight years with greater flood severity, with 1982, 1992 and 2006 standing out as years with extreme peak water levels. For these years, a consistent pattern of higher than normal water levels at a majority of stations is observed, suggesting wide-scale flooding. These events have water levels with return periods greater than
10 years (probability of exceedance less than 10%), although the range of return periods at different stations is quite large for events with peak water levels above 1.

Varying spatial patterns of water level in the delta for large flood years expose the different influences of ice or discharge dominated events on intra-delta flooding, as the examples in 1992 and 2006 reveal. Higher levels on the Mackenzie River associated with a large amount of backwater (2006), result in higher water levels in the mid-delta, while levels are not especially large in the outer delta (although flooding of low lying land may still take place). This confirms the greater role of ice in influencing southern delta flooding, as other investigations have suggested (e.g. Bigras, 1988). Alternatively, when water levels in the delta are considerably above the average and Mackenzie levels, associated with larger levels of discharge (1992), both the mid- and northern delta are influenced. This corroborates with observations by Terroux et al. (1981), ultimately showing that flooding in the northern delta is less dependent on ice effects.

A comparable distinction can be made between ice and discharge driven events when the timing of break-up initiation in the delta is considered. Earlier break-up initiation occurs in years when ice is the dominant driver of break-up peak water levels (e.g. 2006). This timing would suggest that less ice decay from solar radiation and warming took place prior to break-up, resulting in a more competent ice cover and instigating widespread ice jamming. This agrees with Bigras’ (1988) deduction that ice cover strength in the delta is a central factor controlling the formation of ice jams, in addition to influencing the length of time a jam will remain static and the amount of backwater builds-up. However, further analysis of the hydroclimatic drivers of extreme floods in the delta is required to substantiate this hypothesis. For discharge driven events (e.g. 1992) break-up initiation occurs later than average, potentially allowing a greater amount of snowmelt runoff from the basin to reach the delta during the break-up period and augment water levels despite a reduced prevalence of ice effects. Extreme events, whether ice or discharge driven, do not exhibit any trend in duration, likely due to a complex array of factors influencing the progression of break-up in the delta.

The spatial and temporal patterns above reveal an apparent duality in the drivers of break-up peak water levels during extreme events. Continuing investigation into the hydroclimatic controls of extreme break-up floods will likely further clarify the influence and expression of these drivers.

Acknowledgments
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References


Short-Term Forecast Method On Maximum Ice Jam Flood in Songhuajiang River

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ABSTRACT

On the basic of analyzing the air temperature, ice change process, ice flood evolve position at upstream and downstream, the flood propagating speed in ice cut period of the Songhuajiang River, and according to water balance principle, using the method on the water level with discharge at upstream and downstream to forecasting the maximum level of ice jam for the Songhuajiang River. Meanwhile, the authors also consider to the ice flood that it is affected in the air temperature and different icecap strength in this paper. There is a certainty basic of water balance in this way, there is upper precision to apply to forecast.

Key words: ice jam; ice flood level; ice cover strength; ice thickness; water balance
1. INTRODUCTION
The Songhuajiang River is an only river flowing from south to north in China, and high latitude, cold weather in winter, long time freezing water and large quantity of water storage are its characteristics. During thawing time, ice jam exists all the time because the water of upstream river is heated more early than that of downstream river, and the ice of the river melts from the headstream. Ice jam usually exists in the downstream section of the Songhuajiang River, and has 20% occurrence probability. Ice flood comes tempestuously and strongly which induces huge economic loss. The highest water level of Yilan hydrologic station of Songhuajiang River reached 97.10 m in 1960, which was 3.10 m higher than the maximum ice jam flood and just 0.61 m lower than the highest summer flood water level in history. Sixteen ice jams occurred in the downstream section of Songhuajiang River lasting 360 km from Yilan to Fujin during April 5 to 19 in 1981. Its large quantity and huge dimensions are infrequent in history. The heightness of the jams reached 6 to 8 m, and the short duration was 1 to 2 days while the long was 7 to 8 days. The highest water level of Jiamusi hydrologic station reached 78.89 m close to the highest summer flood water level in history. Thousands of houses and millions of farmlands were submerged and destroyed, and a lot of water conservancy projects and ships were devastated during every ice jam flood. It became water calamity to the people living along the river. Therefore more researches should be made on ice jam flood and the forecast of its highest water level to prevent ice and flood. It is an important task of hydrological workers. According to the hydrology, climate and channel characteristics of Songhuajiang River, several kinds of forecast methods were improved and gave good results.

2. BASIC THEORY OF FORECAST METHOD
It is well known that in normal condition flood process is affected by the depth of channel, additional gradient and interval water quantity. Flood waves expand, twist and deform. The water balance of the upstream and downstream cross section of river can be expressed as follows:

\[ Q_{\text{lower},t} = Q_{\text{up},t} - \Delta Q + q \]  \[1\]

Where \( Q_{\text{up}} \) and \( Q_{\text{lower}} \) are the fluxes of upstream and downstream cross section respectively, \( \Delta Q \) is the flux change aroused by channel condition and additional gradient during flood transmission, \( q \) is interval water increment and \( t \) is flood travel time. Based on formula [1] the forecast formulas \(^{[1]}\) of water level and combined flux considering interval inflow, channel speciality and additional gradient are as follows:

\[ H_{\text{lower},t} = f(H_{\text{up},t-\lambda}) \]  \[2\]

\[ H_{\text{lower},t} = f(\sum Q_{\text{up},t-\lambda}) \]  \[3\]

Formula [2] is single channel, and formula[3] is a channel with branches. \( H_{\text{up}} \) and \( H_{\text{lower}} \) are the water levels at upstream and downstream cross section respectively, \( \sum Q \) is combined flux, \( t \) is time and \( \lambda \) is flood travel time. As ice cover existing, multiple factors including water flow, ice
cover etc. make flood waves more complicated. In the situation of inverted river melting, a part of water and ice form superposition mechanism. As ice jam existing, discharge capacity decreases and water level ascends because of the ice block and accumulation, and the quantity of ice almost determines the highest water level. Formula \[2\] and \[3\] can be improved as follows:

\[
H_{\text{lower,mt}} = H_{\text{lower}} + \Delta H_i = f(H_{\text{up,1-3}}, \Phi, h_i) \tag{4}
\]

or

\[
H_{\text{lower,mt}} = H_{\text{lower}} + \Delta H_i = f(\sum Q_{\text{up,1-3}}, \Phi, h_i) \tag{5}
\]

Where \(H_{\text{lower,mt}}\) is the highest water level of downstream ice jam, \(H_{\text{lower}}\) is the normal highest water level of downstream section with no ice jam, \(\Delta H_i\) is the additional water level raised by ice jam block, \(\Phi, h_i\) is the critical strength of ice cover, \(\Phi_i\) is strength and \(h_i\) is critical ice thickness. It can be concluded that besides of the flux (water level) of upstream cross section and interval inflow, the highest water level raised by ice jam also has relation with the strength of ice cover (ice sheet quantity and ice thickness). The forecast of the highest water level refers to the calculation of ice quantity and the strength of ice cover.

3. STATISTICS METHOD

3.1 Water level and flux statistic of upstream and downstream station

The forecast of the section down from Yilan was based on the discharge of Harbin station considering the interval inflows of Mudanjiang River and Tangwang River. The water level and flux relations between upstream and downstream corresponding section in main ice jam years were showed in Table 1.

| Year | Harbin station Water level | Flux (m³/s) | FCS 5 \(d^3\) | FMS 2 \(d^3\) | CF3 \(m^3/s\) | Jiamusi station Water level | Flux (m³/s) | MD | TTd | MD
|------|---------------------------|-------------|----------------|----------------|---------------|---------------------------|-------------|-----|-----|-----
| 1955 | 115.97                    | 4.15        | 3070           | 2157           | 4.11          | 2200          | 4.11        | 155 | 4.14 | 4.15 |
| 1957 | 116.50                    | 4.19        | 3070           | 2157           | 4.11          | 2200          | 4.11        | 155 | 4.14 | 4.15 |
| 1960 | 116.07                    | 4.13        | 3070           | 2157           | 4.11          | 2200          | 4.11        | 155 | 4.14 | 4.15 |
| 1961 | 115.66                    | 4.13        | 3070           | 2157           | 4.11          | 2200          | 4.11        | 155 | 4.14 | 4.15 |
| 1964 | 116.10                    | 4.13        | 3070           | 2157           | 4.11          | 2200          | 4.11        | 155 | 4.14 | 4.15 |
| 1973 | 115.33                    | 4.16        | 3070           | 2157           | 4.11          | 2200          | 4.11        | 155 | 4.14 | 4.15 |

a) Flux of Chenming station 5 days later; b) Flux of Mudanjiang station 2 days later; c) Combined flux; d) Travel time; e) Month & Day

3.2 Ice amount calculation

The decrease of ice thickness \(\Delta h\) is calculated based on the exchange principle of atmosphere, water and heat, and then the critical ice thickness \(h\) and \(\Phi h\) can be deduced. For the complicated structure of ice cover, heat conduction multiple-layers structure model and the decrease rate of ice thickness are utilized to calculate the critical ice thickness.
In the process of ice thickness decreasing as the ice cover is heated, the decrease differential equation of ice cover thickness changing with time is as follows\(^{[2]}\):

\[
h_i \frac{dh_i}{dt} = k(T_i - T_w)/\rho_i \cdot L
\]

Where \(\rho_i\) represents ice density (kg/m\(^3\)), \(L\) represents water latent heat (J/kg), \(T_i\) represents atmosphere temperature, \(T_w\) represents water temperature, \(k\) represents thermal conductivity (w/(m\(^2\)·K)).

The calculation formula of ice cover thickness can be expressed as the integral of formula \([6]\):

\[
\int h_i dh_i = \int \frac{k(T_i - T_w)}{\rho_i} \cdot dt
\]

Suppose \(k(T_i - T_w)/\rho_i \cdot L\) is a constant in time interval, the ice cover thickness of the end of time interval \(i\) can be calculated as follows:

\[
h_i = \sqrt{\frac{2k(T_i - T_w)}{\rho_i} \cdot t_i}
\]

On the condition of no wind, no snow on ice surface and no riverbed thermal radiation, let \(T_w = 0\) and replace \(T_i\) with air temperature \(\theta\), then the decrease of ice cover thickness can be got from formula \([8]\):

\[
h_i = K \sqrt{\sum_{i=1}^{n} \theta}
\]

Where \(K\) is a comprehensive index combined by thermal conductivity, ice density and latent heat, \(n\) represents days of time interval. The experiential relation between cumulated positive temperature \(\sum Q\) and ice thickness decrease \(\Delta h\) in Jiamusi station calculated by formula \([8]\) is showed in Fig.1. According to \(\sum Q\) in Fig.1, the critical ice thickness of river ice melting day can be calculated \(h_{\text{critical}} = H_{\text{max}} - \Delta h\), where \(h_{\text{critical}}\) represents the critical ice thickness.

**Figure 1.** Relation between cumulated positive temperature \(\sum Q\) and ice thickness decrease \(\Delta h\) in Jiamusi station
4. ESTABLISHMENT OF FORECAST SCHEME

4.1 Corresponding water level (flux) method

According to the corresponding flood peaks of Harbin and Jiamusi station, the corresponding water level (flux) relation and travel time were calculated by formula [3], and showed in Fig.2. As considering interval inflow, the relation established by combined flux based on formula [4] was showed in Fig.3.

![Image of Fig.2: The highest water level relation of ice jam flood between Harbin and Jiamusi station]

According to the figures, it can be concluded that points on the water level at upstream and downstream (flux) relation graph were dispersive, so prediction error was big, which mainly affected by river ice cover (ice quantity and ice cover strength). Considering these influence factors, the method was improved.

4.2 Correlation method on river ice melting thickness and maximum water level of flood

The relation between the critical ice thickness and the maximum water level based on temperature is as follows:

\[ H_{\text{max}} = f (h_{\text{opening river}}) \]  \[10\]

Where \( H_{\text{max}} \) represents flood highest water level (m), \( h_{\text{opening river}} \) is the ice cover thickness of river ice melting day deduced from Fig.1, its relation was showed in Fig.4.
The relation figure had high precision except for some specific points. It can be concluded that ice cover thickness absolutely determined the additional water level ascent of ice jam.

4.3 Correlation method of ice cover thickness and water level ascent

According to formula [4] and [5], the maximum ice jam flood includes two parts of the normal flood highest water level and the additional water level ascent affected by ice cover strength, namely $H_m = H_{\text{normal}} + \Delta H$, $H_{\text{normal}}$ can be deduced by the relation of upstream and downstream corresponding water level (flux), and $\Delta H$ can be calculated by the equation $\Delta H = \text{f}(\Phi,h)$, which were showed in Fig.5 and Fig.6. The experiential formulas used to calculate ice cover strength and critical ice thickness are as follows.

\[
\Phi_t = 5.5(1 - \frac{L}{\sqrt{T}})^2 \quad [11]
\]

\[
h_r = h_n(1 - \left(\frac{T}{\sqrt{T}}\right)^2) \quad [12]
\]

Where $\Phi_t$ represents strength, $h_r$ represents the critical thickness, $t$ represents river ice melting day difference between Harbin and Jiamusi, $T$ represents the time difference between the earliest river ice melting day of Harbin and the last day of Jiamusi, $h_n$ represents the ice thickness of Jiamusi station when the Harbin section of the river is thawing.

The ice cover strength was calculated by the multiplication of $\Phi_t$ and $h_r$ in the formulas upwards, and the results of Jiamusi station were showed in Table 2.
Table 2. Calculation table of ice strength in Jiamusi station

<table>
<thead>
<tr>
<th>Year</th>
<th>RIMD&lt;sup&gt;a)&lt;/sup&gt;</th>
<th>HJD&lt;sup&gt;b)&lt;/sup&gt;</th>
<th>JTWHM&lt;sup&gt;c)&lt;/sup&gt;</th>
<th>h&lt;sub&gt;i&lt;/sub&gt;</th>
<th>φ&lt;sub&gt;i&lt;/sub&gt;</th>
<th>φ&lt;sub&gt;h&lt;/sub&gt;</th>
<th>H&lt;sub&gt;harbin&lt;/sub&gt;</th>
<th>H&lt;sub&gt;jiamusi&lt;/sub&gt;</th>
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<td>11</td>
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<td>0.57</td>
<td>2.68</td>
<td>1.54</td>
<td>115.92</td>
<td>78.14</td>
</tr>
<tr>
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<td>4.10</td>
<td>14.18</td>
<td>8</td>
<td>0.7</td>
<td>0.52</td>
<td>2.30</td>
<td>1.20</td>
<td>115.97</td>
<td>76.9</td>
</tr>
<tr>
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<td>2.30</td>
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<td>1.1</td>
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<td>1.88</td>
<td>116.50</td>
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<sup>a)</sup> River ice melting day; <sup>b)</sup> River ice melting day difference between Harbin and Jiamusi; <sup>c)</sup> The ice thickness of Jiamusi station when the Harbin section of the river is thawing.

As the temperature is increasing sharply and the dynamic of upstream section is strengthening, reverse river ice melting is formed. Ice cover had a high strength when river ice melting and water level was ascended for ice jam and block to form ice jam flood. The correlation method of ice cover strength and water level ascent used to forecast the maximum ice jam flood had high prediction precision and representation<sup>[4]</sup>. The results are showed in Fig.5.
The calculation of ice jam additional water level ascent $\Delta H$ was showed in Fig.6. The envelope curve represents the normal relation with no ice jam, $\Delta H$ represents the difference between normal water level and ice jam additional water level, namely $\Delta H = H_m - H_o$, where $H_o$ represents the corresponding highest water level with no ice jam (deduced by the envelope curve).

4.4 Watershed water storage method
Affected by frozen soil hydrology, the channel ice quantity and ice flood process in Spring of cold area river has obvious relation with earliest watershed water storage$^{[5]}$. The maximum ice jam flood was calculated by rainfall-runoff relationship showed in formula [12]:

$$H_m = f(P + P_c)$$

Where $H_m$ represents the maximum ice jam flood, $P_c$ represents earliest watershed precipitation calculated monthly form July last year, $P$ represents the critical rainfall of river ice melting period in early April.

The correlation graph was established based on years of observations and showed in Fig.7. The method had good correlations, but for the large watershed area and the influence of ice cover, the graph had a low precision.
5 CONCLUSIONS
Several kinds of short-time forecast method on maximum ice jam flood were presented in the paper, which had some physical cause basic and a better prediction. For the influences of temperature, ice thickness and ice cover in ice jam flood period, the prediction on maximum ice jam flood had some errors. Anyway, more works should be done to improve it.

REFERENCES:


Innovation in River Ice Monitoring and Management in Alberta, Canada

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In Canada, north flowing rivers are susceptible to ice jam formation that can lead to rapidly rising water levels and severe flooding. When river ice processes create ice jam conditions near populated areas, the potential for serious property damage and even loss of life is significant. To reduce the potential impact of ice jam flooding it is imperative that river ice processes be monitored and, where possible, managed.

Alberta Environment monitors river ice processes throughout the province of Alberta. Based on potential impact and an established record of ice jam related flooding, two locations are closely monitored each year; the Peace River at the Town of Peace River and the Athabasca River at Fort McMurray. Both locations are located on north flowing rivers in northwestern and northeastern Alberta, respectively.

Alberta Environment has had the opportunity to partner with various organizations in exploring new and innovative techniques for the monitoring of river ice processes and, where the river is regulated such as the Peace River using flow manipulation to manage river ice processes. These techniques have been developed and evaluated with a goal to provide local municipalities with more warning and information for emergency preparedness and response.

This paper presents a description of the monitoring and operational challenges at the two locations and the new techniques that have been implemented or are being evaluated to meet these challenges.
1. Introduction
In the province of Alberta, during the onset of winter or spring, there is always a potential for flooding caused by ice jams. This potential for flooding can be of significant concern, when ice jams occur near populated, low lying areas. To better manage the concern and risk associated with this potential, the Alberta Government’s Ministry of the Environment or Alberta Environment (AENV) is responsible for monitoring river ice processes throughout the province and providing expert advice to local municipalities. The municipalities in Alberta that have the highest risk of experiencing ice jam related flooding are the Town of Peace River (TPR) along the Peace River and Fort McMurray along the Athabasca River. AENV has developed specific river ice monitoring systems for each location so that relevant information required for emergency preparedness or operational decision making is available on a real-time or near real-time basis. Each location provides a different set of challenges regarding monitoring due to different hydraulic regimes and operational requirements. New and innovative monitoring techniques have been developed for both locations to improve the understanding and response to changing river ice conditions.

2. Peace River Background
The Peace River originates within British Columbia and flows east into Alberta, to the Town of Peace River (TPR). From the TPR, the river flows north to Fort Vermilion where it turns east and continues to the Peace-Athabasca Delta. Approximately 400 km upstream of the TPR, the Peace River is regulated by the W.A.C. Bennett Dam, which impounds Williston Reservoir, and by the ‘run of river’ Peace Canyon Dam (PCN) roughly 20 km downstream of the Bennett Dam. Both dams are operated by the British Columbia Hydro and Power Authority (BC Hydro). Roughly 5 km upstream of the TPR, the Smoky River enters the Peace River and at the Town the Heart River enters the Peace River. Both the Smoky and Heart rivers are naturally flowing rivers.

Figure 1 shows a map of the Peace River basin within the province of Alberta and the river reaches in the vicinity of the Town of Peace River. At the TPR, roughly 38% of the drainage basin is regulated by the Bennett Dam. Of the unregulated portion, 43% is contributed by the Smoky River sub-basin. Because of this and its proximity to the TPR, the Smoky River has a significant effect on ice jam flood management and operational response during both freeze-up and breakup and is monitored closely.

River ice freeze-up on the Peace River at the TPR typically occurs in early January, with the historical (1973 – 2007) average freeze-up date being January 6. In the lower reaches of the Peace River freeze-up typically begins in early or mid December. Ice pans bridge over in the lower reaches and the ice front begins to advance west and southward, opposite the direction of flow. The ice cover that forms in the upper reaches is considered to be a consolidated ice cover rather than juxtaposed. As the cover advances upstream, water levels can increase by as much as 3.5 meters and there is a potential for the ice cover to consolidate further, which could increase the freeze-up water level even further. If a secondary consolidation happens at the TPR, there is a risk of groundwater seepage flooding into the lower lying residences. If the consolidation is large enough, there is a risk of overtopping of the Town’s dykes. Once freeze-up at the TPR has occurred, the ice cover continues to advance upstream usually reaching the maximum upstream extent of advance by mid-February.
In the spring the ice cover then begins to recede thermally. The typical date of breakup at the TPR is April 11. The risk of severe ice jam flooding at the TPR increases when the breakup is dominated by mechanical processes rather than thermal. Thus, freeze-up and breakup at the TPR are monitored and managed so as to mitigate the risk of ice jam formation.

In 1975, 7 years after Alberta and British Columbia signed a memorandum of understanding regarding the monitoring of river ice processes on the Peace River, the Alberta-British Columbia Joint Task Force on Peace River Ice (JTF) was formed. Currently, the focus of the JTF is the monitoring of freeze-up and breakup, and the management of river ice processes that may have an effect on the TPR. Operational procedures (controlling the releases from the Bennett Dam and PCN) agreed upon by the JTF are implemented to influence these processes with the goal of protecting the TPR from ice related flooding. This can be especially difficult since the 2 to 3 day travel time that a flow change takes to arrive at the TPR must be accounted for.

Freeze-up management operations are implemented with two major goals in mind; achieving a target freeze-up elevation of no greater than 315.0 m (+/- 0.5 m) at the Water Survey of Canada (WSC) gauge at the TPR and maintaining a nearly-constant discharge while the ice cover is forming at the TPR. The target freeze-up elevation is set to provide sufficient freeboard to protect the town’s dykes from being overtopped by a secondary consolidation of the ice cover. The constant discharge is maintained with a view to not initiate a secondary consolidation because of excessively fluctuating flow conditions. The target elevation is reached by meeting a constant, target discharge in the Peace River at the TPR.

This target discharge is agreed upon by the JTF and is achieved by implementing controlled flow releases from PCN that are equal to the target discharge, less the measured and estimated inflows between PCN and the TPR. The controlled flow releases are held relatively constant throughout freeze-up at the TPR in order to promote stability in the ice cover as it sets in. The decision to implement flow control is based on the rate of advance of the ice cover towards the TPR and a forecast of its arrival at a ‘rendezvous point’ roughly 16 km downstream of the TPR. Using this forecast, the JTF determines when the ice front is roughly 2 days from arriving at the ‘rendezvous’ point so that changes in the flow releases meets the advancing ice front downstream of the TPR. The ‘rendezvous point’ ensures that if the change in flow were to have a negative effect on the ice cover then it happens sufficiently downstream of the Town so as to not cause flooding. Once freeze-up at the TPR is complete and the ice cover upstream is deemed competent enough to withstand a consolidation, flow controls are lifted and BC Hydro operates the dams on a load-demand basis.

Mid-winter operations typically consists of monitoring the location of the ice front after it is deemed competent and general river conditions, however during certain years further operational response may be necessary. If water levels at the WSC gauge in the TPR rise and remain above 315.5 m for an extended period of time, groundwater seepage flooding in the Lower West Peace development of the TPR occurs. If this occurs, operational responses are determined and implemented according to the JTF guidelines, though this response and its sustainability may be limited by load demand or system stability within BC Hydro’s power grid.
Breakup operations begin approximately 2 to 3 weeks before the estimated dates of breakup of the Peace and Smoky rivers. The operations are implemented to ensure that water level and discharge limits do not exceed the 1:100 year level of flood protection at the TPR. This proves to be difficult when the travel time between PCN and the TPR is taken into account along with the effect of the breakup of the Smoky River. Thus, the operational forecast must predict, at least two days in advance, the breakup of the Smoky River so as to allow enough time for implemented flow releases to reach the TPR.

If breakup is progressing thermally along the Peace River, then the JTF may decide to increase flows to encourage the thermal decay of the ice cover and the recession of the ice front. However, if it appears that mechanical breakup may occur, the JTF will recommend a flow decrease. This is to prevent the worst case scenario of ice jam formation and flooding at the TPR during breakup if the Smoky River breaks up mechanically while the ice front is upstream of the TPR. If this occurs, there is a significant chance that an ice run during the mechanical breakup of the Smoky River will negatively impact the intact Peace River ice cover and cause it to breakup prematurely. The risk of a premature breakup of the Peace River ice cover is that a jam may form just downstream of the Town, potentially raising water levels above the dyke elevation. Once breakup at the TPR is complete and the risk of flooding at the Town has passed, the JTF removes any flow controls that were implemented and BC Hydro resumes operations on a load-demand basis. AENV continues to monitor breakup downstream of the Town until breakup of the majority of river is complete.

3. Athabasca River Background
The Athabasca River is a north flowing river that is located entirely within the province of Alberta. The Athabasca River flows in a north-easterly direction out of the Rocky Mountains, it then turns south down toward the Town of Athabasca. After passing by the town it then turns north again. The Athabasca River continues north for approximately 300 km before flowing east towards Fort McMurray. At Fort McMurray, the river again turns north for approximately 300 km before reaching the Peace Athabasca Delta. Figure 2 shows a map of the Athabasca River basin as well as a map of the river reaches in the vicinity of Fort McMurray. The slope of the Athabasca River decreases by an order of magnitude and the river significantly widens just downstream of the MacEwan Bridge at Fort McMurray.

Since the Athabasca River is north flowing, the upstream portions of the river begin to melt and breakup prior to the downstream reaches. This leads to ice runs that are impeded by a solid ice cover that has not deteriorated enough to be moved. This, combined with the change in hydraulic conditions that occurs at Fort McMurray, makes it an ideal location for ice jam formation. There are also numerous islands within this reach of the river which also restrict the movement of an ice run and promote jamming. Finally, the confluence of the Clearwater River is also located within this 5 km reach. If an ice run on the Athabasca River jams within this reach or just downstream, it has the potential to block off the confluence and reverse the direction of flow at the confluence. This causes water levels on the Clearwater River to rise and may result in flooding in downtown Fort McMurray which is located at the confluence of the Clearwater and the Athabasca rivers.
4. River Ice Monitoring and Operational Innovation

In order to continue to provide Albertans and emergency response personnel with the best and most up-to-date information, Alberta Environment is continually evaluating and refining its river ice monitoring program. We continue to look for methods and technology that will improve the river ice program.

One of the challenges of monitoring river ice processes at Fort McMurray and the Town of Peace River is the remote nature of most of the river reaches. Although the areas that are of specific importance with respect to flooding are populated, it is the upstream and downstream reach ice conditions that must be monitored in order to provide forecasts. Due to this remoteness, monitoring networks of real-time hydrometric gauges, along with aerial observation flights, are typically utilized to gather data.

On the Athabasca River, the real-time hydrometric network is composed of WSC gauges as well as gauges owned and operated by AENV. This network of monitoring stations collects hydrometric and meteorological data, and is also equipped with alarms that indicate ice movements or when water level thresholds are exceeded (Robichaud et al, 2005). Table 1 lists the stations on the Athabasca and Clearwater rivers that have been equipped with alarms. Table 1 also provides the AENV real-time call name, distance upstream of Fort McMurray, and the types of alarms that the stations have been equipped with. The types of alarms are:

- **Water Level (WL)** – sets a threshold water level that when exceeded will trigger an alarm.
- **Ice Movement Indicator (ZI1) – Pull Wire.** Installed in early spring, the pull wires are connected to the monitoring station on one end and are connected to a weight on or frozen into the ice cover, typically a log. When breakup at that location begins and the ice shift or is shoved downstream, the pull wire breaks its connection to the monitoring gauge causing an alarm to be transmitted.
- **Water Level Difference Indicator (ZI2) –** alarm is set based on rate of rise. When the difference between two sequential water level readings exceeds the preprogrammed alarm threshold, the alarm is transmitted.

Figure 3 shows a map of the Athabasca and Clearwater rivers and the locations of the monitoring stations along the rivers upstream of Fort McMurray that are used heavily for river ice breakup monitoring. The Athabasca River at Athabasca River station is not featured in Figure 3. Three of the monitoring stations upstream of Fort McMurray were specifically installed as ice monitoring stations. These stations are:

- Athabasca River below Crooked Rapids;
- Athabasca River below Cascade Rapids; and
- Athabasca River above Mountain Rapids.

Each of these three stations is equipped with Random GOES capability. Data transmissions via the GOES platform are transmitted on a one to three hour interval. This time scale is not frequent enough to provide municipalities with sufficient response time should a large ice run occur. In order to maximize the response time offered by the data collected by the gauges for emergency officials, Random GOES transmissions are now utilized. A first in Alberta, Random GOES
technology utilizes the GOES satellite to transmit data however, rather than sending data at scheduled time, data is sent whenever a piece of data breaches the preset threshold. After the Random GOES transmission is sent to Edmonton via satellite, the AENV data collections system generates an email alarm that sends a text message alert to a predetermined group of cell phones. This provides personnel with the alarm regardless of the time of day; the approximate time that it takes from when a threshold is breached to when the cell phone receives the text message is 2-3 minutes. This has greatly improved AENV’s capability to provide advanced warning to downstream municipalities after an alarm is triggered.

After the success of the Random GOES transmissions in the Athabasca River ice monitoring program, its use has been expanded into the Peace River monitoring program. Two WSC stations have been equipped to send the random signals: the Peace River at Peace River and the Peace River above the Smoky Confluence.

Another recent addition to the data collection system for the river ice monitoring program is the use of an acoustic sensor at the AENV Grand Rapids station (Mahabir and Garner, 2007). This sensor is placed on the end of a boom that is extended out over the river and connected to a GOES station on the bank. The downward facing sensor emits sound waves and measures the return time of the reflected sound waves. Snow depth as well as ice elevation can be monitored with this setup. Any rapid changes in ice level, indicating a possible ice run, is relayed to the ice monitoring team.

In addition to the remote monitoring network, other new technologies and operational strategies have been incorporated into the programs. The new technologies include; using satellite imagery to determine information regarding the ice cover (the imagery is being utilized on both the Athabasca and Peace rivers) and a Shallow Water Ice Profiler Sonar (SWIPS) unit for recording data related to frazil ice on the Peace River through a partnership with BC Hydro. The new operational strategies that have been developed on the Peace River, as part of the ongoing work conducted by the JTF, include a new criterion for determining the cessation point for controlled flow releases during freeze-up and the examination of increased flows to improve flow efficiency and thereby lower water levels in certain areas.

As discussed previously, one of the challenges of monitoring the river ice processes along these rivers is the remote nature of the rivers themselves. Frequent surveillance can be difficult to maintain and alternatives are being explored. An alternative that is currently being adapted into AENV’s monitoring operations is the use of RADARSAT-1 satellite imagery of the ice covers on the Peace and Athabasca rivers. Information has been provided regarding the ice covers in near real-time to Alberta Environment since 2005 through a partnership with C-CORE, as part of its Polar View river ice monitoring service. The satellite images are processed by C-CORE using backscatter profiles to determine the extent and condition of the ice cover along specific river reaches. Colour coded maps are produced that depict the extent and condition of the ice cover along the river. On the Athabasca River, archival satellite images have been processed along with newly acquired images. Figure 4 shows an image taken on April 18, 2008. The ice cover condition has been classified into three types: water or possibly water on ice, non-consolidated or intact ice, and consolidated ice. This image shows that the ice cover from km 315 to km 460 is thermally deteriorated and that the channel has opened up in numerous locations. Fort McMurray.
is located from km 295 to km 286. This image was ground truthed and proved to be very accurate in capturing the current ice conditions. On the Athabasca River, these images are used in preparation for breakup operations to determine the condition of the ice cover prior to and throughout breakup and to provide additional information on reaches that are difficult to monitor frequently.

On the Peace River, C-CORE has been working on providing satellite images that will provide information regarding the advance and recession of the ice cover. Knowing the position of the ice cover is extremely important to the decision making process of the JTF. Figure 5 shows two successive images taken over the same reach of the river and the ‘change detection’ image produced from those images. These images provide not only the position of the ice front, but the condition of the ice cover in the vicinity of the ice front. This data can then be used to make operational decisions and validate the results of computer models currently being employed by the JTF to simulate the advance of the ice front. Ultimately these images may allow for a reduction in the number of observation flights used to document ice cover condition and it will provide for an alternative means of ascertaining the position of the ice front when inclement weather precludes aerial observations.

The deployment of the SWIPS (Shallow Water Ice Profiler Sonar) instrument in the Peace River is another technology that will aid in our understanding of river ice process. The SWIPS is an upward looking sonar device that utilizes acoustic backscatter to measure the distance to various objects within the water column. A profile of the water column is produced that provides spatial information regarding suspended particles such as frazil ice, bottom of frazil pans, and the water surface. This data can then be correlated later on with other data such as; air temperature, position of the ice front, water levels, to provide additional information regarding freeze-up, breakup and frazil ice distribution during the river ice season.

Analysis of the data collected during freeze-up by the SWIPS has shown that suspended frazil ice concentration and surface frazil pan concentration decreases during increases in solar radiation, when surface ice concentrations are below 65% (Jasek, 2007). When surface ice concentrations were above 65% only the suspended ice concentrations were reduced by higher solar radiation. Suspended frazil concentration also decreased, despite freezing air temperatures, as the ice front advanced toward the position of the SWIPS instrument, however after the ice front had passed and continued upstream, frazil ice concentrations increased with the increases in water level. This suggests that as the ice cover consolidates it creates enough turbulence to release the frazil ice into the water column. It was also observed that under a stable ice cover, the frazil ice distribution within the water column is directly related to water flow speeds and water level. Jasek (2007) provides detailed information regarding the SWIPS instrument on the Peace River.

5. Trial Operational Procedures on the Peace River

Depending on the operational scenario, there are conditions where the previously agreed upon typical course of action is not suitable and new options must be explored. Currently, AENV is in the evaluation process of two new operational options for use on the Peace River. The first is a new trial criterion for the cessation of flow control during freeze-up at the TPR and the other is an operational response to a secondary consolidation at the TPR.
Currently, the official criterion that is used to mark the point at which flow control can be lifted after freeze-up at the TPR has occurred, is to have thermal ice upstream of the TPR that is considered to be sufficiently thick so as to arrest a consolidation. This is to prevent a consolidation from compromising the stability of the ice cover within the TPR. Operationally, this agreed upon criterion has been applied as 0.40 m of thermal ice thickness at Dunvegan (approximately 100 km upstream of the TPR). Once 0.40 m of thermal ice was achieved at Dunvegan, controlled flow could be lifted however the JTF identified this criterion as a limiting factor when attempting to respond to ice related issues during freeze-up. It also could represent a lengthy restriction on BC Hydro’s production capacity, as in some years the ice front would not reach Dunvegan, let alone have 40 cm of thermal ice form.

This necessitated a new criterion that would allow operational responses to occur sooner and to place less of a restriction on BC Hydro’s operations. The new trial criteria proposed measuring 0.40 m of thermal ice in a 10 km reach starting from McLeod Cairn (approximately 9 km upstream of the TPR) (Jasek, 2006). Once 0.40 m thick thermal ice could be measured at McLeod Cairn and at a point at least 10 km upstream of McLeod Cairn, flow controls could be lifted. This criterion has been employed operationally for the last two years on a trial basis and is continuing to be evaluated. The JTF is finding the current results promising and is encouraged by the operational flexibility that it provides for responding to freeze-up related issues, however, further evaluation and analysis is required before it is officially adopted into the JTF operating procedures.

A new operational response to a secondary consolidation at the TPR that is being explored by the JTF is when the secondary consolidation raises the water level within the Town above the groundwater seepage guideline. As described previously, when the water level is above the guideline for a period of approximately 2 weeks or longer, groundwater seepage flooding begins to occur in basements within the subdivision of Lower West Peace at the TPR. Obviously, decreasing the water levels as much as possible and ideally below the groundwater seepage guideline is the goal of the JTF when faced with this operational scenario. However, this can become difficult due to the additional surface and frazil ice that is now within the reach and the JTF operational limitations associated with BC Hydro’s need to maintain system stability and load demand. Immediate decreases in flow releases may not bring the water levels down enough and BC Hydro may not be able to maintain them. Therefore the JTF has identified another possible course of action whose results are still being analyzed and evaluated. This solution consists of maintaining higher flows and possibly implementing flow increases to decreases water levels at the TPR (Friesenhan, 2005). This new operational solution is predicated on the assumption that higher flow releases will redistribute the frazil ice underneath the secondary consolidation and improve flow efficiency. Once flow efficiency is improved then water levels should decrease. Initial results are promising though further analysis and evaluation are required before this operational solution is considered a viable option (Jasek et al, 2005).

6. Conclusions
The introduction of new monitoring techniques and technology to AENV’s river ice monitoring program has been successful in providing additional and timely information to local municipalities and emergency personnel. The addition of river ice monitoring stations equipped
with Random GOES capability, the installation of the SWIPS unit and the use of satellite imagery provided by C-CORE has enhanced our monitoring capability and shortened our response time, in addition to enhancing our understanding of river ice processes on the Peace and Athabasca rivers. The information regarding river ice processes, provided by these innovations, is of critical importance to local municipalities to properly plan and respond to a river ice emergency. In order to ensure that the highest quality information possible is acquired and conveyed to those parties, AENV is committed to continually improving its river ice monitoring program. AENV continues to explore and evaluate new technologies related to river ice data collection and transmission.

7. Acknowledgments
Many of the new techniques and innovations for river ice operations and monitoring presented in this paper were made possible by the significant contributions of Martin Jasek from BC Hydro and Dr. Faye Hicks from the University of Alberta. Alberta Environment would like to acknowledge their significant contributions to our river ice program.

8. References


Figure 1. Map of the Peace River basin within Alberta and river reaches in the vicinity of the Town of Peace River.
Figure 2. Map of Athabasca River basin and river reaches in the vicinity of Fort McMurray.
Table 1. River ice operational monitoring stations.

<table>
<thead>
<tr>
<th>Station Name</th>
<th>AENV Station Abbreviation</th>
<th>Distance upstream of Fort McMurray (km)</th>
<th>Alarm Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Athabasca River at Athabasca</td>
<td>RATHATH</td>
<td>392.1</td>
<td>WL</td>
</tr>
<tr>
<td>Athabasca River above Grand Rapids</td>
<td>WATHGRAN</td>
<td>132.5</td>
<td>WL, ZI2</td>
</tr>
<tr>
<td>Athabasca River below Crooked Rapids</td>
<td>RATHCKRP</td>
<td>35.1</td>
<td>WL, ZI1, ZI2</td>
</tr>
<tr>
<td>Athabasca River below Cascade Rapids</td>
<td>RATHCARP</td>
<td>25.2</td>
<td>WL, ZI1, ZI2</td>
</tr>
<tr>
<td>Athabasca River above Mountain Rapids</td>
<td>RATHMTRP</td>
<td>17.1</td>
<td>WL, ZI1, ZI2</td>
</tr>
<tr>
<td>Clearwater River at Draper</td>
<td>RCLEDRAP</td>
<td>15.0</td>
<td>WL</td>
</tr>
<tr>
<td>Clearwater River above Christina River</td>
<td>RCLECHRI</td>
<td>38.0</td>
<td>WL</td>
</tr>
</tbody>
</table>

Figure 3. Map of the river ice monitoring stations up along the Athabasca and Clearwater rivers near Fort McMurray.
Figure 4. a) Processed satellite image of the Athabasca River ice cover on April 18, 2008. b) Crooked Rapids (km 334) flow is from right to left. c) Km 360 to km 350, flow is from bottom to top.
Figure 5. Satellite imagery of the Peace River a) December 28, 2006 satellite image. b) January 2, 2007 satellite image. c) Change detection image generated by C-CORE.
Improve Forecast Method on Ice Dam in The Heilongjiang River

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Abstract
The article analyzes the hydrologic and meteorological features of the river in high and cold mountainous area as well as the characteristics of river course, based on which to study the cause of formation of ice dam, to calculate the river course water (ice) storage, the through flow of melted snow and of rain (snow) fall during the melting period, as well as the stability of critical ice cap, so as to build a forecast model on the highest water level of the ice dam, to improve the methods of forecast and to increase the accuracy of forecast.

Key words: forecast method; ice dam flood; snow runoff; rain fall; ice cover
1. Introduction

The upper reaches of Heilongjiang River where the most ice dams occur are located in high and cold mountainous area. The rivers in this area are generally narrow with varied depths, widths and big drop. The climate here is frigid, the ground is by snow and the soil is frozen. In a year, the river has been covered by ice for 170 days and the soil has been frozen for nearly 300 days. In spring, the climate is unusual with relatively higher water (ice) storage in river source and thicker snow deposit. When temperature rises up rapidly and is accompanied by rainfall in spring, there will be big ice flood formed in upper reaches. In narrow or bending places along the river course where it’s hard for icy and water to pass through, there will be ice jam and ice dam in the front of ice cap. As for a river located on high and cold mountainous area, the formation of ice dam is actually a process that, under the condition of ice melting in an inverted order from downstream to upstream, the ice flood created by melt ice and snow and rainfall breaks the icecaps with powerful water flow force and results in the gathering and pilling up of broken icecaps. The specific size of ice dam is decided by the amount of water (ice) storage and winter snow deposit in the river course during the earlier stage as well as by the amount of rainfall, the progress of ice melting in an inverted order, and the stability of icecaps during the melting period.

If we take an ice dam flood as a single flood process, then the total amount of flood can be calculated according to following formula:

$$W_T = W_I + W_S + W_R$$  \[1\]

where, $W_T$ is total amount of ice dam flood ($m^3$), $W_I$ is water amount of ice in the river course, is the amount of water stored in the river course, $W_S$ is amount of melt snow, and $W_R$ is amount of rainfall during the melting period. The upstream of Heilongjiang River is located in mountainous area with a large gradient and drop. Since the climate is frigid, the water storage in the river is little and can accordingly be ignored. Therefore, the total flood amount and the water level of flood peak are decided by the water amount of melt snow and ice and of rainfall, while the level of dammed water during the flood peak is closely related to the degree of ice melting in inverted order as well as to the stability of icecaps. In the case of sole relation between ice jam flood amount and peak, the highest water level of ice dam can be roughly calculated according to following formula:

$$H_m = H_0 + \Delta H$$

$$H_0 = f(R_S + R_R + R_I)$$

$$\Delta H = f(\varphi h)$$  \[2\]

where, $H_m$ refers to the highest water level of ice dam in the form of certain range in unit of meter; $H_0$ refers to the normal water level during the process of mixing the water amount of melt snow and ice and of rainfall in the unit of meter; $R_S$, $R_R$, $R_I$ refer to the water amount of melt snow, rainfall and melt ice respectively in the unit of $m^3$, $\Delta H$ refers to the additional dammed water level caused by the accumulation of ice and icecaps; $h$ refers to the thickness of critical ice, $\varphi$ is the bending resistance coefficient of icecap in the unit of $kg/cm^2$. Formula (2) basically reflects the relationship between water amount balance and heat balance caused by ice dam jam
flood, while \( \Delta H \) also indirectly reflects the form and degree of ice jam flood under the condition of ice melting in inverted order.

2. Calculation of Ice Jam Flood
According to formula (1) and (2), in order to get the water amount of ice dam jam flood and the level of the dammed water, we should calculate the total through flow of melt snow and rainfall in corresponding drainage area and river section as well as the amount of ice stored in the river course.

2.1 The calculation of melt snow and rainfall through flow
As for the through flow of melt snow and rainfall, it’s mostly a process of water mixing during the melting period. As for the upstream of Heilongjiang River, this is the main source for ice dam jam flood. We analyze the hydrologic model of frozen soil in cold area and adopt the following water amount balance formula

\[
R = P - E - (W_m - W_0) \quad \text{[3]}
\]

In the formula: \( R \) refers to the total amount of melt water and rain fall in the unit of mm; \( P \) refers to the amount of snow and rain fall during the winter in the unit of mm; \( E \) refers to the amount of evaporation and emission in the unit of mm; while \( W_m \) and \( W_0 \) refer to the maximum and initial water storage amount in the drainage area respectively. During the melting period, both the method of flow concentration and the water amount of melt snow and rain fall have been affected by the depth of melting frozen soil as well as by the water amount contained in soil. The theories of runoff yield under saturated storage apply to both the available field-carrying capacity and the method of flow concentration. But being affected by the special features of frozen soil and dynamic rules of water content in soil, the method of flow concentration and the fixed quantity of parameters will change with the depth of melting frozen soil. During the melting period, the flow concentration layer of soil can in fact be divided into two layers of melting layer (the layer with air) and frozen layer (the layer with ice). The calculation melt snow and rain fall through flow and the fixed quantity of parameters has material difference from the calculation of rain flood during frozen-free period of time. And the rain and snow through flow during melting period can be calculated according to following formula:

\[
R = f(P_e + W_0) = f(P_e + W_{0u} + W_{0L})
\]

\[
R_{\text{Rain & Snow}} = f(P_e + W_0) = f(P_e + W_{0\text{upper}} + W_{0\text{lower}}) \quad \text{[4]}
\]

where \( P_e \) refers to effective amount of water (snow) fall; \( W_0 = W_{0u} + W_{0L} \), \( W_{0u} \), \( W_{0L} \) refer to the initial water storage amount of the upper and lower layer respectively. The maximum water storage capacity of the drainage area \( W_m \) is divided into two rain layers of upper and lower parts; from the date that the frozen soil starts to melt, along with the increase of temperature and the change of depth of melting frozen soil, the upper layer changes from zero to \( W_m \), while the lower layer changes from \( W_m \) to zero, so as to calculate \( W_{0u} \) and \( W_{0L} \) respectively. The upper layer is affected by the evaporation capacity, while the lower layer is affected by the melting speed of the frozen soil, then the initial amount of water stored in upper and lower layer is,
\[ W_{0a,t+1} = W_{0a} + p_t + e_t + r_t \]
\[ W_{0L,t+1} = K_{Dt} \cdot W_{0L} \]
\[ K_{Dt} = \left(1 - \frac{t}{T}\right)^a \]

where \( W_0, t \) and \( W_0, t+1 \) refers to the amount of water stored at the end of the period in the unit of mm; \( K_{Dt} \) and \( a \) refers to the run-out coefficient and index of the amount of water stored in lower layer. Analysis shows that the change of frozen soil depth is almost a straight line, therefore the value of \( a \) is equal to 0.8.

2.2 The calculation of water amount stored in the river

There is no water conservancy project along the upstream of Heilongjiang River to adjust the water flow; therefore the minimum flow in winter is no more than 20\( m^3/s \), and the most part of water stored in the river is in the form of ice. The amount of ice stored in the river course can be calculated with the cross-sectional method according to the following formula,

\[ W_I = \int_0^L h_m B dl \]

where \( h_m \) refers to thickness of ice layer in the unit of meter; \( B \) refers to the width of ice surface, and \( L \) refers to the length of river in unit of meter. While calculating by different river sections and taking into consideration of different widths of ice surface, the following formula can be used

\[ \Delta W_I = h_m \frac{B + B_c}{2} \Delta L \]
\[ W_I = \sum_{i=1}^n \Delta W_I \]

where \( B \) refers to average width of upper and lower section when ice starts melting; \( B_c \) refers to average width of both upper and lower section of ice surface in the unit of meters; \( h_m \) refers to the average maximum ice thickness of upper and lower sections in unit of meter; \( \Delta L \) refers to the distance between two sections in the unit of meters; \( n \) refers to the number of river sections.

According to the above calculation, the amount of ice stored in the river course is directly related to the water level of ice dam; but the thickest ice mostly appears around the last ten days in March, and there’s a process of temperature rising and ice diminishing till the melting period starting in the last ten-day period in April. Taking into consideration of actual critical ice depth and icecap stability, we should also calculate the depth and strength of icecaps.

2.3 The calculation of critical ice thickness and icecap strength

During the melting period, after the air temperature rises, the ice layer starts to melt and reduces in depth; its pressure resistance accordingly reduces either. If the thickness of ice layer has changed \( dh \) within a period of time of \( dt \), the differential equation of ice depth changing during a period of time is,
\[
\frac{dh}{dt} = \frac{\lambda(T_w - T_a)}{\rho L}
\]  

[8]

In the formula: \( L \) refers to the latent heat of phase change of ice (calorie/cm³); \( \rho \) refers to the density of ice (g/m³); \( T_w \) and \( T_a \) refer to the temperature of ice surface and the lower layer of icecap respectively; \( \lambda \) is the coefficient of heat-transfer of ice. When there’s no snow and no wind on the ice surface, and not taking consideration of riverbed heat radiation, to replace \( T_w \) with temperature \( \theta \) and to make \( T_a = 0 \), to calculate the integration of formula (8), we can get the reduced depth of icecap:

\[
h_C = K \sqrt[n]{\sum_{i=1}^{n} \theta}
\]  

[9]

where \( K \) is a comprehensive coefficient reflecting thermal conductivity, the density of ice, and latent heat; \( n \) refers to the number of time intervals in the unit of days; \( \theta \) refers to average daily temperature in the unit of °C. With the method of diachronic factors, the reduced ice layer during the melting period is

\[
h_t = h_0 \left(1 - \left(\frac{t_n}{T}\right)^{1.5}\right)^2
\]  

[10]

where \( h_t \) and \( h_0 \) refer to ice depth (m) and initial ice depth (\( t_n \)) in calculating \( t \), \( T \) refers to the total time used for calculating \( h_t \) and the icecap melts away completely (day). Similarly, the declining process of icecap strength can be calculated with following formula,

\[
\Phi = \varphi_0 \left(1 - \sqrt{\frac{t}{T}}\right)^2
\]  

[11]

where \( \varphi \) and \( \varphi_0 \) refer to the icecap strength coefficient at the time of \( t \) and the strength coefficient at the beginning of calculation (In Russia \( \varphi_0 \) is 5.5kg/cm²), and the critical icecap strength \( \varphi_h \) during the melting period can be calculated according to the formula (11).

3. Calculation of Ice Dam Stability

The stability of an ice dam is closely related to the time it lasts and depends on the strength of icecap, the length of river course jammed, the level of dammed water and the flow capacity. The Galinda Ice Dam was formed on 27 April 1960; its water level reached 13.56m and the ice dam collapsed on 12 May, lasting a total of 14 days. The ice dam more than 10m high lasted 8 days, including 3 days after the ice stopped flowing from the upstream with relatively high degree of stability. The stability of an ice dam is related to the features of both ice dam and river course. The length of ice dam observed in Russia is 50km, and the rate of flow is about 3,500m³/s; the river section is narrow with a width of 400km -520km, which contributes to the stability of ice dam. In the past, the ice dam jammed here has lasted for relatively a long period of time,
including more than 5 days in 1958, 1964, 1971, and 1973 and more than 14 days in 1960 and 1980.

4. Example of Forecast
The main source of the ice dam jam flood on Heilongjiang comes from the Argun River as well as the section from Moguhe to Oupu in the meddle and lower reaches of the Shile River, after calculating according to formulas (3) ~ (6), the through flow of melting snow and rain fall $V_S$ and $V_R$, the amount of water frozen $V_{Ice}$, we conclude the highest ice dam level of Oupu Station as shown in Figure 1, which shows a close relationship between them and can be used for relatively accurate forecast. In Figure.1 we know that curve-a is obtained with equation $H_m=f(V_S+V_R)$, and curve-b is obtained with equation $H_m=f(V_S+V_R+V_I)$. Taking into consideration of icecap strength $\phi_h$, we calculate according to formulas (10) and (11) the highest water level of ice dam in Mohe Station as shown in Figure 2, indicating the direct relationship between the highest levels of ice dam with the strength of icecap.

![Figure 1: Highest ice dam level of Oupu](image1)

![Figure 2: Function of $\phi_h$ and $h_m$ at Mohe](image2)

As for the calculation of additional dammed water level, taking Galinda station as an example, the relationship between the highest ice dam jam flood level $h_m$ in Mohe and Galinda Station is
shown in Figure 3. The outside line in Figure 4 is the water level under the assumption that there is no ice dam jam, that is, $H_0$ in formula (2), and $\Delta H$ is the additional dammed water level of ice dam; and the regular water level of Galinda $H_0$ can be obtained through the water level in Mohe Station, $\Delta H$ is got through actual measurement in Galinda with $H_m-H_0$. The relationship between additional dammed water level and icecap strength coefficient $\phi$ is shown in Figure 4. As for a river located in high and cold mountainous area with water conservancy project to regulate the water flow, the amount of water stored in river course is small and mainly in the form of ice, which is decided by the size of rain fall in early fall and that of snow fall during the frozen period. Under natural conditions, factors including low temperature and high strength of icecap play an important role in deciding the level of ice dam. Therefore, for most rivers, the study of the relationship between frozen water level and the highest level of ice dam ensures a relatively accurate forecast and the forecast is also applicable for a long period of time.

![Figure 3](image1.png)

**Figure 3** Relationship of $h_m$ between Mohe Station and Galinda Station

![Figure 4](image2.png)

**Figure 4** Function of $\phi$ and additional dammed water level $\Delta H$ at Galinda Station

But the forecast made on the highest ice dam level also has certain inaccuracy, mainly because that the regional scope of rainfall and snow and ice melting is inaccurate. This, together with the few rain-gauge stations, the measurement of ice depth is also not accurate enough and it’s hard to
decide on the starting date for $\varphi$, the incoming water amount for the calculation of highest water level is merely an approximate value. Still, this method is a big step forward in comparison with previous experience-based method in the explanation of physical cause of formation of ice dam in the river located in high and cold mountainous area as well as in theories and analyzing methods.

5. Conclusions
The rivers located in high and cold mountainous area are different from those in low-altitude plains in terms of hydrographic and metrological features and the characteristics of river course. Accordingly, the formation cause of ice dam is also different from that of the rivers located in plain area. The subsequent water source for the ice dam in a river of plain area is limited and the study of ice dam formation can focus on the relationship between ice water driving force and resistance. As for the formation of ice dam in high and cold mountainous area, apart from the relationship between ice water driving force and resistance, other deciding factors include the amount of ice, the through flow of melt snow and rainfall, as well as the form and process of water concentration. With ice melting in inverted order, the ice and flood combine in different sections during the process of flood and accumulate at the edge of original icecaps on the way the flood rushes. Therefore, the water amount of ice dam jam flood is decided by the amount of ice stored in the river course, the through flow of melt snow and rainfall, and the strength of icecap.

The article is based on the hydrologic and meteorological features of the river in high and cold mountainous area as well as on the hydrologic effect of frozen soil, calculates the highest water level of ice dam according to the principle of runoff yield under saturated storage by considering the through flow of melt snow and rainfall as well as the amount of input water and icecap strength, and the result is fairly satisfactory.

Reference


Ice-Jam Measurements versus Model Predictions:
A Case Study, Matapedia River, 1995

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Abstract

During the 1995 spring breakup, an ice jam formed on the lower Matapedia River and remained in place long enough to permit collection of field data on the longitudinal variation of its thickness as well as on water surface elevations along its length. Together with bathymetric data that were collected in the following summer, the spring measurements form one of the most complete ice-jam data sets that have been obtained to date. Application of the model RIVJAM reproduced the measured water level profile of the jam very closely, while a reasonable simulation of the average jam thickness was obtained. The availability of thickness data helped minimize the ambiguity that is usually associated with model calibrations, and enabled direct assessment of the jam roughness. Model sensitivity was explored by changing calibration parameters and examining the model output for those runs that adequately reproduce the observed jam length. The results revealed that the model is moderately sensitive in this particular case study, much as had been found in earlier test runs using hypothetical examples.
1. Introduction

Ice jams have frequently occurred along the Matapedia River, Quebec, occasionally resulting in damage to property and infrastructure. In the flood of 16–18 April 1994 (Beltaos and Burrell, 1995), severe ice runs and jams along the Matapedia River destroyed the St. Alexis bridge and moved the superstructure of a railway bridge across Clark Brook, near the town of Matapedia, about 8 m off its foundations. Matapedia, situated at the confluence of the Matapedia and Restigouche rivers, was affected by ice jams in both rivers.

An ice jam that formed on the Matapedia River in 1995 was more benign, but offered excellent data collection opportunities. It remained in place long enough in an accessible location to permit measurement of the water level and thickness profiles throughout the length of the jam. Thickness was measured with an “Ice Jam Profiler” (IJP), a flow-propelled drogue that reports real-time data by radio-telemetry (Ford et al., 1991; Beltaos et al., 1996), providing a practically continuous time series that is later converted into a longitudinal profile. The large number of measured thickness values enables statistical determination of the absolute roughness characteristics of the jam underside (Beltaos, 2001), a parameter that is normally assessed indirectly via flow resistance calculations.

Severe ice jams usually attain an “equilibrium” condition, characterized by a long reach of relatively uniform thickness and flow depth, separating short transitional reaches at its upstream and downstream ends. Thus, the available measurements often pertain exclusively to the equilibrium reach. Calibrations with “equilibrium” data sets do not really test a model’s differential formulation for thickness and flow depth because longitudinal gradients essentially vanish in equilibrium reaches. Of the two transitions, the most important by far is the downstream one because it contains the “toe” (downstream end) of the jam, where water surface slope and jam thickness are maximized. It is in this region that the potential of bed scour is greatest (Wuebben, 1988; Ziegler et al., 2005). Being relatively short, the 1995 Matapedia River jam was largely comprised of transitional reaches, as will be illustrated later (Figure 2).

The resulting data set is, to the writers’ knowledge, unique in its detail and model-testing opportunities. It was recently used in a “blind” test of several models (Carson et al., 2007), which was performed under the auspices of the Committee on River Ice Processes and the Environment (affiliated with the Hydrology Section of the Canadian Geophysical Union). In the blind test, the models were applied in predictive mode without prior calibration; the modelers were only given channel bathymetry and flow as well as jam location and length. For obvious reasons, the RIVJAM model (Beltaos, 1993, 1996) was excluded from the blind test; however, the 1995 Matapedia River data set made it possible to evaluate theoretical concepts of ice-jam hydromechanics that have been incorporated in this model.

The purpose of this paper is to describe the data set and evaluate the performance and sensitivity of RIVJAM when applied in calibration mode. Following descriptions of the study reach and of the field measurements, the main features of the RIVJAM model are outlined, and its application to the Matapedia River dataset is discussed with respect to calibration, sensitivity, and with previously established ranges of model coefficient values. The hydraulic roughness of the jam is quantified next, and compared with values obtained from the model.
2. Study Reach

The Matapedia River on the Gaspé Peninsula, Quebec, is a tributary of the Restigouche River, which defines a portion of the boundary between the Canadian provinces of Quebec and New Brunswick. The river flows 65 km southwardly from Matapedia Lake at Amqui to Matapedia. The study reach is the lower 38 km from Routhierville to Matapedia. The Matapedia River basin lies in the Notre Dame physiographic region, a portion of the northern Appalachians, with worn plateau-like, flat-topped hills, often over 900 m in elevation. The Matapedia River carved a narrow channel through the Notre Dame hills, which partially shade the channel during the spring breakup period. Characteristic flows at the nearest Water Survey of Canada hydrometric station, which was in operation in 1995, are tabulated below.

### Table 1. Daily Flows Observed at Hydrometric Station 01BD002, Matapedia (Riviere) En Amont de La Riviere Assemetquagan (Lat/Long: 48° 5' 12¨N/67° 6' 2¨W)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1968-95 Long-term Winter Season Flows (m³/s)</th>
<th>1994-95 Winter Season Flows (m³/s)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Dec</td>
<td>Jan</td>
</tr>
<tr>
<td>Average Daily</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Daily</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Daily</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


The Matapedia River basin has a moderate continental climate. Table 2 presents the climatic normals (1970-2000) for three climatic stations in the Matapedia River basin. End-of-March snow cover depth is approximately 60 cm. Snowmelt due to rising temperatures in April combined with occasional rainfall often results in eventful breakup and jamming.

### Table 2. Climatic Normals for the Matapedia River Basin

<table>
<thead>
<tr>
<th>Climate Station</th>
<th>Mean Annual Rain (mm)</th>
<th>Mean Precipitation (mm)</th>
<th>Mean Temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Jan</td>
<td>Feb</td>
<td>Mar</td>
</tr>
<tr>
<td>St Alexis de Matapedia</td>
<td>722</td>
<td>340</td>
<td>81.3</td>
</tr>
<tr>
<td>Causapscal</td>
<td>732</td>
<td>294</td>
<td>73.5</td>
</tr>
<tr>
<td>Amqui</td>
<td>665</td>
<td>327</td>
<td>75.4</td>
</tr>
</tbody>
</table>

Source: Adapted from Environment Canada (1996).

3. Field Measurements

The 1995 ice jam on the Matapedia River was noticed at about 1430 h (all times are in ADT) on April 22. Breakup on the Matapedia had already commenced but there were sections of the river that still had sheet ice cover. The jam was located just downstream of the St Alexis bridge, as shown in Figure 1. The channel was open for ~ 3.3 km upstream of the jam, and shear walls were
evident in that reach. According to local residents, the ice ran past the bridge on April 21. Figure 2 illustrates the surface texture of the jam near its mid-length.

Figure 1. Plan view of Matapedia River in the vicinity of St Alexis and extent of ice jam, April 22, 1995.

Figure 2. Looking across the Matapedia River, April 22, 1995. Flow is from right to left.
Flow data at the nearest gauge (see Table 1), which is located 12 km above the St Alexis Bridge, indicate that runoff commenced in mid-April, accelerating in the next several days and reaching a partial peak on the 24th. According to Water Survey of Canada, the last day with ice effect on stage (or “backwater”) was April 22. The reported flow of 140 m$^3$/s would have been less than the value indicated by the open-water rating curve for the same river stage. A plot of hourly gauge data during the same period has indicated that the local ice cover moved out during the afternoon of the 21st, resulting in a sharp drop of about 1 m in the local water level. Therefore, the backwater effect on the 22nd would have been very small. This was confirmed by comparing reported flow (140 m$^3$/s) to preliminary flow (145 m$^3$/s), which was based on the open-water rating curve. The reported flow value of 140 m$^3$/s for April 22 is thus considered reasonably accurate. The pro-rated flow at the St Alexis bridge is estimated as 205 m$^3$/s, based on the basin area ratio of 1.46 (Gilles Barabe, Ministere de l’Environnement et de la Faune, Pers. Comm. 1995).

Following inspection of the river in the vicinity of the jam, an IJP unit was released in the river from the St Alexis bridge at 1525 h, and its progress tracked by radio signal (and visually in the open water reach between the bridge and the head of the jam). Non-zero thickness values began to be received as soon as the drogue submerged under the head of the jam, and continued until 1625 h, when it became evident that the drogue was stuck under the ice cover. The time series of thickness was converted to distance series by first noting the time of passage of the drogue at locations spaced ~100 m apart, and then interpolating, using the average drogue speed within each interval.

The longitudinal variation of the reported probe depth (equal to the draft of the jam or ~ 0.92 x thickness) is shown in Figure 3. The most striking feature of the jam profile is the large variability of the thickness, which is in accord with the very rough surface texture ice jams. The trajectory of the probe is entirely controlled by the flow, and its lateral position within any given transect is not known. Thus the profile can be regarded as a set of values of a random variable.

![Figure 3](image-url)  
*Figure 3.* Ice jam profile, Matapedia River above Matapedia, April 22, 1995. From Beltaos et al (1996b) with changes.
Despite the randomness, a clear trend toward increasing downstream thickness is evident in Figure 3, very much in accord with what is known about the mechanics of breakup jams (Beltaos, 1995). The location of the toe shown in Figure 2 is defined as the downstream limit of bank-to-bank rubble, and is coincident with the point of maximum thickness. Downstream of the toe, there was a decreasing percentage of rubble mixed with large plates of sheet ice, tapering to ~10% at 600 m from the toe. The presence of rubble downstream of the toe is implied in Figure 2, because the indicated thickness of 1-1.5 m is well in excess of the local sheet ice thickness of ~0.4 m.

Starting at 1700 h, the water level variation along the length of the jam was obtained by running a level survey as far downstream as was permitted by the fading daylight at 2100 h. Though it was not possible to “close” the survey on that occasion, temporary benchmarks were established at both ends and the difference in their elevations was re-surveyed starting at the downstream end in June 1995 (discrepancy, or closure error = 12 mm). Figure 4 shows the measured water levels, along with data obtained during the June survey under open-water conditions.

The channel bathymetry in the jammed reach was surveyed in detail during the June 1995 field trip. Cross sections at distances (above the river mouth) of 6.36, 6.79, 6.98, 7.07, 7.18, and 7.38 km were surveyed, and supplemented by sections at 7.80 and 8.41 km, which were measured in the summer of 1994 (Beltaos and Burrell, 1995). A typical section is shown in Figure 5, along with local water level and average jam thickness, as they were measured on April 22, 1995.

![Figure 4](image-url)

**Figure 4.** Water level variation along the ice jam of April 22, 1995 (flow = 205 m$^3$/s). Also shown are open-water levels in the same reach on June 13, 1995 (flow = 120 m$^3$/s).
4. The RIVJAM Model

RIVJAM is a one-dimensional, steady-state, numerical model that solves two ordinary differential equations (ODEs) with two unknowns, which derive from the balance between external and internal forces applied on an ice jam (Beltaos 1993, 1996). Flow characteristics are calculated on the basis of conventional, two-layer flow concepts, and composite hydraulic resistance considerations. Three key features distinguish RIVJAM from most other models of ice jams: consideration of flow through the voids of ice jams; use of an upstream-marching algorithm to solve the differential equations; and linkage of the composite friction factor, $f_o$, to the thickness of the jam and the average flow depth under it. The latter feature is expressed by the simple relationship:

$$f_o = c\left(\frac{t_s}{h}\right)$$  \[1\]

in which $c$ = dimensionless coefficient $\sim 0.4-0.6$; $t_s$ = submerged portion of the jam thickness or ice draft; and $h$ = average depth of flow under the jam. Equation 1 is based on empirical evidence that links the roughness height of the underside of an ice jam to its thickness.

5. Model Application to Matapedia River Jam

The application of RIVJAM to the 1995 Matapedia data set has three objectives. The first objective is to find out how well the model reproduces the measurements when the input
coefficients are selected so as to obtain good agreement between observed and computed results. Once this task is accomplished, a second objective is to study model sensitivity by changing various coefficients (one at a time) and assessing the magnitude of discrepancies between model output and measurements. The third, and perhaps the most important, objective is to determine how well the values of the model parameters, obtained in this unusually rigorous test, agree with values obtained in previous applications.

Model calibration: Rather than carry out a complete but laborious optimization procedure, it was decided to use established default values for those coefficients that experience has shown to be relatively constant in different applications and fine-tune the ones that appear to be more variable. The former category includes: jam porosity \((p = 0.4)\); ratio of internal streamwise stress to average vertical stress generated by buoyancy \((K_x = 12)\); and coefficient \(\mu\) (dimensionless parameter related to the internal strength of the rubble = 1.2). The following parameters were varied within known ranges until agreement with the data was optimized: coefficient \(\lambda\) (flow-through-voids characteristic, selected value = 1.5 m/s); ratio of ice- to composite-friction factor, \(f_i/f_o\), (selected value = 1.20); and resistance coefficient, \(c\), defined in Eq. 1 (selected value = 0.4). The results are compared with measurements in Figure 6, where it is seen that the model is very closely matching the measured water levels, and slightly under-predicts the measured thickness between 7400 and 7700 m.

The selected coefficient values are within previously established ranges from data on other rivers (Beltaos, 1996). A Matapedia River ice jam that occurred in 1994 upstream of the St-Alexis reach was also modeled with RIVJAM (equilibrium jam; Beltaos and Burrell, 1995), using a nearly identical set of calibration parameters. The only difference is in the coefficient \(c\) (Eq. 1), which was taken as 0.4 for the 1995 jam as opposed to 0.5 for the 1994 jam.

![Figure 6](image-url)

**Figure 6.** Model calibration results; \(p = 0.4; K_x = 12; \mu = 1.2; \lambda = 1.5\) m/s; \(f_i/f_o = 1.20; c = 0.4\).
Model sensitivity: Having established a set of input parameters that result in good agreement with the measurements, departures from the calibration output when the input parameters $c$ and $f_i/f_o$ are varied were examined. In each of these runs, toe conditions were adjusted until the simulated length of the jam matched the actual length. The results were still satisfactory when $c$ attained the upper limit (0.6) of its common range, but less so when $f_i/f_o$ dropped to the lower limit of its own common range (Figure 7). These findings are in agreement with those of a detailed sensitivity analysis performed on hypothetical examples (Beltaos, 1996).

![Figure 7](image)

**Figure 7.** Sensitivity run, $f_i/f_o = 1.0$ instead of 1.2 used in calibration run.

6. Hydraulic Roughness of the Jam

As discussed by Beltaos (2001), thickness data can be used to determine the absolute roughness of the jam, by taking the 84th percentile value of deviation from the mean ($\varepsilon_{84}$). After subdividing the profile of Figure 2 into ranges within which the average thickness does not appear to change significantly, statistical analysis was performed on each subset. The resulting absolute roughness increases linearly with thickness, as shown in Figure 8. A similar finding was reported by Nezhikhovskiy (1964) who used data from freezeup jams.

Using the friction-factor formula developed by Limerinos (1970) for open channel flow, and adapting it to the ice-controlled flow layer gives ($R_i =$ hydraulic radius associated with the ice cover):
Figure 8. Variation of jam roughness with thickness. Data points were generated from thickness measurements; linear fit has a slope of 0.41.

\[ f_i = [1.16 + 2 \log (R_i / \varepsilon_{84})]^{-2} \]  \[2\]

Using Equation 2 and the results of Figure 8, it is possible to calculate the value of \( f_i \) along the jam via thickness values obtained from the calibration run, which provides a fair description of the “average” jam profile (Figure 6). This variation can be compared with the values indicated by RIVJAM, using Eq. 1, along with the input value of 1.2 for the ratio \( f_i/f_o \). This comparison is shown in Figure 9, which suggests very close agreement.

Figure 9. Variation of ice jam friction factor with river distance, as determined by: (a) Eq. 2 with \( \varepsilon_{84} = 0.41t_s \); and (b) Eq. 1 with \( f_i/f_o = 1.2 \).
7. Summary and Conclusions

A favourable coincidence of local conditions (excellent accessibility, persistent, non-equilibrium jam) enabled acquisition of an unusually complete data set, which can be used for model testing that is far more rigorous than mere comparisons with a few water levels or with equilibrium-jam data. With a suitable choice of the model parameters, RIVJAM predicted the measured water level profile of the jam very closely, and provided a reasonable simulation of the average jam thickness, whose measured values exhibit strong spatial variability. The selected values of the model parameters are consistent with values established by previous applications. Model sensitivity was also explored by changing the values of two calibration parameters that are more variable than others, and examining the model output for those runs that adequately reproduce the observed jam length. The model was found to be moderately sensitive, in accord with earlier findings based on hypothetical examples.

8. References


Condition Forecast of Yellow River Inner Mongolia Reach

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Abstract: An Artificial Neural Network Expert System is developed for ice condition forecast of the Yellow River Inner Mongolia Reach. The system is based on GIS technique while its kernel is BP network arithmetic. The key parameter forecast includes the change of the water temperature with time and the beginning dates of the ice runs appearing, freeze-up and breakup, taken account of the hydraulic factors and thermodynamic factors, such as air temperature, water temperature and volumetric discharge as well as water stage and ice concentration on water surface. Using the expert system, the ice conditions in the 2004-2005 ice period and the 2005-2006 ice one, such as the water temperature and the beginning dates of the ice run, freeze-up and break-up etc, were forecast on-line. It has been verified that above 75 % of the forecast results are qualified or excellent while the forecast period is in 15 days.

Key Words: ice condition forecast, Yellow River Inner Mongolia Reach, Artificial Neural Network, expert system and GIS technique.
1 Introduction
The Yellow River is called as Chinese Nationalities' Cradle. The Inner Mongolia reach lies at the northern end of the Yellow River basin. It is 820km long. There the course of the river is serpentine and the slope is large in upper reach and small in lower reach. It is extremely cold in winter. The time of minus air temperature usually lasts 4 to 5 months and the lowest can reach -35°C. The characteristics of the river ice are that the freeze-up is from downstream to upstream in winter but the break-up is from upstream to downstream in spring. The ice-melt flood in spring increases from upstream to downstream quickly, usually leading to ice jams and ice dams and then causing the disasters. For instance, a river bank burst caused by ice flood occurred in Wulan river reach in Wuhai City on 17th December, 2002, which forced about 4000 residents in 50 km² area to be resettled emergently and resulted in the economic loss of about 20 million US$. Therefore, the accurate ice condition forecast is very important, so as to have time to take effective measures for potential disasters.

Now a great progress have been made in the simulation models of the river ice processes, including ice formation, the progression of ice cover and ice jam, and the changes of the corresponding discharge and water lever (Lal and Shen, 1991; Yang and the others, 2002). However, the key problem is that the models are hard to be used for the ice condition forecast in the Yellow River Inner Mongolia Reach. It is a heavy sediment-carrying river and its course varies with time. Few data the simulation models require, such as the sizes of the cross-sections of the river, are available.

In the Yellow River Inner Mongolia Reach there are only the four hydrologic stations, located at Shizuishan, Bayangaole, Shanhuhekou and Toudaoguai, respectively. Through them the hydrologic data in terms of the daily mean air temperature, water temperature and volumetric flow as well as the water stage and the ice conditions et al have been documented for decades and the real-time data are also available. This provides a possibility for the ice condition forecast with statistic analysis.

The existing ice condition forecast models for the Yellow River based on the statistic analysis may be classified as the three kinds: empirical indexes, correlation analysis and empirical mathematical models (Ke, Wang and Rao, 2000). In order to improve the accuracy of the ice condition forecast and to meet the requirements for on-line forecast, based on GIS-Geographic Information System technology, an ANN-Artificial Neural Network-Expert System has been developed. Both the data of the real-time hydrology and the results of the ice condition forecast will be not only displayed by numbers, charts and alarm signals on a computer screen, but transmitted on time to every decision departments by Internet. Its main contents will be described as below.

2 An Ice Forecast Model Based on ANN
The keys in the ice condition forecast for the Yellow River Inner Mongolia Reach are the beginning date of the ice runs appearing, the beginning date of freeze-up when the river are covered by ice and the beginning date of ice cover break-up as well as corresponding to maximum ice-flood discharge and water stage etc.

The ice condition forecast is affected by many factors, in which the hydraulic factors and thermodynamic factors, such as air temperature, water temperature and volumetric discharge as
well as water stage and ice concentration on water surface, are the key ones. The equation may be written as follows

\[ D_i = f(T_{w,j-1}, T_{a,j-1}, Q_{j-1}, H_{j-1}, C_{j-1}; T_{w,j}, T_{a,j}, Q_j, H_j, C_j; T_{w,j+1}, T_{a,j+1}, Q_{j+1}, H_{j+1}, C_{j+1}) \]  

[1]

where the subscript, \( i \), represents the \( i \)th hydrologic station; \( D = \) the forecast parameters; \( T_w = \) water temperature, \( T_a = \) air temperature, \( Q = \) volumetric discharge, \( m^3/s \); \( H = \) water stage, \( m \); and \( C = \) ice concentration on water surface, namely, the ratio of ice-covered surface to whole water surface. \( T_w, T_a, Q, H \) and \( C \) include the historical data and real-time ones. \( C \) may be replaced by the other data such as the time of the accumulatively negative air temperature if it is unavailable.

Equation [1] indicates that the forecast parameters, \( D_i \), depends not only on the hydraulic parameters and thermal ones at the \( i \)th hydrologic station, \( T_{w,j}, T_{a,j}, Q_j, H_j \) and \( C_j \), but also on those at its upstream and downstream hydrologic station, \( T_{w,j-1}, T_{a,j-1}, Q_{j-1}, H_{j-1}, C_{j-1}, T_{w,j+1}, T_{a,j+1}, Q_{j+1}, H_{j+1} \) and \( C_{j+1} \).

The main task of the ice condition forecast herein is to find out the relationship between the forecast parameter, \( D_i \), and the historical data and real-time ones.

Any of ANNs is of self-study capabilities. Based on experiences, it may make the reasonable deduction and give the satisfied solution for non-linear and uncertain puzzles in forecast. What is important, the existing traditional ice condition forecast models are limited in accuracy due to lack of observed data, complexity of corrective factors and puzzle of information etc. Accordingly, using an ANN to improve the ice condition forecast may be a good alternative. By comparison, BP network, Back-Propagation network, is used herein.

BP network has been world-widely used (Nagy, Watanabe and Hirano, 2002; J. S. Wu, Jun Han, Shastri Annambhotla). It has the strong ability of nonlinear projection and flexible network structure. All of its network structure, the number of layers, the number of nerve units and study coefficients can be adjusted according to the specific cases. And to realize such models is easy and quick.

By comparison and practice, a BP network with a hidden layer in Figure 1 is applied to the ice condition forecast. The vector \( \mathbf{X} = [x_1, x_2, \ldots, x_m] \) is the input vector, whose elements consist of \( T_w, T_a, Q, H \) and \( C \); the vector \( \mathbf{O} = [O_k, \ldots, O_l] \) is the output vector, whose elements consist of the forecast parameters; The vector \( \mathbf{W} = [w_{i1}, w_{i2}, \ldots, w_{im}] \) is the adaptive weighting factor vector. If the adaptive weighting

![Figure 1. BP network with a hidden layer](image-url)
factors are given by ANN self-training, the functional relationship among the input and hidden layer and output can be established.

3 Ice Condition Forecast Expert System
According to the requirements for the ice condition forecast, based on GIS platform and real-time hydrologic database, an expert system has been developed, adopting Object Oriented Programming (OOP). The framework of system is shown in Figure 2. It has the following characters.

3.1 GIS Platform and System Environment
The GIS platform used is Windows-based. The Main interface is made up of menu bar, tool bar, map window, and status bar, as shown in Figure 3. One can create, edit, and calculate the models within the GIS environment. All input data and output results, including time-variant information, are fully integrated with GIS database and accessible to GIS tools.

3.2 Data Warehouse of Ice Condition Forecast
The data warehouse of ice condition forecast is made up of the weather data, the real-time hydrologic data, the historical hydrologic data, and the spatial geographical data and attributes data etc. The real-time data are on-line derived from “The Real-time Water and Rain Regime Database in the National Flood Prevention System”, including the air temperatures, volumetric discharges and the water stages as well as the ice concentrations on water surface etc. The spatial geographical information and attributes data are logged as shapefile format supported by MapObjects, and correlated with other database’s tables through the special fields.

![Figure 2. Framework of ice condition forecast expert system](image-url)
3.3 Ice Condition Forecast Models
It is the kernel part of the system, including Statistic model, Mathematics model and ANN model. The contents of the ice condition forecast include not only the process of water temperature, volumetric discharge and water stage, but the beginning date of ice runs, ice cover thickness and date of break-up as well as the maximum flood discharge and maximum water stage etc. What is most important, based on the forecast results, one can analyze whether or not there are a probability that the ice jams or ice dams occur at some key positions.

3.4 Query and Display Functions
Based on GIS, one can dynamically query and display the historical and real-time water regime and ice regime information in the system, e.g. the beginning dates of the ice runs, ice cover and break-up at some positions etc.

3.5 Expandability Function
The system is developed by the module programming method. Each model is an independent module. When exterior conditions vary, only the related module needs to be updated.

4. Applications
The ice condition forecast expert system has been put into use on-line since 2004. The input data are the historical ice conditions and the real-time ones recorded by the four hydrologic stations, located at Shizuishan, Bayangaole, Shanhuhekou and Toudaoguai, respectively. Meanwhile, the
predicted air temperatures, provided by a weather forecast system, are also taken as the input data. The results of the ice condition forecast for the 2004-2005 ice period and the 2005-2006 ice one are as follows.

4.1 Water Temperature Forecast
As the winter is coming, the air temperature in the Inner Mongolia region begins to drop down. The change of the forecast water temperature with time at the Bayangaole Station and at the Sanhuhekou Station is shown in Figure 4 and in Figure 5, respectively, in which 11-2 represents Nov. 2 and the others are similar. The forecast period is in 15 days, i.e. the forecast water temperature is obtained 15 days earlier than the measured one. Obviously, the change of the former with time is well coincided with that of the latter, including the shape and magnitude.

![Figure 4](image)

**Figure 4.** Change of water temperature at Bayangaole station in 2004

![Figure 5](image)

**Figure 5.** Change of water temperature at Sanhuhekou station in 2004

4.2 Forecast for Beginning Dates of Ice Run, Freeze-up and Break-up
The forecast results and the observed ones in the 2004-2005 and 2005-2006 winters are listed in Table 1 and in Table 2, in which the former were obtained 15 days earlier than the latter.
Table 1. Forecast for 2004-2005 Ice Conditions

<table>
<thead>
<tr>
<th>Ice condition</th>
<th>Hydrologic station</th>
<th>Observed (Month.date)</th>
<th>Forecast (Month.date)</th>
<th>Error (Day)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beginning date of ice run</td>
<td>Shizuishan station</td>
<td>12.22</td>
<td>12.22</td>
<td>0</td>
<td>excellent</td>
</tr>
<tr>
<td></td>
<td>Bayangaole station</td>
<td>12.26</td>
<td>12.29</td>
<td>3</td>
<td>qualified</td>
</tr>
<tr>
<td></td>
<td>Sanhuhekou station</td>
<td>11.24</td>
<td>11.24</td>
<td>0</td>
<td>excellent</td>
</tr>
<tr>
<td></td>
<td>Toudaoguai station</td>
<td>11.25</td>
<td>11.25</td>
<td>0</td>
<td>excellent</td>
</tr>
</tbody>
</table>

| Beginning date of freeze-up| Shizuishan station    | 1.8                   | 1.8                   | 0           | excellent|
|                            | Bayangaole station    | 12.29                 | 12.31                 | 2           | qualified|
|                            | Sanhuhekou station    | 12.20                 | 12.21                 | 1           | excellent|
|                            | Toudaoguai station    | 12.28                 | 12.28                 | 0           | excellent|

| Beginning date of break-up| Shizuishan station    | 3.4                   | 3.2                   | 2           | qualified|
|                           | Bayangaole station    | 3.18                  | 3.19                  | 1           | excellent|
|                           | Sanhuhekou station    | 3.21                  | 3.24                  | 3           | qualified|
|                           | Toudaoguai station    | 3.19                  | 3.22                  | 3           | qualified|

Table 2. Forecast for 2005-2006 Ice Conditions

<table>
<thead>
<tr>
<th>Ice condition</th>
<th>Hydrologic station</th>
<th>Observed (Month.date)</th>
<th>Forecast (Month.date)</th>
<th>Error (Day)</th>
<th>Remarks</th>
</tr>
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<tbody>
<tr>
<td>Beginning date of ice run</td>
<td>Shizuishan station</td>
<td>12.4</td>
<td>12.10</td>
<td>6</td>
<td>not qualified</td>
</tr>
<tr>
<td></td>
<td>Bayangaole station</td>
<td>12.4</td>
<td>12.2</td>
<td>2</td>
<td>excellent</td>
</tr>
<tr>
<td></td>
<td>Sanhuhekou station</td>
<td>11.29</td>
<td>12.2</td>
<td>3</td>
<td>qualified</td>
</tr>
<tr>
<td></td>
<td>Toudaoguai station</td>
<td>11.28</td>
<td>11.24</td>
<td>4</td>
<td>qualified</td>
</tr>
</tbody>
</table>

| Beginning date of freeze-up| Shizuishan station    | 12.26                 | 12.27                 | 1           | excellent|
|                            | Bayangaole station    | 12.9                  | 12.14                 | 5           | qualified|
|                            | Sanhuhekou station    | 12.5                  | 12.2                  | 3           | qualified|
|                            | Toudaoguai station    | 12.5                  | 12.8                  | 3           | qualified|

| Beginning date of break-up| Shizuishan station    | 2.22                  | 2.12                  | 10          | not qualified|
|                           | Bayangaole station    | 3.9                   | 3.10                  | 1           | excellent|
|                           | Sanhuhekou station    | 3.19                  | 3.26                  | 7           | not qualified|
|                           | Toudaoguai station    | 3.16                  | 3.20                  | 4           | qualified|
According to *Hydrographic Information Forecast Standard* (China, 2000) listed in Table 3, 100% of the forecast results in Table 1 are qualified in the 2004-2005 winter. The errors between the forecast results and observed ones for the beginning dates of the ice runs, freeze-up and break-ups were within 3 days. In the 2005-2006 winter, 75% of the forecast results in Table 2 are qualified.

### 5 Conclusions

The Yellow River Inner Mongolia Reach is a place where the serious ice flood disasters take place frequently in winter. The accurate ice condition forecast is very important, so as to have time to take effective measures for potential disasters.

In order to improve the ice condition forecast and to meet the requirements for on-line forecast, an ANN Expert System, based on GIS-Geographic Information System-technology, has been developed. One can create, edit, and calculate models within the GIS environment. All input data and results, including time-variant information, are fully integrated with GIS database and accessible to GIS tools. Moreover, both the data of the real-time hydrology and the results of the ice condition forecast may be not only displayed by numbers, charts and alarm signals on a computer screen, but transmitted on time to every decision departments by internet.

The ANN Expert System has been verified to be successful in the two year ice condition forecast of Yellow River Inner Mongolia Reach. While the forecast period is in 15 days, 100% of the forecast results are qualified in the 2004-2005 winter. The errors between the forecast results and observed ones for the beginning dates of the ice runs, freeze-up and break-ups were within 3 days. In the 2005-2006 winter, 75% of the forecast results are qualified.

### Reference


Experience with Dispersing Ice Jams in Manitoba

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This paper describes the efforts in recent years to prevent flooding due to ice jams in southern Manitoba by dispersing river ice. The intent of the evolving program is to break or weaken ice in advance of the annual breakup in rivers where ice jams have frequently caused flooding. An amphibious excavator has been successfully deployed in a number of problem locations. In addition to using the amphibious excavator, the Province of Manitoba has experimented with cutting the ice using various methods. The intent was to promote an orderly ice breakup and to enable the amphibious excavator to break thicker ice, and to increase ice-breaking productivity. The efforts show promise and the Province of Manitoba will apply the lessons learned to future ice breaking efforts.

1. Introduction

Manitoba is located in the heart of the North American continent and has a continental climate in the south of the province, with average January temperatures of approximately -20 degrees Celsius. Ice cover forms in winter on most rivers in Manitoba. Frazil ice jams have been known to occur during the early winter freeze up period, and have caused flooding of valuable properties. Similarly, spring snowmelt brings rapidly increasing river flows and potential for development of ice jams during the breakup season. The relatively low slopes of the river channels, particularly in southern Manitoba, typically exacerbate this formation of spring ice jams.

In recent years, the implementation of hydraulically operated amphibious excavators has allowed municipal and provincial government agencies in Manitoba to reduce the risk and frequency of severe ice jams in areas that have traditionally had problems. The program of ice dispersion has also being assisted by the development of an ice-cutting program to assist the amphibious excavators by cutting trenches in the river ice in advance of the excavator, and to permit easier movement of the ice cover as the rising flow occurs during spring breakup.

This paper outlines the recent experience in southern Manitoba, and describes the program of reducing the flooding risks due to river ice formation and jams in selected problem areas of Manitoba.

2. Evolution of the Ice Clearing Strategies

An area of particular concern has been the Red River north of Winnipeg, particularly the portion from Selkirk to Lake Winnipeg, as shown on Figure 1, where the river has a very gentle slope as it nears the natural delta of the Red River where it enters Lake Winnipeg.
Serious ice jams in 1996 and in 2004 led to formation of the Red River North Ice Mitigation Committee (RRNIMC) to study possible mitigation measures to prevent or reduce ice jams in this area. The RRNIMC is composed of representatives of the Rural Municipality (R.M.) of St. Andrews, the R.M. of St. Clements, the City of Selkirk, and the Manitoba departments of Water Stewardship and Conservation. The RRNIMC sponsored a workshop in Winnipeg on March 8,
2005 to study various possibilities for ice jam mitigation. This involved both government representatives as well as outside speakers with experience in river ice management. It soon became apparent that the causes and potential solutions to the ice jam problem in the Red River were complex. Data on ice thicknesses obtained by Manitoba Water Stewardship showed that there was no obvious relationship between ice thickness and the occurrence of ice jams. Ice quality was considered a factor, but weakening of ice by dusting or insulating was not considered to be environmentally acceptable. One of the key options identified in the workshop was breaking the river ice by some means.

The Province of Manitoba subsequently invited a firm from Quebec to demonstrate their machine, called the “Amphibex”. The Amphibex is a hydraulically operated amphibious excavating machine which can ‘walk’ overland to a desired location on a river and then float while breaking ice with its hydraulically operated backhoe. It is capable of breaking strong ice up to 0.6 metres in thickness. The manufacturer, Normrock, imported a machine to clear an ice jam that had formed on the Red River in Winnipeg at the Redwood Bridge, where ice jams have historically been a problem. Photos 1 to 3 show the Amphibex and its deployment in April 2005. It was successful in clearing out the downstream end of an ice jam that was lodged against the Redwood Bridge, and effectively allowed the jammed ice to release downstream before excessive flooding could develop upstream of the accumulating ice jam.

Photo 1. Amphibex – mobilizing onto the Red River, 2005
Photo 2. Amphibex – dispersing an ice jam at the Redwood Bridge in Winnipeg, April 4, 2005

Photo 3. Amphibex – 2005
This demonstration, as well as good reports from the previous exploits of the Amphibex in other similar problem locations in Canada, led the decision to purchase an ice excavator for exclusive use to combat ice jams in Manitoba. The Amphibex was purchased from Normrock and was cost-shared by Provincial and Municipal governments. It is owned and operated by a Corporation consisting of the Rural Municipalities of St. Andrews and St. Clements, the City of Selkirk, and the Province of Manitoba’s Water Stewardship Department. The supplier helped to train several local personnel to operate the machine.

3. Deployment of the Amphibex to Date

Figure 1 shows a map of southern Manitoba and the key areas where ice-clearing efforts have been concentrated to date. In addition to the first application in Winnipeg, the Amphibex has been increasingly used in 2006, 2007 and 2008 to prevent or reduce the chance of serious spring ice jams. The key locations were:

- Cutting and breaking of ice in the critical areas from Selkirk, northwards to Lake Winnipeg
- Breaking winter ice at the Portage Diversion outlet near Lake Manitoba
- Breaking winter ice on the Whitemud River near the town of Westbourne on Lake Manitoba
- Breaking winter ice on the Saskatchewan River near Ralls Island at The Pas
- Breakup of a frazil ice jam on the Waterhen River to reduce flooding in the community of Waterhen, Manitoba.
- Clearing ice to free ice-locked ferry at Norway House, Manitoba. The ferry is the only means of land access to the community of 5000 and needed to be freed in order that essential items could be trucked in to the community.

The cost of operation of the Amphibex varies, but a reasonable estimate of the average cost is approximately $450/hour, including fuel, operator wages, maintenance, and establishment of a sinking fund for future replacement and upgrades.

Operation of the Amphibex requires at least a few days notice to transport the machine to its desired location. For safety purposes an amphibious vehicle is kept within sight, and in radio contact, of any ice breaking or ice cutting operations. In addition, all operators wear flotation suits.

Experience has shown that the Amphibex is an appropriate and effective tool for the prevention and dissipation of ice jams in Manitoba. However, it cannot perform under all conditions. In some situations, such as a major ice jam at Selkirk in April, 2007, it can be too dangerous for the Amphibex to venture onto the ice. Sudden movement of the ice jam could crush the machine and endanger the lives of the operators.

In addition, the immense forces of the ice on the machine have taken their toll on the equipment and have resulted in continuous breakdowns. Selective deployment of the machine is required to avoid undue risks and avoid major damage to the equipment. There have also been modifications to the arms and to the hydraulic system of the icebreaker.
4. Improvements

The limitations of Manitoba’s first Amphibex have become evident in the first few years of usage. One weakness has been that the Amphibex has difficulty in effectively and consistently breaking ice that is in excess of 0.6 metres in thickness. As a result, Manitoba continues to actively pursue the following courses of action: modifications to the existing Amphibex, and the purchase of a second stronger Amphibex, and the development and implementation of an ice-cutting program. It is expected that these actions will increase the thickness of the ice that can be broken and increase the rate at which it can be broken.

Manitoba has initiated experimentation with innovations to improve the overall program. Improvements to the first Amphibex have been undertaken and included modifications and redesign / manufacture of the joints and structural arrangements of the hydraulically activated arm to make it more robust and minimize downtime due to breakages. Improvements to the hydraulic systems were also implemented. The total cost of these improvements was $200,000.

Other means to improve the performance have also been pursued. While the Amphibex can break ice of 0.6-metre thickness, it is with difficulty, and the speed of its advancement along the river is severely impeded under such conditions. As a result, the Province of Manitoba has undertaken an experimental program to test various means of cutting trenches in the ice to make the subsequent action of the Amphibex easier and the overall process more efficient and cost effective.

Three methods have been tested to date:

- A proprietary machine called the “Ditch Witch”, shown in Photo 4
- A circular saw mounted on a tracked Cat as shown in Photo 5
- A machine developed locally using the concept of a router blade, as shown in Photo 6.

The tests were done to evaluate each method with regards to speed, dependability, and weight of machine. All tests have been undertaken in the Red River near Selkirk.
Photo 4. Ditch Witch – 30 hp, 4WD, cuts ice at 6 m per minute. Relatively light weight, and dependable, but slow.

Photo 5. Circular saw – approximately 90hp, track drive. This machine is faster, being able to cut ice at 15 m per minute and is dependable. It will cut through almost anything. However it is relatively heavy, and you drive over the ice that has been cut.
Photo 6. Rotary cutter (or router) – Runs off of standard PTO mounted on a 34 hp tractor. It is able to cut ice at 25 m per minute. It is light and fast, but dependability was an unknown. Later, it demonstrated dependability under field operations, but the blade gets dull when cutting thru ice under bridges due to gravel, or cutting thru old fishing shack debris.

5. Conclusions

The recent efforts in Manitoba to disperse ice have been generally successful although the effectiveness has not yet been proven under the most difficult situations. There has been success in dissipating frazil ice jams in the autumn and in breaking up small ice jams during the spring. The methodology appears to have prevented ice jams on the Red River in 2008 although it is not clear whether serious jams would have formed without these activities due to low river flows. Minimization of potential flood damages has been possible in many areas, and Manitoba officials are optimistic that these mechanical ice mitigation tools will prove to be successful. The program will be expanded in the future and improved as the technology is improved through testing and field experience.
River ice jams can have significant impacts on water resource development in cold regions. Due to difficulties in observing the phenomena in the field, our understanding on the dynamics of ice jam formation is very limited. Numerical models, such as HEC-RAS, have been used to determine the equilibrium ice jam configuration and its backwater effect. These models were developed based on the static equilibrium of the floating surface ice mass accumulation in the jam. They cannot be used to study ice dynamics. In this paper, the DynaRICE model is validated with a set of field observed ice jam data on the Thames River (Beltaos 1988). The model is then used to study the ice jam formation dynamics in a natural river to illustrate the dynamics of ice transport and jamming processes. The study showed that the ice congestion is a result of the convergence of ice mass produced by a complex interaction of flow and ice dynamics.
1. Introduction

River ice jams can have significant impacts on water resource development in cold regions. Ice jam formation and release phenomena are very dynamic. Numerical models, e.g. HEC-RAS, based on the classical static ice jam theories (Pariset and Hausser, 1961; Beltaos, 1983) have been used to determine the impacts of ice jams and to develop ice jam control measures. Since the dynamics of ice motion were not considered, they cannot be used to study ice jam dynamics. Shen et al. (2000) developed a dynamic ice transport model DynaRICE. The model has been validated with analytical solutions, and applied to several field problems (Shen et al., 2000). Liu and Shen (2004) used the model to study the dynamics of ice jam release.

Due to the lack of detailed field measurement of ice jam thickness profiles, field verifications of the model simulation results were limited to the time and location of jam formation and the associated water level changes. No direct comparison with observed ice jam thickness was made. In this study, the DynaRICE model was validated with a set of ice jam data observed in the Thames River, Ontario (Beltaos, 1988). The model was then used to illustrate the dynamics of jam formation in a river reach.

2. Model Formulation

The formulation of the DynaRICE model was presented in Shen et al. (2000). The model is a two-dimensional ice dynamics model for analyzing dynamic transport of surface ice in rivers and lakes. The model simulates the coupled dynamics of ice motion and water flow, including the flow through and under the surface ice rubble. The momentum equation for ice dynamics is

\[ M \frac{D\vec{V}}{Dt} = \vec{R} + \vec{F}_a + \vec{F}_w + \vec{G} \]  

in which, \( x, y \) and \( t \) = space and time variables; \( M = \rho_i N t_i \) = ice mass per unit area; \( \rho_i, N, t_i \) = density, area concentration, and thickness of ice, respectively; \( \vec{V} = u\vec{i} + v\vec{j} \), ice velocity; \( D/ Dt = \) material derivative; \( \vec{F}_a = \) wind drag at the air-ice interface, \( \rho_a = \) density of air; \( \vec{F}_w = \) water drag at the ice-water interface; \( \vec{G} = \) gravitational force due to the water surface slope; and \( \vec{R} = \) internal ice resistance = \( \int \left[ \frac{\sigma_{xx}}{a} (\sigma_{xx}N t_i) + \frac{\sigma_{xy}}{a} (\sigma_{xy}N t_i) \right] + \int \left[ \frac{\sigma_{yy}}{a} (\sigma_{yy}N t_i) + \frac{\sigma_{yy}}{a} (\sigma_{yy}N t_i) \right] \). A constitutive law is required to describe the ice internal stress. In this study, the internal stress is described by a viscous-plastic constitutive law, \( \sigma_{ij} = 2\nu \dot{\epsilon}_{ij} + (\zeta - \nu) \dot{\epsilon}_k \delta_{ij} - P \delta_{ij} / 2 \), in which, \( \zeta, \nu = \) nonlinear bulk and shear viscosity defined as \( \zeta = P/2\Delta \) and \( \nu = \zeta / \epsilon^2 \), with \( \Delta^2 = D_i^2 + \left( D_{ii} / \epsilon \right)^2 ; D_i, D_{ii} \) are first and second invariant strain rates, respectively; \( \dot{\epsilon}_x = \dot{\epsilon}_y = \dot{\epsilon}_x \), \( \dot{\epsilon}_y = \dot{\epsilon}_y \), \( \dot{\epsilon}_y = \dot{\epsilon}_y / \epsilon ; \) and \( \dot{\epsilon}_y = (\dot{\epsilon}_y / \dot{\epsilon}_x + \dot{\epsilon}_y / \dot{\epsilon}_y) / 2 \). The principal axes ratio of the elliptical yield curve \( \epsilon = 2 \) (Hibler 1979). The pressure term is formulated as:

\[ P = \tan^2 \left( \frac{\pi}{4} \pm \frac{\phi}{2} \right) \left( 1 - \frac{\rho_i}{\rho} \right) \frac{\rho_i g l_i}{2} \left( \frac{N}{N_{\max}} \right) \]
in which \( \phi = \) internal friction angle of ice, 46\(^\circ\); \( N_{\text{max}} = \) maximum allowable ice concentration, 0.6; and \( j = 15, \) an empirical constant. The + and - signs are for convergent and divergent states of ice flow, respectively.

The DynaRICE model has been successfully applied to several river ice jam studies, e.g. Liu and Shen (1998), Lu et al. (1999), and Shen and Liu (2003). Table 1 summarizes the representative dimensions of these river reaches. Table 2 presents the magnitude of terms in Eq. 1 to provide information on the relative importance of different terms on ice dynamics.

**Table 1.** Representative dimensions of simulation cases

<table>
<thead>
<tr>
<th>River</th>
<th>Upper Niagara River</th>
<th>Missouri River</th>
<th>Shokotsu River</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>1000 m</td>
<td>300 m</td>
<td>50 m</td>
</tr>
<tr>
<td>Depth</td>
<td>3 ~ 10 m</td>
<td>3 ~ 5 m</td>
<td>2 m</td>
</tr>
<tr>
<td>Domain length</td>
<td>35 km</td>
<td>10 km</td>
<td>5 km</td>
</tr>
</tbody>
</table>

**Table 2.** Magnitude of terms in Eq. [1] for river ice dynamics

<table>
<thead>
<tr>
<th>Term</th>
<th>Magnitude</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M \frac{DV}{Dt} )</td>
<td>1.1 ~ 550 N</td>
<td>Assuming ice thickness of 0.2 ~1.0 m, and ice velocity changes from 1 m/s to zero in 1 to 100 sec.</td>
</tr>
<tr>
<td>( F_w )</td>
<td>1 ~ 30 N</td>
<td>Assuming ( C_w = 0.01 ~ 0.03, ) and water velocity = 0.1 m/s</td>
</tr>
<tr>
<td>( F_a )</td>
<td>0.2 N</td>
<td>Assuming ( C_a = 0.0018, ) and a strong wind of ( V_a = 10 ) m/s</td>
</tr>
<tr>
<td>( G )</td>
<td>1.1 ~ 5.5 N</td>
<td>Assuming water surface slope = 0.01 ~ 0.0001, and ice thickness = 0.2 ~1.0 m</td>
</tr>
<tr>
<td>( P )</td>
<td>500 ~ 2500 N</td>
<td>Assuming ice thickness = 0.2 ~1.0 m, and ( N = N_{\text{max}} )</td>
</tr>
</tbody>
</table>

In the DynaRICE model, the hydrodynamics was simulated with a finite element method using a lumping technique and leap-frog time integration scheme. A Lagrangian smoothed particle hydrodynamics (SPH) method (Lucy, 1977; and Gingold and Monaghan, 1977) is used to simulate the transport and dynamics of surface ice under the actions of wind and current forces. The SPH method was chosen because of its flexibility in simulating the highly deformable complex ice processes. Recently, the hydrodynamic component of the model has been replaced by a new finite element model based on the streamline upwind technique, which is capable of simulating high velocity transitional flows and wet-dry bed conditions (Liu and Shen, 2003). The constitutive law for ice resistance has been improved with a viscoelastic-plastic model (Ji et al. 2004).

3. Model Validation

The lack of ice jam thickness data limits the information available for directly validating ice jam models. The data gathered during the January 1986 breakup on the Thames River by Beltaos (1988) provided a needed data set for testing the accuracy of the numerical model. This set of data was used to check the validity of the DynaRICE model.
The study reach extends from a Water Survey of Canada gauge in Chatham to the Golf Course, as shown in Figure 1. During the January 1986 mid-winter breakup, an ice jam occurred in the study reach. The length of the jam was about 10 km, as show in Figure 2. The process of this breakup ice jam formation is summarized in Table 3, which shows two major events were observed. Simulation was carried out according to these events. In the simulation, the estimated water discharge before and after the first jam formation were 280 m³/s and 290 m³/s, respectively. Initially, the ice cover upstream of km 32.04 was set to break and move downstream. The ice supply at the upstream boundary was specified by a combination of an ice floe thickness of 25 cm and an ice concentration between 0.3 and 0.35. The observed water surface elevation at Chatham Gauge located at km 30.72 was used as the downstream boundary condition. Manning’s coefficient for the riverbed was set at 0.025. Manning’s coefficient for the underside of the ice cover and jam was calibrated with the observed staging during jam formation. This coefficient varied linearly with the jam thickness from a minimum value of 0.03 to a maximum value of 0.11.

<table>
<thead>
<tr>
<th>Time</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>10:30am, January 22, 1986.</td>
<td>Ice cover thickness varied between 20 and 30 cm. Because of rainfall, some ice cover was set in motion.</td>
</tr>
<tr>
<td>10:30am, January 22, 1986.</td>
<td>A 10-km long jam formed at the bend designated as 35.82 km.</td>
</tr>
<tr>
<td>10:30am ~ 6:00pm, January 22, 1986</td>
<td>Small jam remained stable</td>
</tr>
<tr>
<td>6:00pm ~ 8:00pm, January 22, 1986.</td>
<td>The 10-km long jam released. Ice cover downstream began to break up.</td>
</tr>
<tr>
<td>7:30pm, January 22, 1986</td>
<td>Jam began to form at the bend at km 32.04.</td>
</tr>
<tr>
<td>10:30am, January 23, 1986</td>
<td>Final jam formed at the leading edge of the unbroken ice cover at km 32.04.</td>
</tr>
</tbody>
</table>
Figure 1. The Lower Thames River near Chatham, Ontario.

Figure 2. Conditions in the Thames River, January 23 A.M., 1986 (Beltaos 1988)

Figure 3 shows the comparison of simulated profiles of ice jam and water level at 11:30 am, January 23, 1986, with the observed data. The simulated results compared well with the field data. In the vicinity of the ice jam toe, the simulation results are slightly different from the observed data. This could be due to the field measurements of the ice jam toe being taken in late February, almost a month after the jam formation date of Jan. 23rd. The jam toe configuration might have changed somewhat from the jam formation time to the measurement time, even though the jam remained in place during this period.
4. Ice Jam Formation Dynamics

It is impractical, if not impossible to observe ice jam dynamics in the field. This section uses the DynaRICE model to simulate and illustrate the formation process of a breakup jam in a natural river reach using a reach of the Porcupine River, Yukon Territory, as shown in Figure 4 (Jasek, 2003). In the simulation, the entire domain was initially covered by a layer of ice rubble 0.4 m thick with a concentration of 0.6. The upstream hydraulic boundary condition was a constant water discharge of 436 m$^3$/s. The water level at the downstream boundary was set at a constant level of 239 m. Surface ice was supplied from the upstream boundary during the entire simulation period. The incoming surface ice concentration and thickness were 0.5 and 0.4 m, respectively. Manning’s coefficient of the surface ice layer and ice jam varied with the ice thickness from 0.03 for a single layer accumulation to a maximum value of 0.06. The initial water depth is shown in Figure 4. The areas with zero or negative depth represent dry bed areas covered with ice rubble from the broken cover.

Figure 3. Comparison of simulated jam and water surface profiles with field data.
The simulation results showed that at hour 0:15, some of the ice rubble resting on the berms started to move into the main channel, while some of the ice rubble in the main channel began to spill into the berm areas. This was due to the rise of water level upstream of cross section 4. This water level increase was caused by the backwater effect of the ice congestion initiated in the
narrow main channel in the vicinity of cross section 4. This process continued for several hours. More ice rubble on the berm close to cross section 25 floated and drifted to the congested area around cross section 4, causing the congestion to extend upstream, and the thickness of the ice accumulation increased. By hour 9:00, as the congestion at cross section 4 continued to grow, the increase in channel storage reduced the water flow to downstream. This led to the development of ice congestion in the vicinity of cross section 10 downstream. By hour 35:00 this jam was fully developed and stopped moving. The jam at cross section 4 was fully stopped about 3:45 hours later. Figures 5 and 6 show the ice conditions at hour 44:00 after the formation of these jams.

To further analyze the jam formation process, ice conditions in the selected control areas at the jam toes at cross sections 4 and 10, shown in Figure 7, were examined. Figure 8 shows two partial jam releases at cross section 4 due to the increase in streamwise forces on the jam. These releases accelerated the jam formation at cross section 10. After the ice jam formed at cross section 10, the jam at cross section 4 grew and stabilized with the help of the backwater caused by the growth and upstream extension of the jam at cross section 10. Figure 8 shows variations of ice volume and ice velocity in the control area of the jam toe at cross section 10. This figure shows that in the final stage of the jam formation the ice velocity decreased rapidly. This was caused by the accelerated increase of internal ice resistance resulting from the increase in ice concentration and ice thickness when ice converged in the jamming area, as described by Eqs. 1 and 2.

**Figure 6.** Ice thickness distribution and longitudinal profiles of ice, water surface, and velocity in the reach at hour 44:00.
Figure 7. Control areas in the jam toes near cross sections 10 and 4

Figure 8. Variations of ice volume in control areas at cross sections 4 and 10.

Figure 9. Variations of ice volume and ice velocity in the control areas at cross section 10.
5. Conclusions
A two-dimensional coupled flow and ice dynamic model was used to study the ice jam dynamics. The model was first validated with a set of measured ice jam field data. The model was then applied to a river reach to illustrate the dynamics of ice jam formation process. This study showed that the formation of an ice jam is a result of complex interactions of river flow and ice motion. The key factor for ice jam formation is the congestion of ice due to the convergence of ice flow.

Acknowledgement
The writers would like to thank S. Beltaos for providing the field data on Thames River ice jam, and to M. Jasek for providing the bathymetry data of Porcupine River.

References


Effects of Unsteadiness and Ice Motion on River Ice Jam Profiles

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This paper utilizes River1D model’s new ice jam component to explore the river ice jam dynamics under various flow conditions. The model solves the 1D conservation of mass and momentum equations for water flow and ice motion using finite element method in an uncoupled sequence. A one-seventh power law formulation is used to describe the internal resistance and strain rate relationship of the moving ice accumulation. The model is applied to a series of experimental ice jam consolidation events. Model results are compared with those predicted by a steady ice jam profile model with steady gradually varied flow approximation and Mohr-Coulomb failure criteria, to evaluate the extent in which unsteadiness and ice motion can affect jam-thickness profile. Ice jams collapsed by various inflow hydrographs are simulated, and the influence of hydrograph shapes on the jam-thickness is examined.
1. Introduction

The high water levels associated with breakup ice jams in rivers often pose a significant flood threat to public safety. A number of computational tools are available for predicting water surface and ice profiles associated with river ice jam occurrence; most are based on static ice jam stability theory (e.g. Pariset et al. 1966, Uzuner and Kennedy, 1976) combined with a steady gradually varied flow approximation (e.g. RIVJAM: Beltaos and Wong 1986, ICEJAM: Flato and Gerard 1986, and HEC-RAS: Daly and Vuyovich 2003). These models can provide information on the expected thickness and water surface profiles of stable ice jams. However, inherent in their use are the assumptions that stream flow is steady and jam strength is deformation-independent. Even if one neglects the obvious unsteady aspects of ice jam formation and focuses on the final ice jam profile as the characteristic of interest, the question remains as to whether the final “stable” ice jam profile predicted by such models is applicable under dynamic ambient flow conditions, such as might be expected during hydro-peaking events on regulated rivers.

Steady ice jam profile models have been extensively tested using field data on water level and/or ice thickness data obtained on stable ice jams (e.g. Beltaos 1993, Healy and Hicks 1999) and credible predictions of ice jam configuration have been achieved by choosing appropriate model parameters. Unfortunately, it is difficult to assess the validity of steady ice jam profile models for ice jams formed under unsteady flow conditions, because of the logistical difficulties associated with measuring discharge and ice accumulation thickness variations during ice jam formation in the field. Fortunately, some success has been achieved in measuring such processes in the laboratory. Zufelt (1990, 1992) qualitatively observed that the final ice jam profile was influenced by unsteady flow conditions, noting that the consolidation process could be interrupted and resumed by discharge fluctuations. Healy and Hicks (2006, 2007) measured ice jam formation and shoving events under steady and unsteady ambient flows. They found that reasonable profiles describing the final ice jam configurations could be obtained using steady ice jam profile models for ice jams formed under both steady carrier discharge conditions, and for ice jams consolidated by rapid discharge increases. These two flow conditions bracket the practical range of flow scenarios; with flow conditions during actual ice jam formation events are typically somewhere in between.

This paper utilizes the University of Alberta’s River1D model’s new ice jam component to further explore the applicability of steady ice jam profile models, by studying the effects of unsteady flow on ice jam profile shape. This new version of the River1D model employs a purely Eulerian frame of reference for both the hydrodynamic and ice dynamics components. A new one-seventh power law formulation for the constitutive law (She et al., submitted) is used for determining the internal ice resistance of the moving ice accumulation. This new model has already been validated with analytical and experimental test cases of ice jam consolidation (She et al., submitted). Here, profiles of ice jams formed during three experimental consolidation events are modeled using River1D’s new ice jam component model, and compared to calculated profiles obtained using the RIVJAM model formulation (Beltaos and Wong 1986). Comparison of the model results, together with the measured jam thickness, provides some ideas on when the unsteadiness and ice motion effects should be taken into account for predicting flood potential of
river ice jams. The influences of inflow hydrographs (peaking time and peak duration) on the final jam configuration are also examined.

2. Computational Models

2.1 River1D ice jam model

The River1D ice jam model solves the mass and momentum conservation equations for the water and for the ice in an uncoupled sequence, using the Characteristic-Dissipative-Galerkin (CDG) finite element method (Hicks and Steffler 1992). Assuming a floating ice accumulation, the mass and momentum conservation equation for water flow can be written as:

\[
\frac{\partial A_w}{\partial t} + \frac{\partial Q_w}{\partial x} = -\frac{\partial}{\partial t} \left[ (1 - N) \frac{s_i A_i}{N} \right] - \frac{\partial Q_u}{\partial x} \tag{1}
\]

\[
\frac{\partial Q_w}{\partial t} + \frac{\partial (Q_w V_w)}{\partial x} = gA_w \frac{\partial H_w}{\partial x}
\]

\[
gA_w \left( S_o - \frac{\partial s_i}{\partial x} \right) = \left[ \frac{n_b^2 V_w (B + 2H_w)}{R_b^{1/3}} + \frac{n_i^2 (V_w - V_s) (V_w - V_i) NB}{R_i^{1/3}} \right]
\]

where: \( A_w \) and \( Q_w \) are the area and discharge of flow under the ice layer; \( N \) is the surface concentration of ice; \( s_i \) is the specific gravity of ice; \( A_i \) the ice volume per unit length of channel, (defined as \( A_i = N B t_i \)); \( B \) is the channel width; and \( t_i \) is the ice thickness. \( V_w \) and \( H_w \) are the velocity and the depth of flow below the ice layer, respectively; \( S_o \) is the bed slope; \( n_b \) and \( n_i \) are the Manning’s roughness coefficients for the bed and the ice, respectively; \( R_b \) and \( R_i \) are the hydraulic radius of the bed-affected, and ice-affected areas, respectively; and \( Q_u \) is the water seepage discharge within the ice layer defined as:

\[
Q_u = \begin{cases} 
V_i (1 - N) A_j & \text{if } V_i > V_s \\
V_s A_j & \text{otherwise} 
\end{cases} \tag{3}
\]

Here, \( V_i \) is the ice velocity and \( V_s \) is velocity of the seepage flow through the interstices of the ice layer, and is defined as \( V_s = \lambda S_w \), in which \( \lambda \) is a coefficient describing the flow through the voids in the ice layer; \( S_w \) is the water surface slope; and \( A_j \) is the submerged cross sectional area of the ice layer.

The mass and momentum conservation equations for the floating ice layer are:

\[
\frac{\partial A_i}{\partial t} + \frac{\partial (V_i A_i)}{\partial x} = 0 \tag{4}
\]
in which: $\rho_i$ is the density of ice; $K_{xy}$ is a lateral thrust coefficient; and $\phi$ is the internal friction angle. Here, $\sigma$ is the internal resistance of the ice accumulation, which is determined using a one-seventh power law constitutive relationship (She et al., submitted):

$$\sigma = (T)^{\frac{1}{7}} \left( \frac{t_i}{t_{\text{eq}}} \right)^j P(\dot{\varepsilon})^{\frac{1}{7}} - P$$ \hspace{1cm} [6]

in which: $T$ is a time scale parameter; $t_{\text{eq}}$ is the maximum thickness of the ice accumulation under a specific stress; $t_i/t_{\text{eq}}$ indicates how close the ice is to the final static state; $j$ is an empirical constant; $\dot{\varepsilon}$ is the strain rate; and the pressure term, $P$, is formulated as:

$$P = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) (1 - s_i) \rho_i g t_i$$ \hspace{1cm} [7]

She et al. (submitted) provide full details of this new model’s formulation and validation.

### 2.2 RIVJAM ice jam model

RIVJAM (Beltaos and Wong 1986), ICEJAM (Flato and Gerard 1986) and HEC-RAS (Daly and Vuyovich 2003) are probably the three most widely known steady flow ice jam profile models. They all solve an ice jam stability equation in conjunction with a steady gradually varied flow (GVF) equation. In this investigation, the RIVJAM formulation (Beltaos and Wong 1986) was selected for comparison, due to its capability of handling seepage flow through the voids of the ice accumulation.

In the RIVJAM formulation, two ordinary differential equations are solved using the Runge-Kutta method.

$$\frac{dt_i}{dx} = \frac{s_i \rho g}{2 \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) y_e} \left[ \frac{f_i}{2 f_o} \frac{g_H}{4 H_{w} s_i t_i} \left[ \frac{Q}{A_w} \sqrt{\frac{4 g A_w}{f_o B + \lambda A_j}} \right]^2 + \frac{f_o g_H}{2 f_o} \right]$$

$$- \frac{\tan \phi}{\tan^2 \left( 45^\circ + \frac{\phi}{2} \right) B} t_i$$ \hspace{1cm} [8]
\[ \frac{dH_w}{dx} = S_o - \frac{ds_i t_i}{dx} - \left[ \frac{Q}{A_w\sqrt{\frac{4gA_w}{f_o B} + \lambda A_j}} \right]^2 \]  

[9]

Here, \( \gamma_e \) is the effective unit weight of water, defined as \( \gamma_e = (1 - s_i)(1 - p)\rho g/2 \), where \( p \) is the porosity of the ice layer; \( q_w \) is the discharge under the ice layer per unit width; \( Q \) is the total discharge through and under the ice layer; and \( f_o \) and \( f_i \) are the composite and ice Darcy-Weisbach friction factor, respectively.

### 3. Analysis of an Ice Jam Consolidated by Rapid Flow Increase

Experimental ice jam consolidation events (Healy 2006, Healy and Hicks, 2007) are chosen for exploring the applicability of static ice jam profile models due to the advantage of known ice jam strength parameters (internal friction angle, porosity) and the carrier discharge. The experiments were conducted in a 32m-long, 1.22m-wide rectangular flume set to a slope of 0.00164, supplied with discharges ranging from 33 to 63L/s. Water levels at the downstream end were controlled by a weir and guide vanes. A 1.9cm-thick, 1.22m-long plywood sheet was positioned 24.5m downstream of the head tank to simulate a free-floating intact ice cover, and a wire screen was fixed to its upstream edge to facilitate initiation of an ice accumulation. Manning’s \( n \) for the flume ranged from 0.020 to 0.025 under open water conditions. The specific gravity of the model ice was 0.92 and the angle of repose was found to be 46°. First, an ice accumulation was allowed to form and stabilize in the flume at a low carrier discharge. This initial ice accumulation was then collapsed by a rapid increase in discharge, shoving to a much thicker ice jam.

Manning’s \( n \) of the ice was calibrated in the River1D ice jam model using:

\[ n_i = n_{io}\left(\frac{t_i}{t_{io}}\right)^{1/3} \]  

[10]

in which \( n_{io} \) is the Manning’s coefficient for a single-layer of ice \( (t_{io} = 1cm) \); a value of \( n_{io} = 0.020 \) was used in the model. Manning’s \( n \) for the ice ranged from 0.020 to around 0.050 for all the experimental tests. The same formulation of ice roughness was also used in the RIVJAM computations.

Although the RIVJAM computations can start at any point within an ice jam where the ice thickness and water level are known, it is important to note that the starting point must be within the reach where the ice jam stability equation applies. In the experimental test runs, ice jams were initiated by a wire mesh, providing extra resistance that is not included in the jam stability equation. Therefore, the computations should not start at a point that is too close to the wire mesh. As the ice volume is known for each experimental event, an appropriate starting point could be determined by trial and error, being chosen such that the computations give the correct ice volume.
The simulated results of three experimental ice jam consolidation events are presented in Figures 1-3, representing relatively high (85%), medium (55%) and low (24%) discharge increases collapsing the initial ice accumulation. Subfigure (a) presents the measured initial ice accumulation; subfigure (b) presents the calculated final ice jam profile using the two models, along with the measurements. In all three cases, the RIVJAM profiles were calculated using the final discharge in the experiment.

From the figures it can be seen that the River1D ice jam model consistently provides a good match to the measured final ice jam profile in all the three events. Only one slight difference occurs and that is at the upstream end of the accumulation, where the measured ice jam shows a “hook-like” shape. Healy and Hicks (2007) observed that hydraulic thickening dominated in this zone for all experiments (which is also consistent with field observations). This local hydraulic thickening effect is not considered in the current version of the River1D ice jam model.

The RIVJAM formulation was also found to reasonably predict the profile of ice jams consolidated by high and medium increase in discharge (Figures 1b and 2b), again with the exception of the hydraulic thickening effect at the head of the jam (for the same reasons). However, it was found that the RIVJAM formulation could not match the profile of the ice jam consolidated by lowest increase in discharge (Figure 3b). This is an interesting result, given that the discharge increase in this case was relatively low compared to the other two events; therefore, flow unsteadiness is unlikely to be the reason for this discrepancy. This would suggest that the means by which the ice internal resistance is quantified is the issue. This hypothesis was explored next.

Steady ice jam profile models use Rankine’s passive pressure and the Mohr-Coulomb failure criterion to quantify the internal resistance of the ice accumulation, thus neglecting the effects of ice motion on the internal resistance of the ice accumulation. In contrast, the River1D ice jam model uses a one-seventh power law constitutive relationship which relates the internal resistance of the ice accumulation to the ice motion (She et al. submitted). To facilitate a direct comparison between the two approaches for the unsteady flow case, the River1D ice jam model was modified to include Rankine’s passive pressure and the Mohr-Coulomb criterion as an option. This provided three modeling alternatives for simulating the final ice jam profile created by the low (24%) increase in discharge: (1) a steady model employing the Mohr-Coulomb criteria (i.e. the RIVJAM formulation); (2) an unsteady model employing the Mohr-Coulomb criteria; and (3) an unsteady model with a one-seventh power law constitutive relationship (i.e. the new River1D ice jam model). The final ice jam profiles obtained using these three modeling approaches are shown in Figure 4, along with the measured jam-thickness profile for comparison. As can be seen from the figure, the unsteady model employing the Mohr-Coulomb criterion gives very similar results to those obtained with the (steady) RIVJAM formulation; and entirely different from the River1D ice jam model results. It appears that including the effects of ice motion on the internal strength of the ice accumulation (e.g. as in the new River1D ice jam model) provides a better representation of the observed ice jam thickness profile, for this case where a small discharge increase produced relatively small deformations. The fact that the steady and unsteady models using the Mohr-Coulomb criterion produce very close results, confirms the hypothesis that flow unsteadiness is not the issue here.
4. Analysis of Effects of Inflow Hydrograph Shape on Final Ice Jam Profile

The question remains as to why it is important to include the effects of ice motion on the internal strength of the consolidating ice accumulation in some cases and not others. The fact that it appears to be more important for mild increases in discharge, combined with the observed shape of the final ice thickness profile for this case, suggests that the discharge change is sufficient to initiate a consolidation but is inadequate to complete it. This is consistent with the qualitative observations of Zufelt (1990, 1992) as well. If true, then it is not just the magnitude of the increase in discharge that should be important, but possibly also the gradient and duration of the flow increase, as well. If so, this would be significant from a practical perspective, since the ambient flow would be expected to involve a slow increase in discharge due to snowmelt runoff caused by gradual warming, or a peak ambient flow of relatively short duration if caused by upstream ice jam releases or hydro-power operations. To investigate these effects, several runs were conducted with the River1D ice jam model to evaluate the impact of inflow hydrograph shape on the final stable ice jam profile. The experimental test case involving the high (85%) increase in discharge was used for this analysis (i.e. discharge increase from 33.5 to 61.9L/s).

The effect of varying the time to peak was examined first, and Figure 5 presents the inflow hydrographs input to the River1D ice model for this series of tests. In the figure, \( t_p \) is the time for the discharge to increase from 33.5 to 61.9L/s; values of 13 seconds (i.e. the same as in the experiment), 1 minute, 3 minutes, and 6 minutes were tried. It can be seen from the model results presented in Figure 6 that for ice jams formed under the four inflow conditions, the simulated final thickness profiles are exactly the same. Moreover, they all agree well with the thickness profile computed using the RIVJAM formulation (again calculated for the final discharge of 61.9L/s).

The effect of varying the duration of peak discharge was examined next. Keeping the same inflow rise rate (from 33.5 to 61.9L/s in 13 seconds, as in the experiment), four different inflow hydrographs with 0, 1, 5, and infinite minutes of peak duration, as depicted in Figure 7, were input to the River1D ice jam model. Figure 8 illustrates the results, which suggest that the duration of the peak inflow appears to be a very important factor influencing the final jam configuration. Specifically, the longer the peak flow is sustained, the shorter and thicker the final ice jam configuration. Comparing these result to the ice jam thickness profile calculated using the RIVJAM formulation (again using the peak discharge), also shown in Figure 8, it is seen that the resulting profile is consistent only with sustained flow changes.

5. Summary and Conclusions

The standard in ice jam profile computation that has evolved in the past few decades involves a number of approximations, the two foremost being the applicability of a steady flow approximation, and the suitability of the Mohr-Coulomb failure criterion to quantify the internal resistance of the ice accumulation. The development of new unsteady ice jam formation models, with consideration of the effects of ice motion on the strength of the developing accumulation raise the potential for examining not only the ultimate (stable) ice jam profiles, but also for studying the influences of unsteady flow on the evolution of an ice jam.
In this study, the new ice jam component in the River1D one-dimensional unsteady ice dynamics model has facilitated an investigation of ice jam dynamics under unsteady ambient flow conditions, enabling an assessment of the validity of conventional steady ice jam profile models. It has also facilitated an investigation of the effects of varying rates and durations of discharge increase on final ice jam profile shape. Based on these investigations, it has been found that the unsteadiness caused by very rapid increase in discharge (i.e. the passage of a highly dynamic wave) does not appear to have a significant effect on the ultimate stable jam-thickness profile for large discharge increases. However, for ice accumulations experiencing relatively small discharge increases and deformations, steady ice jam profile calculation may underestimate the ultimate ice jam thickness profile. Further research, both experimental and numerical, is needed to explore the bounds of relevance and ranges of applicability of these steady flow approximations.

The new model has also been used to examine the importance of the rate and duration of discharge increases on ultimate (stable) ice jam thickness profiles. Preliminary results suggest that the ultimate ice jam profile is relatively insensitive to wave steepness (i.e. the rate of change of discharge, or ‘time to peak’). In contrast, it seems that the duration of peak flow can significantly influence the final configuration and that steady ice jam profile models predict ice thickness profiles consistent with sustained flow changes. This importance of peak flow duration on the final ice jam configuration may have important implications for the design of stable ice covers on river subject to hydropneaking.

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References


Figure 1. River1D ice jam and RIVJAM model profiles in comparison with measurements of final ice jam consolidated by 85% increase in discharge: (a) initial; (b) final ice accumulation.

Figure 2. River1D ice jam and RIVJAM model profiles in comparison with measurements of final ice jam consolidated by 55% increase in discharge: (a) initial; (b) final ice accumulation.
Figure 3. River1D ice jam and RIVJAM model profiles in comparison with measurements of final ice jam consolidated by 24% increase in discharge: (a) initial; (b) final ice accumulation.

Figure 4. Comparison of final ice jam thickness profiles computed using different methods to quantify internal resistance of the ice accumulation for a 24% increase in discharge.
**Figure 5.** River1D inflow hydrographs for various times to peak.

**Figure 6.** Simulated ice jam profiles using River1D ice jam model and RIVJAM model formulation for various times to peak.

**Figure 7.** River1D inflow hydrographs for various peak flow durations.

**Figure 8.** Simulated ice jam profiles using River1D ice jam model and RIVJAM model formulation for various peak flow durations.
Each year during the spring, river breakup brings the threat of ice jam floods to many of the communities across Canada which are situated around rivers. Breakup ice jams occur when snowmelt swells the stream, and lifts and breaks the ice cover suddenly. The ice pieces may initially be carried along with the snowmelt flow, but often become congested – perhaps at a bridge, at a tight bend in the river, or in amongst islands – and when this happens an ice jam is formed. The streamflow is blocked by this dam of ice, and water levels can rise at rates approaching two to three feet per minute, with virtually no advance warning. As a result, damages from ice jam floods are usually extreme – averaging about $60M per year in Canada alone (in 1995 dollars).

The speed and scale of this fast rising water presents a very real threat to human safety as well, and this threat can be expected to increase as our communities continue to grow and population density increases. To ensure safe and expedient evacuations from areas threatened by ice jam floods, practical and reliable river ice jam flood forecasting tools are needed. These must provide advance warning of the expected severity, timing, and magnitude of ice jam floods. This poster reports on the efforts being undertaken by researchers at the University of Alberta, hydrologists from the Department of Indian Affairs and Northern Development Canada, and the Town of Hay River Flood Watch Committee, to develop such an ice jam flood forecasting system for the community of Hay River, NWT.
1. Introduction

The river breakup period each spring is a time of severe flood threat for many Canadian communities. Although the clearing of the winter ice cover can be quite innocuous if the ice cover deteriorates thermally and simply melts in place, it can also be quite threatening if a large snowmelt runoff wave lifts and breaks the ice cover, resulting in ice runs and ice jams. Ice jam formation and release events are among the most dangerous types of flood risk situations, primarily because the sudden congestion of a river channel with large quantities of ice can cause water to rise quickly, inundating flood prone areas with little or no warning. It is in this context that ice jam floods are very different from open water (summer) floods; water level rises of several metres may occur in both cases, but for open water floods, this water level rise occurs over several hours or days. When ice jam floods occur, water levels routinely rise several metres in just minutes. For example, Kowalczyk Hutchison and Hicks (2007) documented the water level rising at more than 0.8 m per minute on the Athabasca River, AB during passage of a 4.4 m high ice jam release wave in 2003. Beltaos and Burrell (2005) report on a documented rise of 0.5 m per minute during an ice jam release event on the Restigouche River, NB in 2000.

One of western Canada’s earliest documented river breakup flood events occurred on the Athabasca River at Fort McMurray, and is described in a letter from Henry J. Moberly dated April 25, 1875 which shows the terrifying and life threatening nature of these events (Blench and Associates Ltd., 1964).

“On the 20 Instant about 2 hours after daylight, the river suddenly gave signs of breaking up and in half an hour from that time the water had risen about 60 feet, and the whole place was flooded – the water and ice passing with fearful rapidity and carrying off everything before them. ... From the time the river first gave signs of starting hardly half an hour elapsed before there was 5 feet of water in the highest building in the Fort, and the Interpreter’s house was carried bodily away and dashed to pieces in the Woods; the Workshop and Men’s houses have been almost destroyed.”

Later surveys at the site suggest that the water actually rose about 40 feet (~12 m); nevertheless, a 12 m water level rise in 30 minutes today would still cause widespread fear and devastation in any community. In densely populated communities such as Fort McMurray, AB, where exit routes are limited, panic during the evacuation process itself could compound the threat to human safety. Modern accounts of near escape followed from the much smaller freeze-up ice jam flood in Badger, Nfld. in 2003 (~2 m rise), with many residents describing barely escaping with their lives.

Such incredibly fast rising flood waters provide very little time to perform even the most basic flood mitigation measures, and this is why human safety is severely threatened, and property damages tend to be considerably higher, than for comparable open water floods. Beltaos (1995) estimated that the annual economic loss due to ice jams in Canada is ~$60 million dollars. The Canadian committee on River Ice Processes and the Environment is currently updating that figure based on more recent data, and the numbers are likely to increase substantially. For example, in 1997 alone, ice jams in just two Alberta communities (the Town of Peace River and Fort McMurray) resulted in ~$9 million in damages.

The threat to human safety associated with the flash floods caused by ice jams is exacerbated by our current inability to reliably predict the potential magnitude and timing of ice jam floods. It
only takes a few false positive predictions to create the “cry wolf” syndrome, which leads to the desensitization of residents to the potential flood threat, thereby increasing the risk of injury or loss of life when a serious flood event actually occurs. In communities such as Fort McMurray, AB, where population density is increasing rapidly in the floodplain, and exits routes are limited, the effective evacuation of 10 to 20 thousand people would depend heavily on adequate flood warning and a prompt response by the residents. With 50% of all flood deaths occurring in situations where people attempt to pass a flooded road (US NWS 2007) one sees the particular threat presented in the ice jam flood situation. Clearly there is a significant need, from both the safety and economic perspectives, to have the ability to forecast the occurrence of high water levels during the river ice breakup period.

Prowse and Beltaos (2002) indicate that climate warming could result in significant changes in river ice breakup severity. Thus, the need for reliable predictive models has never been more critical. We currently do not know if climate warming trends, already significant in Canada’s north, will increase the potential risk associated with ice jam flood events, but that potential does exist. One only has to look at the nature of ice jam flood events in eastern Canada, often associated with mid-winter thaw events, to see the potential future scenarios in northwestern Canada. To ensure the safety and security of people and property in the hundreds of communities across Canada that are threatened by ice jams each spring, practical and reliable river ice jam flood forecasting tools are needed to provide advance warning of the expected timing and magnitude of ice jam floods. This poster-paper reports on the status of a new project aimed at developing such operational tools. Specifically, this project involves integrating modern ice jam flood forecasting technologies within an expert systems interface to develop an Emergency Operations Ice Jam Flood Forecasting Expert System, comprised of the following three key components:

1. A long lead time preparedness forecasting model (i.e., providing a few weeks warning), based on hybrid soft computing techniques. This will produce as output a qualitative forecast of the expected magnitude of breakup flooding (e.g., a low, moderate, or high degree of flooding). This model will help to define the appropriate level of emergency preparedness planning required by both the local disaster services agency and the community’s residents.

2. A moderate lead time event forecasting model (i.e., 12 to 24 hours warning), based on ice jam release event modeling. This will produce as output the expected timing of the ice jam flood event in the community. This model will be closely integrated with event detection technologies such as water level sensors, remote cameras, and satellite imagery.

3. A short lead time evacuation forecasting model (i.e., 6 to 12 hours warning), based on ice jam formation event modeling, which will produce as output the extents and depths of flooding anticipated in the community. This model will be closely integrated with community Emergency Operations systems, to facilitate the orderly and efficient execution of an evacuation plan.

The viability of this forecasting approach is to be demonstrated for a northern community which is currently threatened annually by ice jam floods, specifically the Town of Hay River, NWT. (Figure 1). Component 1 of the project is already underway, and preliminary results for the Town of Hay River have been published by Mahabir et al. (2007). This poster-paper reports on progress to date regarding the field program underway in support of Components 2 and 3 of the project, which focus on ice jam release event forecasting, and ice jam formation modeling,
respectively. This research is being undertaken in cooperation with the Town Flood Watch Committee and the Department of Indian Affairs and Northern Development (DIAND).

2. Site Description
The Town of Hay River (Figure 1) has been selected as the demonstration site for this project for a number of reasons. First, it is prone to severe ice jamming and consequent flooding during spring breakup, often resulting in considerable damage and loss of property. For example, during the spring breakup in 2003, residents of the West Point First Nation and Vale Island had to be evacuated from their homes (Figure 2). In addition, flights were disrupted at the Hay River Airport and severe bank erosion occurred near homes on the Kátł’odeeche First Nation Reserve. Most recently, minor flooding occurred on Vale Island during breakup in 2006 and residents were briefly evacuated in 2005 and 2007 while an ice jam developed adjacent to the West Point Fishing Village.

![Figure 1. Location of study site at Hay River, NWT.](image)

3. Past Research
Dr. Larry Gerard of the University of Alberta initiated a river ice breakup monitoring program in cooperation with the Town of Hay River Flood Watch Committee and DIAND in the late 1980’s, and this program was continued into the early 1990’s by Mr. Martin Jasek. As a result of those efforts, ice jam flood forecasting models were developed and updated (Gerard et al. 1988, 1990a, 1990b, 1992; Hicks et al. 1992, Jasek et al. 1993). These research efforts were primarily focused
Figure 2  Map of the Hay River at Hay River.
on predicting the expected severity of the breakup event, and the potential impacts of ice jam release events from upstream. Between 1994 and 2003, research efforts diminished at the site. However, when a massive ice jam flooded the community in 2003 (Figure 3), it was agreed that the timing was appropriate to reinitiate a research program at the site, and to investigate the potential application of new ice jam flood forecasting tools for the community.

Figure 3. Ice jam flooding in Hay River, 2003 (photo courtesy of the Town of Hay River).

3. River Ice Breakup Monitoring Program

The University of Alberta (UA) has been monitoring river ice breakup on the Hay River at Hay River in cooperation with and the Town Flood Watch Committee since 2004. The primary focus to date has been the development of a cooperative monitoring program, with participation by UA, DIAND and the Town. This program now includes the following components:

- The Town Flood Watch Committee installs and operates remote water level monitoring stations at sites upstream of the community, to record and report on flood waves and ice runs resulting from ice jam release events upstream. Figure 4 illustrates one of these stations, which consists of a boom mounted acoustic sensor. Figure 5 illustrates an example of how this information is posted on the Town’s Emergency Operations Centre (EOC) website.
Figure 4. Remote water level monitoring station at Paradise Gardens upstream of the Town of Hay River.

Figure 5. Example of real time water level reporting on the EOC web site.

- The Town Flood Watch committee also conducts ground and air observations as necessary, and when ice jam risk at the community becomes imminent, members of the Flood Watch Committee are positioned at key points around the community to keep the EOC and residents informed of conditions and to facilitate evacuations as necessary.
• The UA and DIAND cooperatively operate remote monitoring cameras at all of the Town’s remote water level stations upstream of the community. This provides continuous information describing ice conditions during breakup, essential to interpretation of the water level data. In addition quantitative data (such as surface ice concentrations, can be measured from these photographs. Figure 6 illustrates an example of this type of data, obtained in 2007, and the corresponding water level record from the Town’s remote station.

• The UA and DIAND cooperatively conduct aerial reconnaissance flights, extending upstream to the Alberta/NWT border, one to two times daily during breakup. These are coordinated with the Town’s reconnaissance flights. Representatives of the Kátl’odeeche First Nation, the West Point First Nation and the Town Flood Watch Committee all participate in these observational flights with the UA and DIAND. Figure 7 illustrates the 2007 breakup progression diagram developed from these aerial observations.

• The UA and DIAND cooperatively conduct water level and ice velocity measurements using standard survey instruments as well as using lasers and global positioning systems. Real time kinetic global positioning systems (RTK GPS) are also used to measure ice jam profiles in the community.

• When possible, the UA, DIAND and the Town Flood Watch committee conduct discharge measurements during breakup using an Acoustic Doppler Current Profiler (ADCP). Town Flood Watch members play a key role in monitoring upstream ice conditions during the measurement to ensure safety.

Figure 6. Example of ice concentration (UA/DIAND) and water level (EOC) data obtained at Paradise Gardens during breakup 2007.
4. Other Surveys in Support of the Project

In addition to breakup observations, UA and DIAND conducted detailed bathymetry and discharge measurements were in 2005 and 2007 to develop a calibrated 2-D hydraulic model of the delta area. This data will be used to facilitate the 2-D ice jam modeling effort in the delta. Figure 8 illustrates part of the bathymetry obtained for this study component in 2005. In 2007, this survey was extended down to the lake for both the east and west channels. In addition, surveys were conducted on Vale Island, particularly in the vicinity of the airport, to aid in developing flood forecasts for airport operations. More detailed surveys were also obtained in the vicinity of the channel split, to facilitate more precise modeling. A portion of this 2007 survey is illustrated in Figure 9.

5. Modeling Efforts

The proposed expert system will be comprised of a suite of predictive models, each aimed at addressing a specific component of the ice jam flood forecasting expert system. Figure 10 illustrates the various model components. Component 1 – Preparedness Forecasting will be based on Regression and Neuro-fuzzy models (e.g. Mahabir et al. 2007) to provide long lead time, qualitative forecasts of expected ice jam flood forecasting for emergency preparedness planning.
Figure 8. River2D bathymetry plot based on 2005 survey.

Figure 9. River2D bathymetry plot based on 2007 survey.
Component 2 – Event Forecasting, will employ the UA *RiverID* ice jam release modeling component (e.g. She and Hicks, 2006) to predict the timing and magnitude of waves resulting from the release of ice jams upstream of the community. This will employ the event detection data coming from the remote water level monitoring stations upstream. In 2008, real time access to these remote stations will begin to include photos as well as water level data.

Component 3 – Evacuation Forecasting, will be based on 2-D ice jam formation modeling in the delta. Model development is currently underway.

![Figure 10. Schematic of modeling strategy for ice jam flood forecasting expert system.](image)

### 5. Summary and Conclusions

This paper presents an overview of a new project aimed at developing an integrated expert system for ice jam flood forecasting. The project employed state-of-the-art monitoring techniques, both to supply model calibration data, and to feed into the flood warning system. In addition, new ice jam flood forecasting techniques will be developed and/or enhanced for practical application. The Hay River at Hay River will provide the test case for demonstrating this expert system. This is a cooperative project involving the University of Alberta, the Department of Indian Affairs and Northern Development Canada, and the Town of Hay River Flood Watch committee. Cooperation and collaboration with the Kátl’odeeche First Nation will also enhance the practical value of this research study.
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References
Ice measurement
Development of an Ice Thickness Monitoring Apparatus Based on a Magnetostrictive Displacement Sensor

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Abstract

Magnetostrictive displacement sensor has the characteristics of high precision, high reliability and multi-position detecting. Basing on the principle of this sensor and the special requirement of environmental suitability, an ice monitor thickness apparatus is developed and it can observe the fine process of ice thickness variations with large temporal and spatial resolutions. The apparatus has been used to measure sea ice thickness variation at Zhong Shan Station, Eastern Antarctica for about 6 months, with the precision of ±2mm, up to an hourly level in temporal resolution. The data give a powerful support to the development and modification of ice mass forecast models. This apparatus satisfies the needs of obtaining ice thickness with high precision for ice science and ice engineering.
**Introduction**

Ice thickness is the most basally integrative and crucial important parameter to describe ice state and to through understand the couple process among atmosphere, ice and water (Perovich et al., 1997), and to settle the interaction process between ice and the structure in ice-infested water (Hopkins, 1997). With the intention to through understand the change of global climate, and to prevent or reduce disaster involved sea ice, river ice, lake ice and reservoir ice, more attention has been play to the thermodynamic growth process of sea ice and fresh ice. At present, some ice mass balance model with high resolution has been set up (Launiainen and Cheng, 1998; Smedsrud et al., 2006), thereby more field data of the variations of ice thickness with high resolution and high sampling frequency is required to give the evidence for validating the numerical modeling results and to give the precise parameter for engineering construction and manage in ice region.

There are hitherto several ice thickness observing methods, among these methods satellite remote sensing is a good way to get data on global scale (Laxon et al., 2003), but it must be modified and explained by field observed data. The traditional drill-hole measurement and thermal wire measurement are recognized as the reliable methods to measure ice thickness in field all the while (Perovich et al., 1997), however, the temporal and spatial resolution of this method is variable and tends to be poor. Thereby they fail to capture the high-frequency variations of ice growth rate due to rapid changes in meteorological conditions and the oceanic heat flux. There are other ice thickness automated instruments, such as radar penetration technique (Sun, et al., 2003), submarine sonar profiling (Rothrock et al., 1999), mooring upward-looking sonar (Strass, 1998), the combination of laser and electromagnetic sounding technique (Worby, et al., 1999), etc. One drawback of these methods, however, is that the precision of them could be affected by ice characters or circumstance condition generally. Thereby it could not detect the fine process of ice growth and decay accurately, and give the precise parameter for engineering construction and manage. Thus it is indispensable to develop a new automatic technology with high resolution for in-situ ice thickness observation.

Magnetostrictive displacement sensor has the characteristics of high precision, high reliability and multi-position detecting (Hristoforou et al., 2006). A new ice thickness monitor apparatus basing on this sensor has been developed basing on the principle of this sensor and the special requirement of environmental suitability (Li et al., 2005). This apparatus has been utilized to monitor landfast sea ice thickness around Zhongshan Station, Eastern Antarctica for about 6 months in 2006.

This paper will discusses the work principle of the design in detail and present its first field application in Antarctica. Especially we assessed the utility of this apparatus.

**Work Principle**

Change of magnetization direction in ferromagnetic substance will arouse the change of medium crystal lattice space, which will also cause the change of length and volume in ferromagnetic substance. That is called by magnetostrictive displacement phenomenon also named by
Wiedemann Effect, its inverse effect is Villari Effect. Magnetostrictive displacement sensor is designed according these two effects (Mehnen et al., 2003). The sensor is illustrated in Figure 1. Under the excited action of the pulse current, the circular magnetic field, vertical to the permanent magnetic field formed by magnetic loop, will be produced by the magnetostrictive delay line (MDL). Subsequently Wiedemann effect will bring the annular mechanical torsional distortion, which could spread to the both ends of the MDL in the form of torsional wave. When the echo reaches the checking machine, the induction electric pulse will be activated by the Villari effect. The position of magnetic loop 1, \( L_1 \) can be expressed as an equation: \( L_1 = v \cdot t_1 \); Here, \( v \) is propagation velocity of torsional wave, \( v = 3000 \text{m/s} \), and \( t_1 \) is the period of time at which the excited pulse appeared up to that the echo reached the checking machine. According to this equation, it is similar with distance measurement theory based on electromagnetic wave or sound wave (Worby, et al., 1999; Sun, et al., 2003), but the propagation time can measured easily and accurately based on magnetostrictive displacement theory, as this wave velocity is steady in different circumstance condition. Therefore the magnetostrictive displacement sensor has a characteristic of high precision for positions detection. In addition, multi-position can be detected by this technique because of the magnetic loops setting in different position can be detected synchronously. The magnetostrictive displacement sensor has been utilized for position measurement in many practice domains, such as for detecting the position of pneumatic pistons, liquid level and absolute ground velocity, field sports etc (Mehnen et al., 2003; Hristoforou and Chiriac, 2002; Hristoforou et al., 2006).

![Figure 1. The principle sketch of magnetostrictive displacement sensor.](image-url)

The character of high resolution and steady operation of this sensor makes it ideal for ice thickness measure. Basing on this sensor and the special requirement of environmental suitability, automatic measurement and low-power dissipation for applications in field, a new ice thickness monitor apparatus and the corresponding software has been designed. This apparatus is composed of apparatus box and two measuring poles as shown on Figure 2. The apparatus box and the measuring poles are connected by an air pipe and a cable. The air pipe is used to connect the cylinder which is fixed in the apparatus box with the buoy which is jointed with the nether magnetic loop; the cable is used to connect batteries with the mini-type windlass, and to transmit...
data records to the data logger fixed in the apparatus box. The air pipe goes pass through one measuring pole, and the MDL is set in the other measuring pole. Two moving magnetic loops are fixed in the movement machines along the measuring poles and one settled magnetic loop are fixed on the top of the measuring pole. The measuring procedure is completed by controlling that two moving magnetic loops. When measurement to be performed, the upper magnetic loop which is jointed with a heavy hammer is released from the draught of the mini-type windlass; and then move down under the gravitation effect, be set on the sea ice/snow cover surface gently. At the same time, the buoy will be filled with air supplied by the cylinder through the air pipe. Afterward, the nether magnetic loop will move up under the buoyancy and be set under the water/ice interface finally. The distance from both the upper magnetic loop and the nether magnetic loop to the settled magnetic loop will be detected by the magnetostrictive displacement sensor. The distance between that two moving magnetic loops is the full-thickness including ice and snow cover. The variations of the positions of the ice/snow cover surface and the water/ice interface are acquired by comparing the position of that two moving magnetic loops with their original value respectively. The measurement data will be transmitted to the data logger.

When the detection process has been accomplished, the upper magnetic loop will move up under the draught of the mini-type windlass through the strings steel wire, meanwhile, the nether magnetic loop will move down after air in the buoy has been set free. This process after detection avoids the upper magnetic loop from being covered by snow and the nether one from being frosted under ice. The data stored in the data logger can be downloaded to computer through the corresponding software.

The design precision of magnetostrictive displacement sensor is ±1mm, and the expectant practical precision of the ice thickness monitor apparatus is ±2mm. The design operation ambient temperature of the apparatus is -55~50 °C.
**Figure 2.** Configuration sketch of the ice thickness monitor apparatus.

**Experiments and discussion**

The apparatus was utilized to measure landfast sea ice thickness around Zhongshan Station, Eastern Antarctica in 2006. The apparatus was disposed in the site (S69°22′9″, E76°21′45″) with ice cover of 0.25m-thickness and water of 9m-depth on 27 March 2006. The measuring pole was fixed in the ice hole of 0.25 m-diameters. The detection spans for detecting the ice/snow surface and the water/ice interface were 0.75m and 2.25m respectively. Three days later, the data collection process was initiated, and endured to 21 September 2006. The ice thickness grew from 26cm to 160cm.

During experiment, the data sampling interval varied from one sampling every 2 hours to every 3 hours. 1460 records of the ice/snow surface and 1460 records of the ice/water interface have been collected in total. After the false data records have been rejected, there were 1352 effective records of the ice/snow surface and 1368 effective records of the ice/water interface respectively. It is illuminated that the effective operation fraction of this apparatus for detecting the ice/snow surface and the ice/water interface in field was 92.6% and 93.7% respectively.

In an attempt to investigate the precision of sea ice thickness records, it must be testified that the data records are valid. For this purpose, a total of 24 drill-hole records have been collected intermittently in range of 2 m around our apparatus. The drill-hole records and the apparatus records derived at the same time are shown on Figure 3. The overall absolute bias between the drill-hole records and the apparatus records is 1.5±1.0cm. It is proved that the apparatus records are valid basing on taking into account the precision of drill-hole records of ±0.5cm and the thickness of the ice can vary even over short distances.
The precision of apparatuses in field can't be estimated by comparing with the drill-hole records as the precision and the sampling frequency of drill-hole is not as high as the apparatuses. In order to estimate the precision of apparatuses in field subtly, an approach that the growth rate of sea ice on a certain day is constant has been assumed. Basing on this approach, firstly, the linear regression according to least square method is applied, and then, the slope of the linear regression trend is recognized as the growth rate of sea ice on that day, finally, the precision of apparatuses in field on that day can be estimated as following:

\[
Er_i = \frac{\sum |Th_i - \bar{Th}_i|}{n} + \sqrt{\frac{\sum (Th_i - \bar{Th}_i)^2}{n}}, \quad (1)
\]

\[
Er_j = -\left\{-\frac{\sum |Th_j - \bar{Th}_j|}{m} + \sqrt{\frac{\sum (Th_j - \bar{Th}_j)^2}{m}}\right\}, \quad (2)
\]

\[
Er = \max\left(\left|Er_i\right|, \left|Er_j\right|\right), \quad (3)
\]

Where, \(Er_i\), \(Er_j\), and \(Er\) are the positive error, the negative error and the general error, respectively; \(Th_i\), \(\bar{Th}_i\) and \(n\) are the sea ice thickness records which are larger than the estimate value, the estimate value according, and the number of \(Th_i\) respectively; \(Th_j\), \(\bar{Th}_j\) and \(m\) are the sea ice thickness records which are smaller than the estimate value, the estimate value, and the number of \(Th_j\) respectively.

The errors of all field data have been estimated. The maximal absolute value of both the positive error and the negative error is 2mm. It is indicated that the precision of apparatuses in field during experiment is ±2mm.
In general the precision estimation of apparatus illuminates the error of data records in field is which is slight worse than the designed precision of the magnetostrictive displacement sensor of under the execrable field conditions in Antarctica, Whereas it reach the practical precision of the ice thickness monitor apparatus. Thereby it is no doubt that this apparatus has a capacity of detecting the variations of snow/ice thickness with high precision. It is suitable for obtaining long time series of ice/snow surface and ice bottom at millimeter resolution synchronously. The filed data with high precision will give a powerful support for analyzing the fine process of ice growth and decay, will give a powerful support for through understanding the couple interaction process among atmosphere, ice and water, furthermore, for modifying ice mass equilibrium modeling seriously.

Conclusions
The developing of this apparatus proves that magnetostrictive displacement sensor is a good developing direction for ice thickness point real time measuring. With the accumulating of field experience and ceaseless improvements and tests, the apparatus will play active effect to sea ice, lake ice and river ice thickness measuring. It will supply technical supporting for studying of ice science and ice engineering.

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References


Research on a New Measurement Method of Ice-Thickness

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Abstract:
As water freezes into ice (or as ice melts into water), great changes will take place in its physical parameters such as (electric) resistance, electric capacity and temperature. The value of these parameters can also be distinguished from those in the air above the ice surfaces that are under detection. According to the aforementioned features, the author of this paper proposed a new theoretical framework of measurement on ice-thickness and beneath-ice-surface water levels by distinguishing the physical characteristics of air, ice and water. The result of field-course data analysis revealed a series of interesting conclusions such as ice as a weak conductor, capacitance interval monotonicity of air, ice and water, and the jumping point of electric capacity under unique temperature, etc. Based on these results, the author introduced the design of the structure of a new-type of ice-thickness sensor, its measurement method, and its field-application in automatic measurement on ice thickness.

Keywords: physics characteristic, resistance, capacitance, thickness of ice layer, sensor

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1. Introduction

Freezing is a common natural phenomenon in many countries and regions such as North China, Russia, North Europe, and North America, which poses serious security problem for many hydraulic engineering facilities. In recent years, global warming and polar iceberg melting have become two of important factors threatening human surviving environment. By the observation on the thickness change of glacial and ocean ice in Antarctica and Arctic, the most direct weather and hydrographic data for analyzing the global climate can be provided. Nowadays, continuous and fixed-pointing auto-measurement of ice-thickness has been one of major subjects of Chinese and international polar scientific expeditions.

Some physical measuring methods are commonly used today, such as satellite radar and sonar measuring method etc, which are no-contact, rapid and big area etc, but have bigger error as a result of ice physical character and can not gain real-time data of ice inside and water level beneath ice in most cases.

After a quantity of experimental data analysis, the paper presents a new auto-measuring method and realizable way for the ice-thickness and level of water, which is based upon the physical characteristic difference among air, ice and water,

2. Ice-thickness measurement theoretical basis on physical characteristic difference among air, ice and water

Weak Conductivity of Ice

Conductivity is one of important and mutual characteristics of ice and water. It has been proved by experiments that natural water with conductivity ion (such as rain water, river water, underground water and tap water) is conductive in the normal atmosphere temperature. With the falling of temperature, water resistance value rises gradually ranging from decades to hundreds kΩ. When the temperature drops to below 0°C and water changes to solid ice. General documents regard ice as insulator with vast resistance value. In order to find a new method to measure ice-thickness, we did a quantity of experiments of river water on resistance changing process during ice forming and melting usead the equivalent circuit showed in Fig. 1. Fig.2 shows a set of typical experimental data on equivalent resistance gained by using the equipment in the process of ice forming-and-melting in April 2004

![Figure 1. The equivalent circuit of measuring for Ice-equivalent-resistance](image-url)
Figure 2. Trial curve of resistance in ice on April, 2004

The curve in Fig.2 shows: with the gradual falling of temperature, ice (solid water) resistance value rises greatly, changing from several MΩ (original conductive water solution) to hundreds MΩ. Meantime, in every experiment, there is a temperature dividing point (mostly lower than -20°C) where the value increases dramatically. Ice still presents conductivity in certain temperature range (for example, 0°C --20°C). Although the conductivity is very weak compared with liquid water, ice resistance is rather different from that of insulator defined in physics. Therefore, we call it weak conductivity, the condition that natural water with ion still presents conductivity after freezing within certain temperature range. Ice weak conductivity’s changing trend is similar to water: the lower temperature, the weaker conductivity.

2.2 Capacitance Research of Air, Ice and Water

Capacitance changing with temperature is another important physical characteristic of ice and water. Using plane-parallel capacitors (made by copper clad plates as Fig.3), capacitance characteristics are measured respectively when the media between plates are air, ice and water. Fig.4 show capacitance data curve measured by two kinds of plane-parallel capacitors in certain temperature range.
Figure 3. Structure diagram of plane-parallel capacitor

Figure 4. The curve of the capacitance data in the process of freezing–melting

From Fig.4, capacitance shows remarkable monotonicity during not only freezing but also ablation process, and capacitance values of air, ice and water differs greatly from one another.

2.3 Ice-thickness Measurement Principle Based on Air, Ice and Water’s Different Physical Characteristics

In the measuring processes of river/ocean ice forming-and-melting and beneath-ice-surface water level, three major substances are involved in the stereo space measurement, which are above-ice-surface air, ice layer and beneath-ice-surface water (air layer may exist between ice and beneath-ice water). After comprehensive analysis of their resistance and capacitance characteristics, based on the resistivity, a new division method is introduced. Since air, ice and water have different electric conductivity, the vertical space of river/ocean ice forming-and-melting measuring process can be divided into three different regions: insulator (air layer), weak conductor (ice layer) and excellent conductor (beneath-ice water). Based on ice weak conductivity and its capacitance monotonicity obtained from experiments and combined with electronic information processing techniques, a new method to measure ice-thickness and water-level beneath ice is presented. The principle is illustrated as Fig.5.
Its measuring principle is: by controlling the electronic switch and vertically cutting the measured space into N planes (N is determined by measuring precision), each plane’s resistance and capacitance are measured under the control of a program. Ice inner situation and its above/below interfaces can be determined by the results, and then ice-thickness.

3. Application experiment of the resistance-ice-thickness sensor

Based on the principles mentioned above, we developed the resistance-ice-thickness sensor and made real application experiments more than 20 days in the ice pond of laboratory. Fig.6 shows the resistance-ice-thickness sensor in field experiment. Fig.7 displays part of typical data curve obtained by the experiment of the resistance-ice-thickness sensor.

Figure 6. The resistance-ice-thickness conductivity sensor in the ice pond of lab.
Figure 7. The data curve from the resistance-ice-thickness conductivity sensor in lab.

4. Conclusion

The data curves in fig.7 show that the new ice-thickness sensor can rapidly and precisely determine air and ice interface as well as water-level scale beneath ice, and then precise value of ice-thickness can be obtained. Its advantages are that not only ice layer’s above/below interface can be measured in still water or flowing water beneath ice, but also water level and air-gap beneath ice can be determined. In addition, fixed-point ice and the whole ice-forming-and-melting process on the vertical cross-section beneath ice can be automatically tracking measured.

References


An Overview of Ice and Bathymetric Profiling using Ground Penetrating Radar (GPR)

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Winter ice roads are built on water bodies in the Arctic to transport goods and machinery to otherwise isolated mines and communities. Ice thickness is one of the major factors that contribute to safe ice road travel. By continuously profiling the ice with Ground Penetrating Radar (GPR), thin areas can be detected by measuring the time it takes the electromagnetic pulse to travel to the ice-water contact; then converting this time to depth using the ice's dielectric constant. Traditionally, for calibration purposes, an auger hole is drilled in the ice and its depth is used to back calculate the dielectric constant of the ice. This calibration method gave a fairly accurate indication of ice thickness. However, changes in the ice's dielectric properties are primarily a function of temperature and density. Therefore, with larger data sets, greater separations in latitude, and differences in the ice's formation, the dielectric is in constant flux. EBA Engineering Ltd. has developed a new radar system that can directly measure the ice's dielectric while profiling. As well as ice data, bathymetry data is also collected to analyze the relationship between water depth and the formation of ice. This paper uses ice road data collected by the new and old radar systems and looks at what factors, including bathymetric, lay behind the ice’s varying properties.
Background

Ice roads (on lakes) and ice bridges (on rivers) are used in northern climates where road access is limited and traditional road or bridge construction is expensive. In the Northern Territories of Canada, ice roads are essential in bringing in heavy equipment and supplies, such as fuel, to otherwise isolated communities and mining operations. Because building an all season road is very costly and can have an environmental impact on the sensitive tundra that would be hard to eliminate, using frozen water bodies is a preferred method of transportation.

The safe building and maintaining of ice roads and bridges is extremely important as accidents are costly, both materially and in human terms. Injury and death from breakthroughs are known to happen in areas that have not been properly assessed. Severe environmental damage to water bodies can also result given the types of cargos being transported.

Speed, bathymetry, and ice properties (such as thickness and ice strength) are three main factors that combine to determine ice road/ice bridge safety and load bearing calculations. Of these three, the speed of the vehicle can be monitored and controlled; the bathymetry of the lake or river bed is a slow-changing variable and can be mapped; and the ice properties will vary and must be measured.

A borehole jack can be used to calculate the ice strength by measuring the hydraulic pressure and jack displacement of the ice at various intervals. Ice strength measurements using a loaded truck and measuring the surface deflection of the ice sheet can also be used. Two methods have traditionally been employed to measure ice thickness. One is augering a hole and simply measuring the ice thickness directly. This can be time consuming, provides only discrete values and is impractical over long roads. The other is the use of Ground Penetrating Radar (GPR) to continuously profile the ice thickness.

GPR can also be used to collect bathymetry. Bathymetry, or lakebed topography, can affect the carrying capacity of the ice in three ways. Firstly, where the lake bottom is shallow, the ice can freeze to the ground (grounded ice or ground-fast ice) and is therefore not considered part of the floating ice sheet. Although the carrying capacity of grounded ice exceeds the capacity of the floating ice cover, it can develop stress points at the transition zones between the two. Secondly, shallow areas or shoals can erode the ice from underneath and prevent the ice from building at that location. Thirdly, lakebed morphology can confine hydrodynamic wave energy that can cause damage to the ice sheets or blowouts (uplifting of the ice sheet.)

GPR Ice and Bathymetric Profiling

Theory

GPR transmits an electromagnetic (EM) pulse of extremely short duration from a transmitter antenna, which is partially reflected from multiple subsurface objects. The reflections are detected at the receiver antenna, which accurately records the pulse travel time. A series of these soundings, taken at regular intervals along a line, builds up a cross-sectional profile of the reflections beneath the line.

As the EM pulse propagates downward through the ground, it encounters and passes through materials having specific physical properties. The two important properties for GPR surveys are the dielectric
permittivity and electrical conductivity. The first property, the electrical conductivity of the material, controls the attenuation rate, of the EM pulse, or how quickly the energy of the EM pulse is absorbed by the material it is passing through. This property governs how far the GPR signal will penetrate the material. The dielectric permittivity is a measure of the charge capacity of various materials (whose ratio to that of free space is called the material’s ‘dielectric constant’) and it controls the velocity of the EM pulse through the material.

Velocity is related to the dielectric constant through this generalized (and simplified) depth equation:

\[ d = \frac{ct}{2\sqrt{\varepsilon_r}} = \frac{\nu m t}{2} \]  

where t is the travel time of the propagation medium, \( \nu_m \) is the velocity of the radar pulse, \( \varepsilon_r \) is the dielectric constant (or relative permittivity), and c is the velocity of the electromagnetic wave in a vacuum.

At interfaces having different dielectric constants, a dielectric contrast exists and a reflection of some of the EM pulse’s energy will occur. The magnitude of the energy reflected is proportional to the contrast in permittivity.

Water has a very large dielectric constant of approximately 81, whereas the dielectric constant of ice can range between 2 and 4. Therefore, a large reflection is always encountered at the water-ice interface. An abrupt change in dielectric constant will be more detectable than a gradual change. This makes GPR the ideal tool for ice thickness profiling. If the thickness of the ice is known (for example, through auger calibration) the velocity and thus the dielectric constant can be back calculated. Table 1 shows typical electrical properties of various materials.

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Dielectric Constant</th>
<th>Velocity (m/ns)</th>
<th>Conductivity (mS/m)</th>
<th>Attenuation (dB/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet Sand</td>
<td>10-30</td>
<td>0.05-0.09</td>
<td>0.2-10</td>
<td>0.03-0.3</td>
</tr>
<tr>
<td>Sand</td>
<td>3-5</td>
<td>0.13-0.17</td>
<td>0.01-1</td>
<td>0.01</td>
</tr>
<tr>
<td>Silts</td>
<td>5-30</td>
<td>0.05-0.13</td>
<td>0.5-100</td>
<td>1-100</td>
</tr>
<tr>
<td>Clay</td>
<td>5-40</td>
<td>0.05-0.13</td>
<td>5-1000</td>
<td>1-300</td>
</tr>
<tr>
<td>Ice</td>
<td>3-5</td>
<td>0.13-0.17</td>
<td>0.01-1</td>
<td>0.01-1</td>
</tr>
<tr>
<td>Granite</td>
<td>4-6</td>
<td>0.12-0.15</td>
<td>0.001-3</td>
<td>0.01-1</td>
</tr>
<tr>
<td>Fresh Water</td>
<td>80</td>
<td>0.033</td>
<td>0.5-2</td>
<td>0.1</td>
</tr>
<tr>
<td>Salt Water</td>
<td>80</td>
<td>0.033</td>
<td>5 x 10^3-3 x 10^4</td>
<td>1000</td>
</tr>
<tr>
<td>Air</td>
<td>1</td>
<td>0.33</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
Traditional Radar Ice and Bathymetric Profiling System

The traditional ice profiling system consists of a bi-static (separate transmitter and receiver antennas) GPR system, with a 500 MHz antenna for ice and a GPS system for positioning. The data is stored digitally on a laptop and can either be interpreted in real-time while collecting or post processed. The real-time interpretation mode usually allows the user to input a velocity so that a thickness scale can be shown on the screen. The operator will then use this display to visually assess the ice condition and identify and mark in the field suspect areas. This mode is usually used by ice road construction personnel and technicians pioneering the ice road routes. If post-processing is required, specific velocities can be applied to various portions within the data set and QA analysis software can be run. It is, however, currently unusual for most users to follow this approach due to the large quantities of data collected and the time required to process the data that permits effective review in a timely manner.

Historically for ice-profiling, EBA has used a digitally modified GSSI SIR-8 GPR system, with a 500 MHz antenna and a Trimble Ag132 / CDGPS real time differential GPS system for positioning. Equivalent hardware such as MALA’s Ramac system or Sensors and Software’s Ice Noggin are used by others. Data collection profiling speeds are typically around 15 km/hr to achieve a horizontal resolution of one trace every 20 cm. In EBA’s case, the raw data is imported into custom analysis routines running in a data analysis package (Matlab, The Mathworks). Data processing parameters are applied to the raw data files using digital filters and gain functions to try to remove as much background noise as possible. For most ice data little processing is needed to see the ice-water reflector. Positioning corrections are applied to remove any offset errors between the GPS antenna location and the GPR antenna. Final plots were generated to display the ice data profiles and corresponding GPS coordinates in UTM coordinates, using the NAD 83 datum or another applicable datum. These processing steps are carried out in a semi-automated procedure to speed processing time requirements. Currently the most intractable issue is how to document and archive all of the data collected in a fashion that can be reported to various management groups with minimal turn-around time.

Typically, for bathymetric profiling a similar system is used with a 120 MHz antenna. This usually gives depth information for a target zone of up to 6 m. Lower frequencies, such as 50 or 80 MHz, can also be employed to profile portions of the lake deeper than 6 m. Due to the optimization of the system for deep information, ice information is usually lost while using lower frequency antennas. Presently, EBA has developed software that joins the bathymetry and ice data based on GPS information to make a combine two-layer thickness interpretation.
**Case Studies**

Several case studies have been reviewed for this paper to study the changes in ice velocity and bathymetry in many different locations and conditions.

![Map of Canada showing each of the 4 study areas.](image)

**Figure 1**: Map of Canada showing each of the 4 study areas.

Four areas shown as follows:
- Tibbitt to Contwoyto Winter Road, Northern Territories, 580 km ice road
- Moosenee to Victor Mine Winter Road, Ontario – 7 river crossings
- Colomac Winter Road, Northern Territories, 183 km ice road
- Athabasca River Ice Crossing, Alberta – 750 m river crossing
Figures 2 and 3 are selected example profiles from the Athabasca River Ice Crossing and from the Moosenee to Victor Mine Winter Road, respectively.

**Figure 2:** This is an example of a GPR ice profile from the Athabasca River Ice Crossing, north of Fort McMurray, Alberta. The bridge is usually built up by man-made flooding and has several areas of grounded ice, where the ice is frozen to the river bed. (a) shows GPS positioning and auger locations (b) shows the interpreted ice depth profile. Cyan is the ice-water interface, red is grounded-ice, and dark blue is the river bottom. Bathymetry was collected separately with a 120 MHz antenna (c) shows the processed 500 MHz ice radar profile.

Of interest at the Athabasca River Ice Crossing is a second reflector that is frequently misinterpreted as an ice/water reflector where grounded ice is present. It is in fact a reflection at the transition from frozen to unfrozen soil. This reflector will increase in depth through the winter season as the frost penetration depth deepens. Stress points can develop at these transition zones and as the water levels fluctuates with river currents or by moving load hydrodynamic waves, traverse cracking can be identified along these points. If the cracking continues, the ice sheet can form hinges, which continues to crack and heal causing in some cases a vertical step that then needs to be bridged to become functional for travel.

Also noticeable is the ice to frozen ground reflection, which is much fainter than the frozen to unfrozen ground reflection due to the reduced dielectric contrast. GPR can, with relative ease, diagnose where water or unfrozen saturated soils are present below the ice based on the strength of the GPR reflection at that interface.

The ice environment can be inferred from the character of the ice/water reflection. At the margins of the
main channel the ice is irregular in thickness when compared to the ice over the main channel. Statistics describing the thickness variations over distance can be used to identify the formation history of an ice sheet and this has a bearing on the long term stability of the ice at the end of the winter road season. Smooth ice indicates that the ice has frozen in place without much movement. Irregular ice indicates a more dynamic environment and generally represents locations where stress may be present and is frequently where ice will deteriorate first at the end of the season.

**Figure 3**: This is a section of a GPR ice profile from the Granny Point Crossing, along the Winter Road from Moosenee to Victor Mine, Ontario. (a) shows GPS positioning and auger locations (b) shows the interpreted ice depth profile. Cyan is the ice-water interface, red is grounded-ice, and dark blue is bathymetry. (c) shows the processed 500 MHz radar profile.

Notice in Figure 3 how uneven the ice growth is. If relying on auger holes to measure ice thickness every 100m, a lot of ice deflection information and thin ice areas could potentially be missed. The thinnest ice zone corresponds to the deepest part of the river channel, most likely where the water flow is the fastest; which in turn is eroding the ice underneath and preventing ice growth at that location.

Shallow areas or shoals can also be problematic as in the case of Waite Lake along the Tibbitt to Contwoyto Winter Road (Figure 4). The movement of water from the passing trucks caused erosion from underneath the ice, preventing the any building up of the ice sheet.
Figure 4: This is a section of a GPR ice profile from the Waite Lake, along the Tibbitt to Contwoyto Winter Road, Northern Territories. (a) shows matched GPS positioning in plan view (b) shows the interpreted ice depth profile. Cyan is the ice-water interface and dark blue is bathymetry. (c) shows the processed 500 MHz radar profile. (d) shows the processed 120 MHz bathymetry profile.

The variation in ice and bathymetry raises a lot of other issues such as:

- Can bathymetry be used to plan for better ice road routes
- How to assume ice velocities given multiple scenarios – for example, transitions from natural ice to flood ice.
- How does grounded ice and transition zones affect load calculations / safety?

Also, the traditional systems and method has several drawbacks, for example:

- Can only collect ice or bathymetry at one time using a one-channel system.
- With large data sets/distances – Augering takes time and velocities can change drastically over a large distances but also locally, for example, transitions from natural to flood ice.
- The collection speed cannot increase without compromising a reasonable horizontal resolution required to identify weak spots such as cracks.

With all these issues and problems it became apparent to EBA that a new system would be desirable.

**Ice Road Radar - New Ice Profiling System**

To address this issue of varying ice properties, EBA has been working to adapt a prototype GPR system (US patent 5835053), developed initially for pavement applications. This system uses a surface-coupled transmitter to emit a pulse and measures the reflection using an array of receiver antennas located at
different known spacings from the transmitter. This will allow for the velocity of the ice to be calculated in real-time as radar data is collected.

The goal of the new system is to:
- Improve on the accuracy of calculating and recording changes in the ice’s velocity.
- Increase the sampling rate to allow for greater collection speeds.
- Have the capability of multiple channels and frequencies so that real-time ice velocity and thickness measurements can be calculated with bathymetry data.
- Real-time depth processing for ice thickness to improve real-time data analysis and data turn-around times.

![Diagram](image)

**Figure 5**: A simplified sketch of the system showing the array configuration for determining time differences is below. (adapted from Davis et al., 1994). Where T is your transmitter, R₁ and R₂ are your two receivers, separated by S₁ and S₂, respectively.

Once the velocity is calculated, the ice thickness can be determined at this location. The approach essentially relies on the Normal Move Out (NMO) method used in seismic data processing by using the array as a Common Midpoint gather (see Yilmaz, 1993, pp. 157).

The system automatically tracks reflectors and uses a rule based approach to differentiate between floating and grounded ice; ice and frozen ground; and frozen and unfrozen ground interfaces. The system will also automatically track the lake-bed reflector.
Figure 6: An example profile showing one channel collected from Bluefish Lake, Northern Territories, collected with the new Ice Road Radar system. (a) shows GPS positioning (b) shows the interpreted ice velocity profile, with cyan being the ice-water interface. (c) shows the processed 1.5 GHz radar profile.

The new ice profiling system records 3 channels of ice data at 1.5 GHz and 1 channel of bathymetry data at 200 MHz. It also records real-time differential GPS and real-time ice thickness and velocity data. It has been in development for three years with final testing taking place this past winter season on the Tibbitt to Contwoyto and Colomac Winter Roads.

Conclusions

Several factors affect the ice's physical properties such as the temperature and the ice’s composition – essentially a mixture of ice, air and water. It is possible to have different ice velocities for one data set and variations during the ice season. The presence of flood ice will also increase the ice’s velocity due to the higher percentage of air in the composition.

In order to tackle varying ice conditions and velocities, and to solve issues inherent with traditional ice profiling systems and data handling limitations, a new ice profiling system is being developed.

With the new radar ice profiling system being capable of collecting velocities in real-time, there could be a reliable means to study the relationship between ice velocity and ice strength. This potentially
provides a means to monitor ice strength continuously in a fashion similar to ice thickness.

Once the ice thickness is calculated, the true lake or river depth can then be calculated based on a two-layer model, therefore making GPR profiling in the winter a highly reliable technique to collect bathymetry information.

References


Acoustic Detection and Study of Frazil Ice in a Freezing River during the 2004-2005 and 2005-2006 Winters

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Results are reported from two annual measurement programs carried out on the Peace River which demonstrated near-realtime capabilities for detailed monitoring of frazil ice particles suspended in the water column using upward-looking acoustic profilers. Prior to local stabilization of the seasonal ice cover, frazil ice was found to be present only during intervals of measurable supercooling when it was characterized by target strengths uniformly distributed through the middle and upper water column. Ice cover stabilization changed this situation, allowing detection of frazil-like targets at all times but with the upper water column now dominated by the presence of a layer of mobile ice particles associated with target strengths which increased sharply with height. Beneath the stable ice cover, target strength variations in the water column were found to be significantly correlated with both lower frequency variations in regional air temperature and with higher frequency changes in local water levels. Our observations were most consistent with both frazil generation in upstream open water areas and the existence of unknown mechanisms whereby local water level- and/or water speed-dependent processes control the vertical distributions of frazil in the water column beneath the stabilized downstream ice cover. The implications of these results for current models of freezing rivers and for the design of subsequent winter river monitoring and study programs will be discussed.
1. Introduction and Description of Measurements

Suspended individual or aggregated crystals of frazil ice in fresh water bodies often have significant impacts upon water supply, hydro electric, fisheries and other management activities. Effective detection and quantitative characterization of such ice can provide direct input to operational decision-making and for formulating numerical river ice and flow models (Shen, 2006) underlying modern flow management (Jasek, 2006). This note presents results from two recent winter deployments of acoustic profilers in the Peace River, Alberta, Canada which demonstrate near-realtime capabilities for quantitatively assessing suspended frazil content and its sensitivities to environmental factors. The utilized upward–looking sonar measurement technique extends an ice profiling technology originally developed (Melling et al., 1995) to obtain (Melling and Reidel, 1995, Marko, 2003) marine ice draft data from self-contained moored instruments. With a few notable exceptions, collected data have been specific to drifting surface ice. In one of the exceptions, “draft” data from the Bering Sea were used (Drucker et al., 2003) to deduce penetration depths of frazil ice entrained in Langmuir circulation plumes. More recently, Leonard et al. (2006) have used ADCP current profiler signal amplitude data to estimate “platelet” or frazil ice concentrations beneath Antarctic multi-year ice.

The results presented here were obtained in freshwater during B.C. Hydro’s 2004-2005 and 2005-2006 winter Peace River monitoring programs utilizing SWIPS (Shallow Water Ice Profiler Sonar) instruments developed by ASL Environmental Sciences Inc. from the company’s IPS4 Ice Profiler platform. Aside from changes in acoustic frequency, the principal distinction of the SWIPS was confinement of all active components, exclusive of the transmitting/receiving transducer and tilt, pressure and temperature sensors, to a sheltered shoreline enclosure. This change minimized risks of loss or damage of control and data storage components and facilitated access to data for opportunistic changing of measurement parameters. The wet instrument components were mounted on a heavy concrete block, deployed on the river bed and linked to the shore station by 150 m power and communication cables. Measurement parameters, such as the ping or acoustic pulsing rate (usually 1 Hz), the numbers of consecutive pings emitted in each sampling interval (burst) and the separations between sampling intervals, were remotely set based upon data periodically uploaded from the unit’s 65 Mb flash memory. Data-taking alternated, at selectable intervals, between a conventional range-finding mode (whereby ranges to the river-air interface or to the ice undersurface were automatically estimated) and a, more data-intensive, true profiling mode which recorded acoustic return amplitudes from each, roughly, 2 cm deep horizontal slice of the insonified water column. This operational mode was essential for frazil ice detection.

Two different SWIPS instruments were deployed: a low frequency 235 kHz SWIPS1 unit, utilized in both annual programs; and a 545 kHz, high frequency, SWIPS2 unit during only the 2005-2006 study. The general location of the SWIPS1 unit was similar in its two deployments although the 2005-2006 site was in shallower water (3.4 m vs. 6m). The original SWIPS2 site was in 5 m of water about 10 to 20 m offshore of the SWIPS1. Measurements of snow, thermal and slush ice thicknesses were carried out from the stabilized ice cover on three dates in the January-March, 2005 period, accompanied by field and laboratory acoustic transmission and scattering studies. 2005-2006 field efforts were limited to estimating river drift speeds after ice
clearance. Instruments were in place from early November through to May and meteorological data were routinely collected at local and upstream Environment Canada gauging sites.

2. Frazil-Related Results from 2004-2005 Program

Although the 2004-2005 SWIPS1 deployment was directed at documenting surface ice draft, data gathered prior to stabilization (immobilization) of the ice cover in early January, 2005 also showed evidence of occasional, weak, water column targets suggestive of suspended frazil ice presence. In each case, targets coincided with observations of the supercooled (temperatures < 0°C) river water required for frazil formation. Initially, these targets appeared to disappear from the water column after stabilization, presumably due to absence of supercooling beneath a stable, insulating, ice cover. Later, however, similar but more persistent weak returns were detected down to depths extending, roughly, 1 m below the ice cover. Such returns were mostly confined to the period February 24-March 16 and are apparent in the hourly-averaged return strength profiles of Figure 1. A video survey on March 2 showed the presence of large concentrations of drifting frazil flakes at the depths associated with the anomalous returns. Comparable concentrations of frazil flakes were also observed on February 3 which were, however, not readily detectable in the SWIPS record. The stabilized ice cover, itself, showed characteristically weaker returns from its lower reaches which were comprised of porous slush ice. This lower ice layer, according to both SWIPS and field measurements, grew thinner over time, indicative of progressive dissipation and partial conversion into the harder “thermal” ice associated with the upper portions of the ice cover. Ranges corresponding to the modeled and measured positions of the bottom of the thermal ice are included in Figure 1 for reference purposes, although comparisons of actual and acoustically-estimated ranges to points internal to the ice cover points are problematic due to the uncertain composition dependences of the anomalously low sound speeds (relative to water values) detected in the lower ice cover (Jasek et al., 2005).

3. Results from 2005-2006 Program

Evidence of frazil–detection during the 2004-2005 program motivated inclusion of a 546 kHz SWIPS2 unit in the 2005-2006 study program. The higher frequency of this unit was intended to increase detection sensitivity based upon expectations that the scattering cross-sections of frazil particles with radii, \( a < \frac{\lambda}{2\pi} \), where \( \lambda \) is the acoustic wavelength, followed a Rayleigh Law proportionality to the fourth power of acoustic frequency. After corrections for different system gains, SWIPS2 return signal strengths were anticipated to exceed SWIPS1 returns from common targets by 33.6 dB. The resulting sensitivity increases are evident in comparisons (Figures 2a, b) of, roughly, 48 hours of coincident SWIPS1 and SWIPS2 profile data presented as target strengths vs. range for successive returns from bursts of 120 1 s-separated pings. The bursts were emitted at half-hour intervals during a January 11-13, 2006 time interval associated with cold air temperatures and supercooling. Two separate intervals of frazil detection were apparent characterized by essentially range-independent average target strengths for ranges (heights) > 1m above the bottom-mounted transducer. Detection duration was relatively brief, about 3 hours, in the first interval (initiated around 08:00, January 12) but a second, more intense, event followed at about 17:00 on the same day and persisted almost to the end of the depicted period. Averaged over a 2.5 hour period, the strongest of the SWIPS1 water column returns (Figure 2a) in the latter period were comparable to the maximal returns observed with the same instrument.
during the 2004-2005 program. This level was approximately 30.7 dB +/- 3 dB below corresponding SWIPS2 return levels, in accord with anticipated sensitivity differences. This agreement validated the Rayleigh scattering assumption and its underlying target size restrictions, suggesting that detected particle diameters were on the order of and smaller than about 0.8 mm.

Shortly after these observations were made, accumulations of anchor ice on both instruments temporarily blocked the acoustic beams, interrupting data collection. Even worse, buoyancy contributions from ice affixed to the concrete instrument platforms and cables produced uncontrolled changes in instrument positions and orientations. The SWIPS1 platform was overturned, ending its meaningful data collection. The SWIPS2 was fortuitously stabilized in slightly deeper water (by 1.5 m) in late January with its acoustic beam tilted 66° off vertical. A subsequent warming period initiated anchor ice clearance and resumption of SWIPS2 data gathering, albeit with a tilted acoustic beam. Although complicating interpretation of returns from surface ice, beam tilting did not preclude frazil ice monitoring since the isotropic scattering expected from particulate targets still allowed extraction of acoustic return strengths as a function of height in the water column, using the cosine of the tilt angle for range to height conversion. The validity of this approach was verified by noting the close similarity of profiles in equivalently intense supercooling intervals as detected, respectively, prior to and after tilting of the SWIPS2 beam (see the Jan. 13 and Feb. 25 profiles in Figure 3a).

Between February 15 and the February 28 date of local ice cover stabilization, the tilted SWIPS2 beam also detected distinctively stronger returns from moving surface floes with typical maximum drafts on the order of 0.75 m. Returns from water column frazil in this period were

Figure 1. Plots of January-April, 2005 hourly averaged SWIPS1 return signal amplitudes in counts, local water levels and measured and modeled positions of the thermal ice undersurface.
found to be either of low or negligible strength approximately 50% of the time, based upon the representative return levels displayed in Figure 3a. Figure 3a also includes May data, acquired to illustrate ice-free background return levels. The general form of the pre-stabilization frazil profile included intense near-bottom returns (believed to arise from suspended sediments) which fell off sharply with height above the transducer. The steep slopes of the curves shallowed significantly approximately 0.6 m above the transducer, producing characteristically depth-independent returns from heights between 2 m and the bottom of the ice cover.

Local stabilization of a seasonal ice cover typically moves up-river as drifting floes and underlying frazil slush eventually slow into immobility. The “ice front” marking the boundary between the stabilized (downstream) and mobile (upstream) portions of the ice cover progressively advances upstream over the course of seasonal development. The sharply higher water levels accompanying local stabilization are more or less sustained through to breakup and maintain throughput of water from unstabilized upstream areas. The 2004-5 data showed the stabilized ice cover to be a mix of the hard “thermal” constituent of drifting floe ice and an underlying, initially thicker, layer of slushy, consolidating frazil ice with the lower boundary of the hard ice progressively advancing over time into the dissipating and eventually (shortly before breakup) disappearing slush layer (see Jasek et al., 2005). Local frazil generation should be largely absent beneath a stabilized local ice cover and additional upstream advances of the ice front might be expected to progressively diminish the local presence of frazil drifting downstream from increasingly more distant open water production areas. Such conditions beneath a stable ice cover are consistent with the bulk of the 2004-2005 low frequency (SWIPS1) data of Figure 1 apart from the anomalous February-March period and video results (Jasek et al., 2005) which were suggestive of a layer of ice particles extending down to depths at least 1 m below the ice cover, possibly arising from erosion at the adjacent ice undersurface.

The 2005-2006 results, acquired at the higher SWIPS2 acoustic frequency, on the other hand, were characterized by the continuous presence of significant frazil returns with strengths often exceeding those of the strongest pre-stabilization returns, particularly at heights beyond pre-stabilization water levels. This situation is illustrated by the six-hour averaged profiles of Figure 3b plotted for stabilized ice cover periods associated with, respectively, near minimal and near maximal acoustic return strengths. The two other curves in the Figure represent data acquired at times prior to ice formation and following break-up and clearance, respectively. The latter, baseline, responses show zones of strong close-in returns with dimensions (1 m) essentially identical to those observed beneath the mobile ice cover (Figure 3a). The smaller dimensions of such zones in the stabilized ice cover profiles, indicative of compression of the turbulent bottom boundary layer, are a likely consequence of the presence of the additional boundary layer at the stabilized ice cover. The sharp rises at the upper ends of the stabilized ice cover curves encompassed water column heights, roughly, up to 1 m below contemporary river water levels. In the absence of on-ice measurement data, insights on the latter, high-lying, strong scattering regime responsible for these rises were drawn from time series profile depictions such as that of Figure 4 corresponding to returns from two bursts of 60 1s-separated pings emitted 30 minutes apart in a March 17 time interval associated with a local minimum in target strength. The key features of the depiction are the distinctive slopes of returns from both strong targets at heights ≥ 5.5 m as well as from much rarer and weaker targets deeper in the water column. Such slopes arise from the fact that targets which retain physical coherence in a tilted acoustic beam over the
1 s pinging intervals are detected at progressively larger ranges as they move through the beam footprint. These results, thus, suggest that acoustic returns from vertical heights $\leq$ about 7.3 m above the SWIPS transducer were often associated with extended targets moving with velocities comparable to the projected horizontal water velocity component. (Comparisons with independent river speed estimates indicated that the tilted beam azimuth deviated by approximately 41° from the local flow direction.) Returns from still higher levels in the water column were excluded from the Figure to avoid contributions from more slowly moving, non-particulate, portions of the stable ice cover (Jasek et al., 2005).

**Figure 2a.** 235 KHz unit data- Initial low frequency results show weak returns from the water column (depths less than 2.94m) appearing primarily after 07:00 1/12.

**Figure 2b.** 546 kHz unit data- High frequency results show full range of return variability in same intervals associated with frazil ice presence (depths less than 4.8 m).

Unfortunately, use of profile data for extracting the detailed composition of water column ice particle content remains problematic in the absence of multi-frequency data which could separately estimate particle number density and size distribution parameters. Thus, the observed sharp rise in target strength at heights greater than 5.5m could be a consequence of increases in
either or both particle numbers and sizes. An intriguing possibility in this respect arises from the well known (Martin, 1981) tendency of frazil particles to sinter or bond together: increasing buoyancy to drag force ratios and, consequently, relative probabilities for occupying upper portions of the water column. This possibility, alone, because of the sixth power particle diameter dependence of Rayleigh scattering cross-sections, could account for rising target strengths at the upper ends of the stabilized ice cover curves (Figure 3b). Clarifications of this and other frazil composition issues clearly require expanded measurement programs. Guidance for such programs is sought below by using the 2005-2006 data to explore empirical relationships between frazil-related target strengths and relevant environmental factors.

**Figure 3a.** Six-hour averaged target strength (counts) vs, height (m) above the transducer at indicated times. Square- (triangle-) denoted data were obtained with vertical (tilted) beams.

**Figure 3b.** Six-hour averaged SWIPS2 target strength vs. height above the transducer under the indicated conditions. Again, squares (triangles) denote vertical (tilted) beam data.
Figure 4. High temporal resolution SWIPS2 returns from 2 bursts of 60 pings emitted at 1 Hz at opposite ends of a 30 minute March 17, 2006 time interval.

4. Frazil Target Strength beneath a Stabilized Ice Cover: Environmental Dependences

The anticipated absence of frazil ice growth beneath a stable ice cover is consistent with the corresponding cessation of anchor ice-interruptions of SWIPS2 measurements. The measurement continuity was notable relative to earlier, pre-stabilization, behavior and persisted despite the fact that frazil-related target strengths increased to levels comparable to and, often, larger than those of the most intense pre-stabilization frazil episodes. Consequently, key environmental determinants of suspended frazil variability under stabilized ice covers might be expected to act through either generation and export of frazil from adjacent upstream regions and/or through controlling ice cover deterioration rates. The latter possibility could, in principle, produce variations in water column ice content without generating “new” frazil. The obvious candidate parameters for such influences are local and upstream air and water temperatures, river flow velocities and the positions of the ice front. Near-bottom water temperatures measured at the SWIPS1 instrument showed no evidence of either supercooling or warming except at the tail end of the ice-covered period and, hence, were unlikely to be directly linked to the observed variability. Air temperature held greater promise as a forcing factor and values measured at hourly intervals 7 km downstream from the SWIPS site were found to be representative both of that site and areas near and upstream of the documented ice front positions (Figure 5). Water speed estimates were available only indirectly from models and empirical relationships to water levels deduced from SWIPS1 hydrostatic pressure data: making such levels a parameter of potential importance both on its own and as a river speed proxy. Environmental parameter comparisons were made with measures of acoustic target strength expressed in terms of averages over heights in the water column corresponding to mid- and upper-water column layers as defined by, respectively, heights between 2 to 5 m and 5 to 7.3 m. These choices reflect the obvious changes in target strength regime apparent in the under-ice data of Figure 3b.
Figure 5. Locations (in km downstream of a dam reference position) of the 2005-2006 ice fronts and the SWIPS and air temperature measurement sites.

Time series of target strengths in each layer were averaged over height and each half-hourly burst interval and passed through a 4-hour running average filter to allow comparisons with both each other and with corresponding air temperatures and water levels. As expected from Figure 3b, the target strengths in the two layers followed each other closely, corresponding to $r = 0.71$ over the full record and $r = 0.84$ when a 40 hour March 15-16 period of obviously disparate behavior was excluded. Maximum target strength magnitudes bracketed the 103 count average values associated with the most intense pre-stabilization frazil episode depicted in Figure 2b. Comparisons with air temperature and water level data also showed correlations: first, in terms of similarities between the broad peaks in the target strength time series and in corresponding temperature and water level minima; and, then, in higher frequency components of variability.

Explorations of these correspondences were complicated by correlations between the environmental parameters themselves which arose from both the direct influence of the atmosphere on river conditions and from adjustments in the managed river flow in response to weather-related power consumption needs. Flow variations were also connected to ice front position movements which either increased or decreased local flow when, alternatively, receding downstream or advancing upstream of the monitoring site. Our analyses applied additional (to the 4-hour filter) 24 hour running average low pass filtering to the hourly air temperature and half hourly water level and target strength time series, providing measures of variability on timescales longer than the, roughly, diurnal periodicities apparent in most of the data. Corresponding plots (Figure 6) show that a broad mid-March drop in air temperatures preceded similar decreases in target strength and rises in water levels. Over the full record, water levels lagged air temperature by about 55 hours with a correlation of $r = 0.6$. Air temperature/target strength correlations were $r = -0.64$ and $-0.78$ for the mid- and upper-water layers, respectively, with corresponding optimal lags of 0 and 75 hours. Excluding data from the period prior to March 9, associated with anomalous positive correlations, produced even stronger air temperature-target strength correlations ($r$ values of $-0.77$ and $-0.86$ for the mid- and upper-water layers, respectively). Given the limited number of degrees of freedom associated with these data, the statistical significances of these comparisons are low but, particularly after the immediate post-stabilization period, are suggestive of physically reasonable negative correlations between target strengths and slow changes in regional air temperatures occurring 0 to 3 days earlier. Target strength correlations with water levels were also negative and similar in
magnitude and lag times but were of opposite sign (changes in water level lagging those in target
strength). Indirect evidence, presented below, suggests that the latter results reflect water level
linkages to the low frequency air temperature variations which drive similar temporal scale
changes in water column target strength.

High frequency components of variability, as defined by the differences between singly (4-hour)
filtered and doubly (4- and 24-hour) filtered time series data, showed much less ambiguous
linkages of the middle and upper layer target strengths to both each other and to similar temporal
scale water level variations. Such linkages are apparent in the difference time series plotted in
Figure 7 for the water level- and mid- and upper-water layer-target strength parameters.
Corresponding air temperature results, not included in the Figure for clarity purposes, also
showed variations on similar time scales which were, however, devoid of a consistent
relationship to the other plotted parameters. The plots show both the close tracking of the
average target strengths in the adjacent water layers as well as the occurrence of short intervals
when such tracking was absent. Prominent divergences in the behavior of the two layers were
apparent on March 6 and 25 as well as in the March 15-16 interval cited above. As well, the high
frequency target strength variations in both layers can be seen to have been closely coincident
with corresponding components of water level change except, again, in a small number of short
time intervals. Overall, rises and falls in target strength accompanied water level changes of the
same sign with negligible time lags. It is notable that the positive signs of these correlations were
opposed to those deduced above from our low frequency target strength/water level
comparisons, supporting the conclusion that the latter correlations arose from low frequency
linkages of both target strength and water level to air temperature. Over the full record, high
frequency target strength/water level correlations of $r = 0.60$ and $0.50$ relative to water levels
were obtained for the mid-and upper water column layers, respectively. Still larger correlations, $r$
= 0.74 and 0.67, were associated with these same pairings of parameters based upon data
recorded after 13:00, March 17. These results reflect the general impression given by Figures 6
and 7 that the effects of water level changes were most apparent in the mid-water layer, possibly
because of the greater direct linkages between upper layer target strength and air temperature.

Explanations of these results in terms of extraneous effects, such as physical oscillations of the
tilted acoustic beam or flow-induced changes in frazil particle orientation were inconsistent with,
respectively, tilt sensor data and the magnitude of the river speed changes. Moreover, empirical
correlations with high frequency water level changes were found to be absent in data gathered
with both tilted and untilted beams during period of frazil formation prior to stabilization. Since
data from these periods were acquired upstream of contemporary ice fronts, this absence
strongly suggests the high frequency target strength variations later observed under the stable ice
cover were not consequences of frazil growth processes occurring in mobile ice and open water
upstream of the SWIPS2 site. Consequently, the empirical linkages between water levels and
target strengths suggested by the data in Figure 7 were unlikely to have been imposed upstream
of the ice fronts but were of more local origin. On the other hand, the negative, probably time-
lagged, target strength correlations with low frequency air temperature variations do suggest that
more gradual changes in detected target strengths did arise from frazil production variations in
areas upstream of the ice front. In this picture, physical mechanisms are required which allow
high frequency variations in water level or speed to impose similar temporal scale changes upon
frazil concentrations originally produced upstream of contemporary or recent ice fronts.
Figure 6. Time series plots of low frequency variability components in local water level, air temperature and mid- and upper-water column averaged target strengths. The water level changes were scaled as $70 \times$ the difference between the measured water levels and 5.4m.

Figure 7. Time series plots of high frequency components of variability in local water level and mid- and upper-water column average target strengths.

Local processes can be visualized in which water level/speed variations either alter the contents of local deposits or, even, sources of frazil ice and/or control transport of upstream-generated frazil to these features and/or to the local water column. The observed negative correlations with air temperature changes appear to favor such mechanisms over ablation-based mechanisms for introducing ice particles into the water column. The observed sensitivities of target strength to height in the upper water column and to small changes in water level and/or flow speed may be indicative of the delicate balancing of frazil concentrations close to the ice cover against...
dispersal deeper into the water column bulk. Slow movements detected on long (many hour) time scales at the nominal ice undersurface (Jasek et al., 2005) and the slanting profiles of Figure 4 suggest that ice targets near this interface move with speeds comparable to those of the river. Detailed documentation of this boundary regime and quantitative data on frazil particle size and number distributions as functions of height in the water column, river bathymetry and ice front positions are critical to understanding frazil ice variability beneath ice covers.

5. Summary and Conclusions

Field tests have demonstrated the capabilities of a higher frequency SWIPS profiler in monitoring frazil ice in freezing rivers. These tests have shown that, prior to local seasonal ice cover stabilization, frazil presence was confined to time intervals associated with measurable supercooling. In the absence of applied electrical heating, such intervals were plagued by anchor ice accumulations which, usually temporarily, interrupted monitoring. Target strengths at water column levels more than 2 m off the bottom varied negligibly with height. The presence of frazil ice targets was continuous beneath a stabilized ice cover in spite of the absence of supercooling and anchor ice formation. Target strength profiles under these conditions were readily distinguishable from their pre-stabilization counterparts, featuring a shallower zone of sediment-related near-bottom returns and a high-lying strong scattering layer which, presumably, included mobile ice particles at heights extending almost up to the nominal ice cover undersurface. Depth-averaged target strengths in the mid- and upper-water layers rose and fell in close concert and, at long temporal variability scales, showed lagged negative correlations with air temperature. Strongest correspondences were noted between higher frequency variations in target strengths and water-level and/or -speed. Corresponding target strength fluctuations, sometimes equivalent to as much as 50% of mean strength values were well in excess of the accompanying percentage changes in water level or speed parameters. Such linkages were exclusive to the ice covered period, suggestive of origins in processes taking place in or beneath the stabilized ice cover. The observed behaviours appear to be directly relevant to monitoring and modeling ice growth and transport in freezing rivers and, hence, to power generation and flood control issues. Immediate future research needs include: multiple frequency measurements and calibrations to better characterize frazil properties as well as additional field and acoustic studies of the ice cover undersurface and the immediately adjacent water layer where water level and/or speed changes appear to affect water column frazil content.

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Acoustic Detection and Study of Frazil Ice in a Freezing River during the 2007-2008 Winter

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An upward looking sonar instrument was deployed on the riverbed in November 2007. The instrument package contained a 545 KHz sonar transducer, as well as temperature-, 2-axis tilt- and pressure- sensors as well an onboard heater and a warm water supply hose to prevent anchor ice formation. The sonar instrument measures the distance to the water surface or the undersides of drifting or stationary ice. By computing the difference between the acoustically-derived distance to the ice and an independently measured water level, the draft of the floating ice can be determined at one second sampling intervals. The instruments can also record the profile of acoustic backscatter returns through the body of the river water column allowing detection of the presence and depth of suspended frazil ice.

Data quantity and quality from prior deployments since November 2004 of this and a lower frequency unit (235 kHz) have been compromised by anchor ice formation on the instrument. This paper describes a new and a much successful design of a mooring system that prevented anchor ice interference.

The location of the underside of the ice cover and the presence of suspended frazil ice as detected both prior to, during and after the formation of the winter ice cover was recorded by this higher frequency instrument for the first time. Relative concentrations of suspended ice and the concentrations and thicknesses of surface ice pans were measured. The ice cover formation (stabilization and shoving events) were also captured for the first time at continuous 1 second full depth-profiling sampling rate.
1. Introduction
Floating frazil ice pans and suspended individual or aggregated crystals of frazil ice in freshwater bodies often have significant impacts upon water supply, hydroelectric, fisheries and other management activities. Effective detection and quantitative characterization of such ice can provide direct input to operational decision-making and for formulating numerical river ice and flow models (Shen et al., 1995; Shen, 2006) underlying modern flow management (Jasek, 2006).

The results presented here were obtained in freshwater during BC Hydro’s and Alberta Environment’s 2007-2008 winter Peace River monitoring programs utilizing SWIPS (Shallow Water Ice Profiler Sonar) instruments developed by ASL Environmental Sciences Inc. from the company’s IPS5 Ice Profiler platform. The SWIPS is an upward looking sonar instrument that acquires acoustic backscatter data from, roughly, 1.1 cm deep, horizontal, slices of an insonified water column. Suspended particles such as frazil ice crystals, the water surface, bottom of floating frazil ice pans and the water surface can be detected. The location of the monitoring program and the basic instrument set-up and components are depicted in Jasek et al. 2005. The acoustic transducer is deployed on the river bottom and is linked to an instrument shelter on shore via a communications cable for data transfer. A steel mooring cable and a power supply cable for an onboard heater are also connected to the shore.

Historically, interference from anchor ice ranged from intermittent blockage of the SWIPS signal (a few days or weeks), to movement and tilting of the instrument off the vertical axis, to complete overturning of the instrument, and, in the most severe case, loss of the transducer, mooring system and data cable.

2. Mooring system design and deployment to mitigate anchor ice interference
Up until the fall of 2006, all deployed SWIPS were linked by a single cable to the shore on one side of the river and attached to platforms with submerged weights of about 32 kg. This arrangement was not sufficient to prevent instrument misalignment or overturning by anchor ice (Marko et al. 2006). Therefore in the fall of 2006, the total submerged weight of the SWIPS package was increased to about 68 kg and the platform was moored to both banks of the river. The unfortunate design choice of dual attachment points allowed anchor ice to form on about 400 m of steel cable on the river bottom, which consequently floated upward into the water column where drag from water and ice movement was sufficient to rupture cable connections to both shores (Jasek and Marko, 2007). This loss of a SWIPS transducer and mooring system in the fall of 2006 inspired a mooring system redesign and the inclusion of anchor ice mitigation features into the deployment platform.

The 2007 SWIPS platform consisted of four stacked 6 mm (1/4”) steel plates supporting a steel box enclosure (Figure 1). The enclosure housed the SWIPS transducer head as well as a 500 W electrical heater. Heat was preferentially lost from the top panel of the box due to its lower (3mm (1/8”)) thickness relative to the side walls (6 mm (1/4”)) which, together with the floor of the box, were lined with rubber to reduce heat losses.
The box and plate structure was covered with a steel frame supporting 6 mm (1/4”) Teflon sheeting at 45 degree slopes (pyramid structure). Teflon was chosen as it has the lowest known ice adhesion strength of any readily available coating material (Mulherin and Haehnel, 2003). This arrangement reduced the strength of the bonds between anchor ice and the structure and minimized the perpendicular impacts from moving supercooled frazil ice crystals which cause anchor ice accumulation. There was also a ¼” gap between the top steel plate and the uppermost Teflon sheet which allowed warm water generated by the heater to escape through a circular hole cut to allow emergence of the transducer beam. The steel base plates and steel box weighed about 162 kg (submerged weight of 142 kg, which was more than double the weight used in the previous year). It was calculated that such a weight would maintain stability even with attachment to a mass of anchor ice with dimensions as large as 1.5 m x 1.5 m x 1.25 m. The horizontal dimensions of the steel base plates were 0.74 m x 0.89 m. Given that anchor ice was not believed to grow to thicknesses much more than 0.5 m in the Peace River, lifting of the instrument by added buoyancy was judged to be unlikely although it was recognized that movements driven by drag forces on attached anchor ice accumulations were still potentially possible. A 19 mm (¾”) plastic hose was connected from shore to the pyramid interior to facilitate delivery of warm water should the 500 W heater prove to be insufficient to clear anchor ice. The hose was insulated with 9 mm thick flexible pipe insulation (not illustrated in Figure 1). The pyramid enclosure was sealed at the seams and at the top steel plate to retain the warm water in its interior to maximize the length of time available for radiative heating once the warm water supply was turned off. (Warm water would quickly rise out of the unit due to the buoyancy differential if the mid to upper surfaces were not sealed.) The seals were not fully air tight in order to allow air to escape when the unit was lowered into the river. Air and small amounts of warmer water were able to escape through the bolt holes that allowed attachment of the Teflon sheets to the steel frame. This feature was judged to be a useful one as anchor ice which tends to accumulate around the bolt heads and is likely to be dislodged by the leaking warmer water.

There were 4 “feet” attached to the 4 corners of the platform (not included in Figure 1), one is visible in the gravel at the bottom of the photograph. The feet were used as mounting points to lower the platform and have a convex lower surface to be compatible with the original deployment plan which was to slide the unit out along the gravel to the monitoring location.

There were about 5 m of 10 mm (3/8”) diameter steel cable attached to the upstream foot closest to the river bank. This cable (with the same thickness used in the 2006 deployment) was attached to a much thicker 25 m long steel cable (38 mm (1 ½”) dia.) ,with a dry weight of approximately 155 kg which was believed to be sufficient to minimize lifting under anchor ice growth attachments up to 0.4 m in diameter. The 5 m length of thinner cable attached to the platform was employed to keep the amount of weight that was being lowered from the boat to a manageable level during deployment. The near-shore end of the 38 mm dia. cable was attached to another piece of 10 mm dia. steel cable which made the actual shore connection and was anchored to a tree. Approximately 25 to 30 m of this cable was in the water.

The deployment of the SWIPS went rather smoothly (Figure 2) on Oct 21, 2007. A boat-mounted crane lowered the platform in waters approximately 5 m deep and approximately 50 m
from the shore after dragging the heavy 38 mm cable along the river bed. The latter cable acted like an anchor, providing stability for platform placement.

The electrical heater appeared to prevent anchor ice from blocking the acoustic beam until the electrical cable was severed late on Nov 30, 2007. An attempt to send warm water down the insulated hose to allow resumption of acoustic data-taking was frustrated by an apparent kink in the submerged portion of the hose. The greater weight of the platform relative to previous deployments provided physical stability throughout the deployment. There was no indication that the beam ever tilted by a measurable amount from the vertical direction.

Recovery of the unit in the spring went relatively smoothly. A 4x4 truck was used to pull the SWIPS platform and 25 m of the attached 38 mm dia. cable out onto the river shoreline.

2. Suspended frazil intensities, ice pan thicknesses and concentrations prior to the formation of the ice cover - winter 2007-2008

Suspended frazil intensities, ice pan thicknesses and surface ice concentrations have been measured prior to the 2007-2008 ice season (Jasek and Marko, 2007). However, anchor ice interference introduced some uncertainty into measurements after about 24 hours of continuous exposure to supercooling while complete blockage of signals was produced by persistence of about 48 hours of such conditions. A 100 W heater used to mitigate anchor ice formation on the SWIPS platform prior to 2007 proved to be unsatisfactory not only because there was no independent way of ensuring that the heater was functioning but, as well, because the heating was not directly applied to the mooring platform where anchor ice attachment produced dramatic and, sometimes, fatal instabilities. The effects of the internal platform heating during the 2007-2008 season were directly monitored with a temperature sensor attached to the transducer. The data showed that the internal SWIPS temperature stayed at about +9 °C while the heater was activated even during frazil ice runs on the river. In contrast with our 2005-2006 season experiences, there was no evidence of signal degradation while the heater was functioning.

In addition to suspended frazil return strengths, surface ice concentration and average ice pan thicknesses, average ice pan durations were also computed. Such durations can be considered as indicators of relative ice pan size if the river velocity can be assumed to be more or less constant. Figure 3 shows a) acoustic profiles, b) post-processed 5-minute averaged values of surface ice concentration, ice pan thicknesses, and suspended frazil return strengths, and c) ice pan duration and air temperature for the Nov 28-29 period. The strengths of returns from suspended frazil were averaged between 1.37 m and 4.28 m ranges above the SWIPS unit in order to be representative of water column values as opposed to returns from suspended sediments or rifting surface ice which are the dominant features of, respectively, lower and higher portions of the water column.

All ice quantity components started to increase gradually from zero values at about 05:00 hours on Nov 28 (Figures 3b and 3c). Unfortunately, the water temperatures recorded during this season were not as accurate as previously since the utilized joint water level/water temperature sensor was deployed in October, 2007 and attached to the water intake structure to prevent movement by anchor ice. This arrangement was ideal for recording water levels but
unsatisfactory for water temperatures. There also may have been some groundwater influence precluding adjustment of the absolute values obtained to a relative zero value due to longer term changes introduced by decreasing ground water contribution over time near the river bank. Additional problems could have been introduced by short term fluctuations during periods of rapidly changing water temperatures caused by density driven convection currents.

The gradual increase in ice quantities continued for about 10 hours until about 15:00, Nov 28 after which a very sudden increase was noted in all ice quantity components. This change likely coincided with the maximum latent heat recovery in the water temperature in the main channel of the river but this could not be confirmed from water temperature data obtained near the bed closer to shore.

Ice production over the course of Nov 28 and 29 appeared to be inversely correlated with air temperature and was negligible after the air temperature increased to -15 °C in the afternoon of Nov 29. Our confidence in the relative quantities of suspended and surface ice present is high for the Nov 28-29 frazil ice run period as the heater was functional.

3. Heater Failure on November 30, 2007

Unfortunately, the heater failed at about 21:30 hrs on Nov 30 as indicated by the internal SWIPS temperature which dropped from +9 to +1 °C in Figure 4b. Acoustic profiles for the period of Nov 29 to Dec 4 are shown in Figure 4a along with the corresponding air temperatures in Figure 4c. By 02:30 on Dec 2 the SWIPS signal was sufficiently attenuated by anchor ice to entirely eliminate returns from suspended frazil (which according to air temperature data should have still been present). By 08:00 hrs the same morning the SWIPS beam was completely blocked. On Dec 3, the SWIPS signal returned probably due to routine shedding of anchor ice but started to fade rapidly again with resumed anchor ice buildup. The internal SWIPS temperature did increase to about +2 to +3 °C but far shy of the +9 °C attained previously. When the SWIPS unit was recovered in the spring of 2008 the industrial grade extension cord wire lead to the heater was found to be worn through in several locations and severed completely at one location, probably due to anchor ice action. Similar problems were not encountered with the much more abrasion resistant Polyurethane-protected data cable. Plans for a Fall, 2008 deployment include use of a Polyurethane heater cable.

4. Comparison of manually and acoustically measured ice pan thicknesses

Since the winter of 2004-2005 manual ice pan thickness measurements have been collected with the aid of a video camera suspended on an L-shaped graduated boom (Figure 5). The submersible camera is an off-the-shelf recreational fishing camera employing a monitor at the end of an 18 m cable. To obtain frazil ice thickness measurements, one field team member looks into the video monitor and lets the boom operator know when the bottom of the camera is even with the bottom of the ice pan. The boom operator then reads the graduation marker on the vertical portion of the boom and the video monitor viewer then writes down the ice pan thickness value. Normally, 30 readings are taken at a particular location over the course of 15 to 40 minutes. The variations in total measurement time were determined by the surface ice concentration which governed the frequency of passing measurement opportunities.
Such information has been used to calibrate ice models such as CRISSP and PRICE on the Peace River. Attempts have been made since 2004 to obtain coincident SWIPS-derived frazil ice pan thicknesses for comparisons with these measurements: an effort which has, until the past season, been frustrated by anchor ice interference.

Figures 3b and 6 show the first comparisons of measured and SWIPS-derived frazil ice pan thickness data. Since the pan measurements were not conducted directly over the SWIPS instruments, it is more appropriate to make comparisons with manual measurements using the 5-minute averages of the SWIPS data. The two independent types of measurements compared very well on Nov 28 and 29 but the manually measured ice pan thicknesses were significantly greater then the SWIPS derived values on Jan 5. On the latter date, the ice pans were significantly thicker, larger in size and associated with higher surface ice concentrations than previously. One possible reason for the manual/SWIPS discrepancy on this date could have been that outer edges of the ice pans were significantly deeper than the middle portions of the ice pans due to increased frequencies of collisions between the ice pans. Since the camera can only see the edges of the ice pans these edge effects would have introduced a bias toward higher ice pan thicknesses.

Note: The 5-minute average SWIPS derived ice pan thicknesses shown in Figure 6 are calculated by considering ranges that only intercept an ice pan. (i.e. zero ice pan thicknesses are not included in the averaging.)

5. Surface ice quantities on Jan 4 to 10, 2008.

After the heater failure on Nov 29 and before the formation of the stationary ice cover on Jan 11, 2008, confidence in the estimated suspended ice quantities was low and not worthy of further analyses. However, ice pan thicknesses and surface ice concentrations were only affected by the most severe anchor ice SWIPS blockages and useful data for these surface ice quantities were obtained during this period. Figure 7 shows the SWIPS acoustic data for the period of January 4 to 12 and Figure 8 shows the derived 5-minute average surface ice quantities for Jan 4 to 9, just before the stabilization period.

The Jan. 4-6 period was relatively warm (Figure 8b, with air temperatures between -10 and 0 °C) facilitating release of anchor ice from the unit and allowing resumed acquisition of ice draft data in the afternoon of Jan 4. The measured surface ice quantities were still substantial at this time (Surface Ice = 94%, Ice pan thickness = 0.46 m and ice pan durations 10 to 35 seconds) due to a previous cold spell that lasted from Dec 30 to Jan 3. However, by Jan 6 this ice had travelled downstream past the SWIPS and the same measured quantities were down to 40%, 0.32 m and 2 sec respectively. Cold weather (temperatures between -22 and -15 °C) returned on Jan 7 with a sharp increase in the surface ice quantities. By the end of Jan 8, the ice quantities reached more or less equilibrium values of 86%, 0.6 m and 10 sec respectively. The increase in the surface ice quantities indicate that surface concentration and ice pan duration (size) respond quickly to the arrival of cold weather while increases in pan thicknesses occur more gradually. The first two of these changes are likely due to the formation of frazil pan rafts while the slower process of pan thickening may be indicative of a dependence on increasing collisions between rafts and additional frazil coming out of suspension underneath these rafts. This Jan. 4 to10 data set provides one example of the surface frazil evolution process. Combining this with data collected
in the future at different air temperatures and in different river hydraulic regimes may be useful in coming up with a theory of surface ice evolution which includes the ice pan size component usually not included in computer river ice models.

6. Ice stabilization process on Jan 10 to 11, 2008

Figure 9 shows the acoustic profiles, surface ice quantities and water levels during the river stage-up process associated with the arrival of the ice front late on Jan 10. It appears that the river stage-up had significant effects on surface ice. Changes such as the increase of surface ice concentration have been observed previously from the air and are due to the slowing down of velocity in the deeper backwater upstream of an ice front which allows the ice pans to come closer together and start the process of rafting. Evidence for the rafting process was apparent in the observed large increases in ice pan durations. Such increases are the combined product of increased rafting and the slowing of river velocity in the backwater upstream of the ice front.

Other quantities such as ice pan thickness which change with time as the ice front approaches have never been previously quantitatively monitored. From a thermal perspective, one should not expect increasing pan thickness in the backwater as the expanding surface coverage further insulates the water column, preventing additional supercooling and thereby reducing contributions from suspended ice flocculation. On the other hand, higher collision frequencies in the presence of greater surface concentrations should produce larger ice thicknesses. An additional potential agent of ice thickening could arise from reduced turbulence in the slower moving backwater which could allow finer frazil (or even un-flocculated) ice particles to come out of suspension. Figure 9 shows some evidence for this possibility in that frazil pan thickness was the first surface ice quantity to begin to rise following the first arrival of the back water increase at about 04:00, Jan 10. The ice pan thicknesses increased from about 0.6 to 0.7 m prior to a significant change in surface concentrations. Thus, this first 0.1 m increase was likely not a consequence of additional frazil pan collisions but reflected frazil ice coming out of suspension due to the lowered backwater turbulence levels. This addition of 0.1 m of frazil removed from the 4.4 m deep water column depth implies an initial concentration of frazil on the order of 0.7% (assuming a frazil pan porosity of 0.7) in accord with estimates suggested by CRISSP modeling on the Peace River (Jasek, 2008). There is also video data recorded during the stage-up period which could allow velocity estimation. Figure 10b shows suspended ice intensities or return strengths which show significant reductions as the backwater increases. Some of the variability prior to the backwater may be attributable to variations in anchor ice blockage of the SWIPS acoustic signal. However, supercooling should no longer be occurring in the Jan 9 – 10 period due to the additional insulation from the atmosphere which accompanied higher surface ice concentrations. Consequently, most or even all anchor ice should have released at the start of this period. Some of the sharp increases in suspended ice on Jan 7 – 8 may have been due to an anchor ice release as there are some coincident sudden changes in the (red) intensity near the bottom line in Figure 10a.

Another noteworthy feature of the backwater period data is the significant increase in rafting implied by the ice pan duration estimates. In particular, there was a sudden increase in thickness from 0.8 to 1.2 m (Jan 10, 16:00hrs) shortly after the first sustained 100% ice concentration...
coverage (Jan 10, 15:00hrs), (Figure 9b). This event was the effective start of the consolidation (thickening) process.

Figure 10 shows the final stabilization processes at the SWIPS site. The ice first stabilized on Jan 11 at about 00:00hrs and was about 1.4 m thick. The ice cover remained stable overnight but at about 12:00 hrs the ice cover shoved again and thickened to 4 m. Higher resolution acoustic profiles of this event are shown in Figure 11. Figure 11b shows that about 45 seconds prior to the remobilization of the ice cover, the suspended ice content increased significantly. Once the moving surface ice run arrived this suspended ice disappeared and then reappeared for a few minutes following the last stabilization (Figure 11c) before disappearing again 10 minutes after the ice cover re-stabilized. This behavior is consistent with the relative velocity (or shear) between the ice cover and water during these highly dynamic events.

After the consolidation event on Jan 11, the river ice cover stabilized as depicted in Figure 12. It was evident that rougher ice occupied the left third of the channel where the SWIPS was located. This was confirmed by surveys (Figure 13) and was a result of the consolidation process as the right two-thirds of the channel continued to run for some time after the left third stabilized. The right two-thirds then formed a relatively smoother cover consisting of juxtaposed frazil ice pans. This was likely the result of low surface ice velocities in the back water upstream of the consolidation that was now about 1.5 m higher than a normal freeze-up water level. Unfortunately, this consolidation also caused high water levels 4 km downstream that resulted in seepage into some basements in the Town of Peace River over the duration of the winter.

7. Suspended frazil intensities and deposition/erosion of the underside of the ice cover, Jan - Mar, 2008.

Figure 14 shows the elevation of the bottom of the ice and the water surface during the stable ice-covered period between Jan 11 and Mar 28. There was a rapid deposition of frazil on Jan 16 a more gradual deposition from Jan 16 to 20 (Figure 15). This may had been due to the atypical low velocities at the relatively isolated SWIPS location (due to the rough ice at the SWIPS site) which allowed rapid frazil deposition from flow entering this now isolated area of the main channel. The other alternative is erosion of ice from upstream that arrived suddenly such as a large frazil “bedform” originating from thick ice upstream of the SWIPS (Figure 12). On Jan 20 the bottom of the stabilized ice cover reached its minimum elevation of the winter, only 1.2 m above the SWIPS. Figure 15 does show increased frazil in suspension that is coincident with the depositional rate. It is somewhat of a mystery what triggered this sudden deposition but was most likely just a local phenomenon as such a deposition river-wide would have caused significant water level increases which were not observed during the Jan 16 to 20 period. The river discharge at this time was constant as releases from Peace Canyon Dam under agreement between Alberta Environment and British Columbia were steady as not to contribute to possible ice cover disruptions or consolidations.

On Jan 26 the thermal ice cover was deemed competent enough to resist any further secondary consolidations and BC Hydro was allowed to increase flows. This was not an easy decision as river level increases could exasperate the seepage into basements. However, it was reasoned that although there would be temporary increases in water levels, the higher flows would erode the frazil ice and increase the conveyance capacity of the river channel and eventually decrease the
water levels. This was especially critical to realize such a result prior to the potential dynamic break-up of the Peace River triggered by the Smoky River which was anticipated in April. A series of weekly pulses of high flows were conducted through the month of February. Figure 14 does show that the bottom of the ice did erode at this site during this period.

From Feb 3 to a week before break-up on Mar 29, Figure 14 shows that the ice cover eroded by about 2 m at the SWIPS location and the water level had dropped by about 1 m. It appears that the weekly pulsing flows eventually aided in reducing water levels. However, further analyses need to be carried out to discern the thinning effects of individual discharge pulses.


8.1 Past Results
The preceding paper showed that, in 2005-6, frazil-related returns from the water column after ice cover stabilization rose from initial very low levels before declining to low levels again just before breakup. This pattern resembled less frazil-sensitive 2004-5 SWIPS observations. 2005-6 connections between return strength and environmental factors were most apparent at diurnal and higher frequencies and, then, with respect to the speed of the river which controlled water levels (Figure 16). At times near peak returns, well over 50% of the high frequency variance was accounted for by water level/speed variations.

The high sensitivity of return strength to small (< 5%) water level/speed changes was tentatively linked to a critical speed sensitivity in the rates of transfer into the water column of ice particles moving along or resident at ice cover undersurface.

8.2 2007-2008 Results
Previous observations of depth independence and 2008 SWIPS positioning in shallower water and thicker ice forced focus in our studies on return strength variations in a single layer 0.9 to 2.0 m above transducer. Processing used averaging over this layer and low pass filtering based on 4- and 24-hour running averaging as in earlier analyses. Data are presented here for the middle and latter portions of the ice covered season associated with our most intense monitoring efforts. Comparisons of 24-hour filtered return strengths with similarly processed water level and air temperature series showed, as in 2005-6, detectable, but inconsistent, linkages on time scales longer than diurnal (Figure 17).

As before, more definitive linkages were observed between the diurnal and shorter term return strength changes (= differences between the 4- and 24-hour filtered series) and corresponding water level series.

Correspondences are again strong, with the larger sudden drops (rises) in water levels tending to line up with decreases (increases) in high frequency return strength (Figure 3). However, smaller diurnal return strength variations are also apparent which have no counterparts in the water level record but can be seen (Figure 19) to be linked to the suitably scaled (by subtraction of 24 °C for visual clarity) high frequency air temperature series. The diurnal components of the 2 series are
closely aligned with negligible lag. The relationship with air temperature was not detected in the 2005-6 data due the obscuring effects of the more diurnal character of that year’s variations in the more influential water level/speed parameter. It should be noted that equivalently good matchups were obtained between the return strength and solar radiation intensity series. However while the noted linkages to air temperature and water level were found to persist even in the periods of very low return strengths immediately following stabilization and immediately preceding breakup, a similar persistence of connections to solar radiation was not observed. Tentatively, we would conclude that the two strongest empirical linkages connect suspended frazil returns to (strongly) high frequency variations in water level/speed and (more weakly), to similar variations in air temperature.

Moreover, these linkages are such that increases in both water level/speeds AND air temperature INCREASE suspended frazil content. In the first case, such a dependence is consistent with expectations from a suspended sediment-like model of frazil content variability. The air temperature result is counter-intuitive for mechanisms based upon contemporary frazil growth (for example, in upstream open water or at the ice cover). In fact, the observed temperature correlations provide a strong argument for the locality of the processes that drive water column frazil variability. It is hard to visualize alternatives such as upstream air temperature sensitive processes which maintain synchronicity between air temperature and acoustic return strengths at our particular downstream monitoring site.

In our view, these results support development of models of frazil variability along the line of those applied to suspended sediment transport whereby, in this case, rapid changes in water flow and, to a lesser extent, air temperature enhance suspension and movement of ice particles from an adjacent reservoir. The critical velocity concept by which increasing flow rates above a threshold produces observed disproportionate increases in water column frazil concentrations arises naturally. The challenging complication is that the properties of the postulated reservoir must vary drastically over the course of the ice covered season to account for observed changes in the apparent effectiveness of the water level/speed and air temperature-dependent suspension mechanisms over the lifetime of a stabilized ice cover. Specifically, three years worth of observations have detected only low levels of frazil target concentrations in periods near the beginning or the end of the ice covered season, irrespective of contemporary high frequency variability in the relevant environmental parameters. Furthermore, the 2004-5 acoustic penetration studies of the only viable candidate as a local frazil reservoir, the slush layer at the bottom of the ice cover, have shown penetration to be negligible after stabilization but that it slowly grows in time to its maximum thickness before slowly thinning again and virtually disappearing a few hours before breakup. This trend in penetration thickness is identical to that which would be required of the availability of frazil in the reservoir of a suspension-based frazil model such as that outlined above.

At least two important issues remain to be addressed involving the mechanisms which:
1) Govern the postulated seasonal changes in the reservoir; and
2) Convert diurnal and more rapid air temperature change signals into essentially un-lagged changes in suspended frazil.
In the first case, continued accumulation of frazil drifting downstream ice from upstream open water growth areas and, then, as temperatures rise and such growth ceases, progressive erosion could account for most of the seasonal cycle leaving only the initial “emptiness” of the postulated reservoir as the puzzle. Two explanations of the latter feature appear to be worth examining. One of these attributes the initial dearth of a mobile slush layer to the sudden change in the relative motion between the river water and the drifting ice floes which become stabilized into the new ice cover. Potentially mobile ice in the lower portions of these surfaces is exposed to drastically increased shears which establish a new equilibrium characterized by greatly reduced frazil particle availability which only slowly grows again over time by accumulation from upstream portions of the water column and ice cover. The second possibility recognizes that the newly stabilized ice cover was largely formed out of flocculated frazil particles large enough to overcome turbulence and rise to the surface to create frazil ice pans. While initially strong to resist erosion by the moving river water, weakening of the bonds in the interlocking lattice of these particles begins to occur over time with the increasing physical separation from supercooled upstream water which accompanies continued seasonal advance of the ice front. In both instances, the rate of recovery should be sensitive to annual variations in ice front advance/retreat and overall air temperature trends. Such variability has been observed but has not yet been examined in detail.

The anomalous coupling of air temperature to water column frazil poses a greater puzzle but similarly almost certainly involves the largely unknown but active dynamics of the slush layer. Substantial slow movements of acoustic targets in this layer have been noted in a previous report on the Peace River studies Jasek et al., 2005), largely based upon the same SWIPS1 penetrations studies cited above. The character of this layer is depicted (Figure 20) in 36 days of 3-hour averaged profile data from the Jan. 21- Feb. 25, 2005 period depicted below along with the modeled (and verified) position of the bottom of the thermal ice layer. An abundance of coherent, diurnal and other higher frequency, structures are detectable along with evidence (at the extreme right) that the thinning of this penetrated layer coincides with the sudden appearances of detectable water column frazil (even at the low SWIPS1 acoustic frequency). It is particularly notable that Feb. 19 video observations from the ice cover reported qualitatively larger numbers of ice particles in the upper 1 m of the water column relative to March 2 observations when water column frazil was detected acoustically with intensities similar to those evident in the Figure at the end of the displayed period. The failure to observe water column acoustic returns at the time of the Feb. 19 observations suggests that the acoustic visibility of frazil on March 2 (and, presumably on Feb. 24-25) was due to the larger sizes of the suspended particles (scattering cross-sections are proportional to the 6th power of particle diameter). The implication here is that erosion of slush layer proceeds with progressive increases over time in the size of particles put into suspension and that this trend coincides with reduced acoustic penetrations of the slush reservoir layer. Understandings of these changes and, more generally, of the dynamics of the slush layer would appear to be essential in making further progress toward quantitative models of water column frazil variability.

Such progress toward is likely to be assisted in the near future by studies underway in Canada involving both laboratory calibration and continued field measurements employing simultaneous measurements at two or more acoustic frequencies. Results from this work should facilitate
conversion of target strength data into the particle concentration and size information needed for quantitative process modelling.


Figure 21 shows the acoustic profiles during the thermal break-up on Mar 28-29, 2008 and Figure 22 shows the underside of the ice cover, water level and water temperature at the SWIPS site. It is evident that there was rapid erosion of frazil ice on Mar 28, about 3 m in 6 hours. This was coincident with a dramatic increase in suspended ice targets and positive water temperatures which begs the question as to whether the frazil ice was being, alternatively, eroded or melted away. It seems most likely that both processes make important contributions to frazil dissipation. In any case, the data do suggest that increases in heat input to the water column can help mobilize frazil transport in accord with the implications of the mid-winter data discussed in the previous section.

After the effective disappearance of the suspended frazil layer at about 15:00 hrs on Mar 28, the thermal cover remained in place until complete breakup occurred at about 14:00hrs on Mar 29. The solid ice thinned only marginally during this time is compared with the previous thinning of the frazil layer. This could be taken as evidence that the preceding frazil depletion event was primarily driven by hydraulic erosion or alternatively a there is a large disparity between the ice-water transfer coefficients descriptive of heat exchanges between the water column and the bottoms of, alternatively, the slush layer and the thermal ice cover.

Figure 23 shows the full time resolution acoustic returns of the thermal break-up of the thermal ice over the SWIPS location. Notable is the apparent rubble ice at the head of the ice cover that came to within almost 1 metre of the SWIPS unit. The apparent weak returns of the water surface following break-up are thought to be caused by absorbed acoustic signal by high bed-load sediments just following break-up.

10. Conclusions

The larger mass of the SWIPS platform deployed in the Fall of 2007 compared to previous deployments facilitated keeping the SWIPS in its deployed position and orientation throughout the 2007-2008 ice season despite anchor ice adherence. Based on previous submerged deployment masses, it is likely that a submerged weight somewhere between 68 and 142 kg is optimal for this purpose. However, it is unclear what role the sloped Teflon surface plays in reducing the necessary platform mass.

It is also likely that the increased mass of the steel mooring cable, 6.2 kg/m compared to 0.7 kg/m used in previous years also contributed significantly to resisting platform movement.

The use of warm water for removing anchor ice from the SWIPS platform poses a difficult challenge since the delivery hose can easily get kinked in a river environment.

The use of the 500W heater appeared to be successful in preventing anchor ice from blocking the SWIPS acoustic beam. However, the utilized standard electrical cord was insufficiently abrasive.
resistant to last throughout the winter. It is therefore recommended that a polyurethane coated cable (similar to the data cable) be used for the heater cable in the future.

With the arrival of supercooling events, the SWIPS data showed all measures of suspended ice and surface ice quantities rose in a few hours from near-zero values to equilibrium values. These quantities then varied inversely with air temperature as expected. The exact timing and value of the supercooling was not measured accurately during the 2007-2008 deployment as the water temperature sensor was mounted too close to the river shore. It is recommended that a water temperature sensor be mounted on the SWIPS unit or further out into the river for next year’s deployment.

SWIPS derived average ice pan thicknesses and manually (camera) measured ice pan thicknesses compared well for thinner (0.1 to 0.2 m) ice pan thicknesses and medium ice concentrations (40 to 60%). For higher ice pan thicknesses (0.4 to 0.6 m) and higher ice pan concentrations (70%) the manually measured thicknesses were significantly greater than SWIPS derived values. This was likely due to the downturned edges of the ice pans resulting from more frequent collisions. This deformation was not discernable from the horizontal perspective of the camera which lacked the vertical perspective of the SWIPS beam. The SWIPS derived ice thickness values are therefore deemed to be more accurate than the camera measured values for higher ice pan concentrations.

Frazil ice pan evolution was quantitatively monitored successfully by the SWIPS. The measurements indicated that surface ice concentration, frazil ice pan thickness and size (approximated by duration) stayed constant when the air temperature remained relatively constant. A temporal equilibrium was reached in which these quantities changed primarily with longitudinal river distance but stayed constant at the SWIPS location. When the air temperature cooled from one temperature regime to another, the surface ice quantities increased at different rates. The increase in the surface ice quantities indicate that surface concentration and ice pan duration (size) respond quickly (one to two hours) to the arrival of cold weather while increases in pan thicknesses occur more gradually (over the course of about a day). The first two of these changes are likely due to the formation of frazil pan rafts while the slower process of pan thickening may be indicative of a dependence on the frequency of collisions between rafts and upon additional frazil coming out of suspension beneath these rafts. Ice pan rafting appeared to suddenly occur when the surface ice concentration was greater than about 60 or 70%.

Increases in surface ice concentration, ice pan size (by rafting) was measured by the SWIPS in the slower moving and rising backwater caused by an approaching ice front. Several mechanisms of ice pan thickening in the ice front backwater could be deduced from the SWIPS data. The first increase in ice pan thickness preceded the increase in surface ice concentration indicating that perhaps suspended frazil ice was coming out of suspension due to the slower moving and less turbulent backwater reach. Subsequent and slightly more rapid thickening occurred with increasing surface ice concentration indicating that frazil ice pan collisions were responsible. Finally, very rapid thickening occurred when the surface ice concentration approached and reached 100%, indicative of thickening caused by shoving of the bank to bank ice run.
The SWIPS successfully quantified the ice cover formation process in that it recorded the initial thickness of the stable ice cover and tracked a subsequent shoving event that thickened it further. Re-suspension of frazil ice during the shoving event was consistent with relative velocity (or shear) between the water and ice layers during the highly dynamic event.

Post-stabilization ice cover thickness was recorded by the SWIPS over the entire ice season and showed a rapid thickening event shortly after ice cover stabilization followed by a gradual thinning (erosion) over the remainder of the winter. The sudden rapid thickening occurred about 5 days after ice cover formation and was more than likely a local phenomenon as there was not a corresponding water level increase and no known accompanying triggering mechanism. These results do suggest that sudden and unpredictable local changes in frazil thickness can occur which may be important for water intake considerations.

Following the initial rapid thickening, the frazil slush layer thinned over the course of the winter and appeared to be thinned more rapidly over the period of discharge fluctuations (frazil flushing operations) aimed at increasing the conveyance capacity of the river channel in order to relieve seepage flooding of basements in the Town of Peace River. Further analysis needs to be carried out however to discern the effect on thinning by each discharge pulse.

Data on the mean return strengths from the water column during the stabilized ice cover period showed predominant dependences on changes in water level/speed and air temperature which occur on diurnal and faster time scales. By far, the closest linkages of return strength were to water levels and speed but the air temperature linkage was also clearly evident and in both cases the connections were local in the sense that they showed no evidence of the time lags one would expect if they represented responses to changes in, for example, distant upstream open water areas. Descriptions of the observed changes were consistent with a mechanism in which increases in water speed and air temperature facilitate (increase) movements of ice particles from a reservoir in the slush portion of the lower ice cover much in the way that increases in river flow raise concentrations of suspended sediments above a silted river bed. Compatibility of this picture with the observed changes in return strengths throughout the season requires additional changes in the nature of the lower portion of the ice cover throughout the stabilized ice cover period. Such changes are consistent with those observed in SWIPS measurements made at acoustic frequencies low enough to penetrate and produce returns from this portion of the ice cover. Detailed quantitative matching of the data with such models will almost certainly require simultaneous measurements and calibrations at multiple acoustic frequencies and the conversion of corresponding return strengths into particle size distribution and concentration characterizations.

The SWIPS recorded the thermal break-up of the Peace River. The eventual break-up of the ice cover over the SWIPS site was preceded by about 23 hours by the rapid erosion of 3 metres of slush. The erosion was coincident with the rise of water temperature that made its way under the ice from the nearby and approaching ice front from upstream. It was unclear how much of this rapid erosion was due to melting and how much was due to mechanical erosion.
It may be beneficial for the future to include a current profiler to be deployed at the SWIPS site in order to expand on the data analysis of frazil pan sizes, frazil ice erosion/deposition and transport throughout the winter and at break-up.

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Figure 1. SWIPS housing in process of being assembled on site.

Figure 2. Deployment of SWIPS on October 21, 2007.
Figure 3. a) Acoustic Profiles (The data interruption at about 10:00 hrs on Nov 29 is due to a data download) b) 5 minute averaged ice quantities, c) ice pan duration and air temperatures.
Figure 4. a) Acoustic Profiles b) internal SWIPS temperature and c) air temperature from Nov 29 to Dec 4.
Figure 5. Frazil Ice Pan Measurements on Jan 12, 2005, photo by Dan Healy, nhc.

Figure 6. Comparison of SWIPS derived ice pan thicknesses and measured ice pan thicknesses on a) Nov 28, 2007, b) Nov 29, 2007 and c) Jan 5, 2008.
Figure 7. SWIPS acoustic data, Jan 4 to 12, 2008. The composite plot had to be pieced together from two separate data sets since the data collection range was increased on Jan 7 in anticipation for the river stage-up associated with ice cover formation.
Figure 8. a) Five-minute averaged surface ice concentrations, frazil ice pan thicknesses and durations, b) air temperatures and water level.
Figure 9. a) Acoustic profiles, b) five-minute average surface ice concentration, frazil ice pan thicknesses and durations, and water levels Jan 9 – 10, 2008. The average air temperature during this period was about -16 °C and ranged between -19 °C and -13 °C.
Figure 10. a) Acoustic profiles and b) water level, surface targets and suspended frazil intensities during the ice stabilization process.
Figure 11. Acoustic Profiles during final consolidation event on January 11, 2008.
Figure 12. Photograph of Peace River at SWIPS location on Jan 12, 2009 looking downstream. Red marks indicate ice cross section measurements and SWIPS location.

Figure 13. Ice cross sections at the SWIPS location on Jan 30 and Feb 14, 2008. River bed cross section is from a nearby cross section 1.6 km upstream.
Figure 14. Water level from pressure transducer and bottom of ice elevation for the entire 2008 ice-covered season.

Figure 15. SWIPS acoustic profiles showing the Jan 15 to 20 depositional event.
Figure 16. Time series plots of 2005-2006 high frequency components of variability in local water level and mid- and upper-water column average return strengths.

Figure 17. Plots of 24 hr-running averaged time series of return strengths (RS24) in the layer 0.9 to 2.0 m above the transducer, local air temperature (T24) and water levels (scaled WL24) calculated from the Solinst hydrostatic pressure sensor on the deployed SWIPS platform. For convenience in the plotting, the water level data have been scaled such that scaled water level = 8 × (water level-317) in m.
Figure 18. Differences in the 4- and 24-hr running average time series as computed for the mid-water layer return strength (RS(4-24)) and water levels (WL(4-24)) derived from Solinst hydrostatic pressure data gathered on the SWIPS instrument.

Figure 19. Differences in the 4- and 24-hr running average time series as computed for the mid-water layer return strength (RS(4-24)) and a local air temperature (T(4-24)) which is offset by subtraction of 24°C. The offset was introduced only to allow easy comparisons of the peaks in the two series. The temperature data were gathered in the Town of Peace River, 7 km downstream of the SWIPS site.
Figure 20. A blow up of the Jan 21- Feb 25 portion of 3-hour averaged SWIPS1 plots presented originally in Jasek et al. (2005) and in (Marko and Jasek, 2008). The plotting is focused on returns from the lower portion of the stabilized ice cover. The yellow curve and added field measurement point represents the estimated bottom surface of the thermal ice cover. The extent of acoustic penetration of the slush layer is probably less than that represented because of the anomalously low speeds of sound propagation noted by Jasek et al. (2005).
Figure 21. Acoustic profiles during the thermal break-up on Mar 28-29, 2008.

Figure 22. Water Temperature and the elevations of water level and the bottom of the ice cover at the SWIPS site during the thermal break-up on Mar 28-29, 2008.
Figure 23. Full resolution acoustic returns of thermal break-up of thermal ice.
Surface Ice Observations on the St. Lawrence River

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This paper characterizes surface ice in a tidal reach near Quebec City, Canada. Based on 3600 rectified images, surface floe dimension statistics are presented. A few kilometers downstream of the site, where the St. Lawrence Estuary is very wide, floe diameters can reach several hundred meters. At this site, the typical diameter is similar to that measured at a site some 130 km upstream. The conclusion is that floe diameter is strongly dependent on channel dimensions. Based on an analysis of 116,000 data, the ice thickness histogram of the surface floes is also presented. Thickness was found to be related not only to recent air temperatures but also significantly related to solar radiation. Surface ice discharge is also calculated. It is shown to be highly variable in both upstream and downstream directions. This is consistent with variations in maximum ebb and flood tide water discharge that is easily six times the net (river) flow rate. Due to the presence of secondary currents, significant hysteresis was present in the tidal ice discharge. Hence, the net ice discharge could not be ascertained. Finally, the use of a laser to measure ice accumulation on tidal flats is presented and discussed.
1. Introduction

In the winter of 2005-2006, instruments to characterize ice were deployed in the St. Lawrence River, near Quebec City, Canada, 215 km downstream of Montréal (figure 1). The original aim of the study was to assess the causes of recurrent frazil ice blockage of one of Quebec City’s water intakes in the St. Lawrence River with an emphasis on frazil ice dynamics (Richard and Morse, 2008 and Morse and Richard, 2008).

This paper focuses on surface ice characteristics, including brash ice, surface frazil floes, skim ice and tidal flat ice. As a complement, surface ice characteristics are compared to those encountered at Lake Saint-Pierre during 2000-2003 winters (Morse et al., 2003a). The paper is organized as follow: section 2 presents information on the instrumented site and on instruments used to obtain field data related to surface ice; section 3 presents the results and an analysis; section 4 presents a comparison with Lake Saint-Pierre data; section 5 is a discussion and section 6 is the conclusion.

2. Study Site and Instrumentation

During the 2005-2006 winter, to understand frazil ice blockage of a water intake, instruments were deployed near the pumping station and water intake. To complement the 2005-2006 observations, a comprehensive field campaign was undertaken throughout the 2007-2008 winter. Multiple on-shore instruments, including still cameras, lasers, marine radar, a meteorological station, as well as submerged equipment, including thermistors, YSI multi-parameter probe, Optical Backscatter Sensor, Ice Profiling Sonar, were used during both studies. As this paper focuses on surface ice, only related instruments will be presented here. As 2007-2008 results are still fresh, they will not be discussed excepting for data provided regarding tidal flat ice. A succinct description of deployed surface ice-related instruments is presented here. Readers interested in more detailed description of other aspects and findings of the study site, as well as the instrumentation used, are referred to the referenced studies published by the authors.

2.1 Study Site

The instrumented site is located in vicinity of Quebec City, Canada. On-shore instruments were installed on the pumping station roof. Submerged instruments were deployed on the St. Lawrence River’s bed at approximately 450 m from the north shore bank. This reach of the St. Lawrence River is subject to a mean tidal range of approximately five meters; water currents regularly change directions at each semi-diurnal tidal cycle. Figure 1 shows the study site location within the Quebec City area. This figure is an ice chart created by Environment Canada (EC) and published on the Canadian Coast Guard (CCG) web site.
2.2 IPS4

The main instrument used for this study was the IPS4 (Upward-looking Ice Profiling Sonar). The latter was deployed on the river bed. An acoustic pulse is sent by a single and quite narrow (1.8°) beam, which has the advantage of precise detection of the interfaces ranges. To record the entire water column profile, the ping rate was kept lower than usual studies, which might only focus on the ice/water-air/water interface echo return, in order to have sufficient internal memory storage. The maximum ping rate in this study was set to one ping per minute.

2.3 Still Camera

On the pumping station roof, a still camera with a 300 mm telephoto lens was installed. As the submerged instruments were located approximately 450 meters from the north shore, the camera was focused approximately on the instrument location. A pulse sent by a data logger acquisition system mounted next to it activated the camera. Every two hours, over the entire winter, three photos were taken 15 seconds apart.

2.4 Meteorological Station

Also mounted on the station roof was a meteorological station, used to obtain local measurements of air temperature, wind velocity and direction, solar insulation and relative humidity. All local parameters were measured every 30 minutes using a data logger to control the acquisition and storage of the data.
2.5 Lasers Rangefinder

During the 2007-2008 winter, a laser rangefinder was mounted on the pumping station roof. It was aimed near the shore to obtain information on tidal flat ice. The laser was activated by a data logger that sent a pulse every second. The data were recorded on a USB stick connected directly to the laser.

3. Results and Analysis

This section presents the results, computational techniques and analysis of all the collected data relative to surface ice. Note that, except for section 3.10 on tidal flat ice, all data are from the 2005-2006 winter.

3.1 Local Water Depths and Water Levels

Local water depths were obtained using the IPS4 pressure transducer after correcting for atmospheric pressure (Marko and Fissel, 2006). In the present analysis, water depths are required to estimate ice floe (keel) thicknesses. Complete winter time series of these data are presented in figure 6 of Richard and Morse (2008).

3.2 Local Mean Water Currents

No on-site instrument was available to measure water current. However, mean channel velocities were estimated by numerical simulation of the St. Lawrence Estuary with the Canadian Department of Fisheries and Oceans’ 3-D model (Saucier et al., 2003) and results were confirmed by the simulation of the St. Lawrence River using a well established and calibrated 1-D model (Lefaivre et al., 2008). The resulting current velocities ($U$) were corrected to account for instruments having been placed near the shore rather than the center of the channel. Flood tide velocities are generally higher (2.2 m/s) than the ebb tide velocities (1.6 m/s).

3.3 Air Temperature, Winds and Solar Insulation

Select meteorological parameters, air temperature ($T_c$), wind direction and velocity ($U_w$) and solar insulation ($I$), were recorded at the on-site weather station for most of the winter. Air temperature reached a winter low of -22°C, while the mean value was about -7°C. There were a total of 6 thaws between mid-November and the end of March, with the longest lasting less than 5 days. Total AFDD for the winter was approximately 672°C-days.

Wind blew in the downstream direction 60% of the time. Mean velocity in this direction was approximately 1.5 m/s. When winds were blowing in the upstream direction, mean velocity was much higher at approximately 2.7 m/s. Note that wind direction ($U_w$) in this paper is that mapped upon the channel direction at the study site. Positive values stand for downstream and negative values for upstream direction. Also, the time series of mean solar insulation, of the last 3 days, is presented in the fourth subplot of figure 5 and will be discussed below.
3.4 Surface Ice Floe Velocities

Using images collected by the still camera, mounted on the pumping station roof, surface ice floe velocity were computed. As the camera was well and solidly mounted, it is assumed that it hadn’t moved throughout the winter, which was later confirmed by visual inspection of the mounting apparatus. Figure 2 is an example of an image taken by the camera in which ice and water are easily distinguishable.

Figure 2. Gray scale example of an image taken by the still 300 mm lens camera on December 22\textsuperscript{nd}, 12:00 PM.

To obtain quantitative data from the images, they were rectified and geo-referenced. After accounting for all the uncertainty within the spatial rectification and its assumptions, it is thought that estimates of ice floe velocity values obtained are probably accurate to within $\pm 15\%$.

3.5 Ice Types and Surface Ice Concentrations

The images also offer qualitative descriptions of surface ice types present at the site under different conditions. There were two significant ice types: ice floes and locally formed skim ice commonly referred to as nilas. To quantify the amount of ice seen on the images, a simple GUI was developed in the MATLAB™ environment by the authors, allowing the user to evaluate the surface ice concentration ($C_v$) by selecting thresholds of gray that separated ice from water. Given ever-changing conditions of luminosity and reflectivity depending on time of day and weather conditions, full automation was impossible. Therefore, manual careful determination of the thresholds for each individual image were established for the two significant ice types present (frazil floes and thin sheets of nilas). Using these values, ice (nilas and frazil) concentration were computed.
It was observed that ice concentration depended on tidal state. In general, the flood tide brought in a much more ice; with a concentration and thickness substantially greater than ebb tide when the floes were less concentrated and ‘younger’, thus, less thick. This significant difference was due to the hysteresis in the water currents as they originated from upstream, after going around a significant rock outcrop known as Pointe Fortin, and from downstream, i.e., the Quebec Bride region.

3.6 Effect of Wind and Surface Ice Concentrations on Surface Floe Movement

Figure 3 presents both surface ice velocities \((U_i)\), with approximate depth-averaged water velocities \((U)\) and wind velocity \((U_w)\), and surface ice floe concentration \((C_S)\) over two 36 hours periods. Figure 3(a) shows that when wind velocities are low, ice floes usually follow the mean surface water current.

However, floe velocity is influenced by both winds and high ice concentration. The effect of wind is presented in Morse and Richard 2008. The effect of ice concentration is presented in Figure 3 (b), where, even if low winds blow in the same direction as water currents, supposedly helping downstream flow, the ice velocities are significantly reduced due to the rubbing of highly concentrated ice against shore (fast) tidal flat ice. The third subplot shows surface ice concentration. Note the difference between 3(a) and 3(b).

**Figure 3.** Apparent reduction of surface ice velocity \((U_i)\) relative to mean water velocity \((U)\) caused by surface ice congestion. Dates are shown as day/month/year. Negative velocities stand for upstream direction; \(U_w\) is the wind velocity mapped in the direction of the channel; \(C_S\) is the computed surface ice floe concentration.
3.7 Surface Ice Keel’s Thickness

By using the IPS4 detected upper interface and knowing water depths, one can estimate the keel thickness of the floating ice floes. The instrument usually returns very clear amplitude signals that precisely determine the air/water-ice/water interface.

In general, ice thickness was inferior to 50 cm at the beginning of the winter season when all the water had not necessarily reached the point of fusion - prior to mid-December. After this initial period, it increased, reaching a local high of 200 cm on January 8th 2006 when air temperatures dropped to a low of -16°C. Values for surface floe keel thickness over the winter normally ranged between 25 cm to 150 cm. Warmer periods could cause ice to almost completely disappear. Cold spells could quickly build up ice floes. For example, by late-February, thicknesses reached a high of approximately 300 cm during a cold spell that lasted about 6 days. Figure 4 shows the distribution of the instantaneous surface ice floe keel. The dashed-dotted bold line represents the fit equation ($R^2=0.99$) $f = ae^{bt} + ce^{dt}$ in which $f$ is the predicted fraction of data points, $t$ is the keel thickness and $a$, $b$, $c$ and $d$ are coefficients with values respectively of 16.220, -0.086, 6.623 and -0.001.

![Figure 4](image)

**Figure 4.** Distribution of the instantaneous surface floe keel thickness.

As shown in Morse and Richard, 2008 (fig. 6), there is a very weak correlation between air temperature and ice thickness only. Thus, air temperature alone cannot adequately explain the ice thickness variations at this site. As discussed above, the hysteresis in the currents, due to channel configuration and changing wind directions, masks the representative mean ice thicknesses of the cross-section as a whole. The second factor is solar radiation. At the end of December, the number of hours in a day is at a minimum. Therefore, solar radiation is also at a minimum. As the winter progresses, the days get longer and reach 12h on the first day of spring, end of March. In addition to influencing air temperature, solar radiation plays a direct role in the surface ice
floe heat balance. As such, we should see its influence on ice thickness. To do so, we present figure 5. In it, ice thicknesses ($t_i$) are mean daily values, combining both floe and ebb ice thicknesses - somewhat accounting for the secondary current effects. Mean air temperature ($T_{air}$) over the last three days is also plotted since it provided the best duration for the relation between thickness and temperature - this was found to be true for another site on the river (Morse et al. 2003a and Siles 2001). Also plotted is mean solar insulation ($I$) over the same three-day period.

The interpretation of the figure is somewhat subjective. Consider the 16th to 20th of February. The mean air temperature is about -15°C and solar radiation is about 40 W/m². During that time, the average ice thickness reaches about 140 cm. However, at the beginning of March, for similar air temperatures, when the solar radiation reaches 70 W/m², the mean ice thickness only reaches 80 cm. Other periods may also be chosen for analysis and are left to the reader.

3.8 Surface Floe Size

The images are also useful to estimate floe dimensions. This has been done using the same rectified and geo-referenced images (figure 2) used to estimate ice floe velocities. One advantage of using images to estimate floe dimensions, rather than static acoustic underwater instruments, such as the IPS4, is that one can estimate both dimensions of the floes on the horizontal plane.

Floes were found to have a slightly elliptical shape although this may be only a numerical effect due to imperfect image rectification. Generally, floes were slightly larger in the perpendicular direction to the flow, than in the parallel. When flood tides come in, floe dimensions are about 50% bigger than those encountered at the ebb tide. This was to be expected since flood tide floes are generally ‘older’ and, thus, thicker, bigger and of a higher concentration. On the other hand, the Quebec Bridge (figure 1) restricts the open water area by nearly 70% and reduces the floe dimensions by grinding the floes together. This was observed, in March 2008, when a drifting floe, supporting an ice canoe, was summarily broken up into 10 pieces as it passed under the Quebec Bridge. Therefore, large floes formed downstream of the bridge are probably also broken up as they head upstream.

Regarding the relation between floe diameter and floe thickness, while it was anticipated that one be proportional to the other, no such correlation was found. This is probably due to the presence of secondary currents and to the grinding effect of the local flow constriction on the diameters of the floes.

1 Note that when no values of $t_i$ are present, such as between Jan. 15th and 20th and around March. 2nd 2006, a thick build up of anchor ice ‘blinded’ the IPS and therfore no thickness data is available. However, low air temperature and available images of these dates suggest that thickness may have been relatively high. Missing data on the fourth subplot ($Q_{ice}$) is due to one or all of these: (i) available images could not allow an estimate of the surface ice concentration due to bad weather; (ii) ice velocities are similarly missing; (iii) anchor ice ‘blinded’ the IPS4, resulting in data loss.

2 In early March 2008, the authors organized a field campaign to measure in-situ characteristics of drifting ice floes in the study area. At that time, floe break up was observed from the canoe.
3.9 Surface Ice Discharge

Unit surface ice discharge has been computed using hourly averaged surface ice keel thickness, surface ice concentration and surface ice velocities. These data are used to estimate the total discharge within the field of view of the image. By dividing this discharge by the ‘real’ width covered by the image, one has an estimate of the unit surface ice discharge within the study area. Additionally, if the total ‘effective’ width of the channel, the open water width excluding border ice, were known, the total surface ice discharge could be estimated. Environment Canada and the Canadian Coast Guard (CCG) produce daily ice charts of the area from helicopter surveys. An

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3 Note thicknesses here are average daily values and are therefore much smaller than the instantaneous values presented in Figure 4.
example of one of these ice charts is presented in figure 1. The ‘effective’ width needed to estimate the total surface ice discharge was computed using these charts and the software Google Earth™ to obtain a relatively precise measure. Subplot 4 in figure 5 shows the computed total surface ice discharge. Note that missing data does not mean a lack of ice but that either (i) the thickness was unavailable due to strong anchor ice build up on the IPS4; (ii) the concentration and/or ice velocities were unavailable because of poor image quality due to bad weather conditions; or, (iii) ice charts weren’t available. Otherwise, the plotted data are considered to be relatively accurate with respect to the computational techniques used to estimate the different parameters.

Figure 5 shows that currents and ice discharge reverse throughout each tidal cycle. Therefore, it is very probable that the net ice discharge is to be a non-null value. Figure 6 shows the estimated histogram of the surface ice discharge at the study site. A positive value means that, during this tidal day, net surface ice discharge is leaving the area downstream. A negative value means that ice is entering the channel, or net surface ice discharge directed upstream. The figure shows that discharge is extremely variable depending on ice concentration, thickness, tidal range and duration, velocity, and channel width. However, these values are not terribly reliable since they assume that the locally observed unit discharge ($q_{ice}$ in m$^2$/s) can be extrapolated to the whole channel width. We do not believe that this assumption is valid because of the strong hysteresis in the data. Therefore, unfortunately, the data did not permit the calculation of the net ice discharge leaving the River and entering the Estuary. This could have been useful as a boundary condition for operational models of the Gulf used to manage ship routes.

Figure 6. Histogram of the surface ice discharge at the study site.

3.10 Tidal Flat Ice

While previous sections use data from the 2005-2006 season, this section uses only laser rangefinder data from 2007-2008. The same study site was instrumented with a laser aimed toward the tidal flat ice near the north shore of the river. The geodesic elevation of the roof-mounted laser is known and the angle towards which it is aimed was also evaluated. Based on these two parameters and on available water levels, one could estimate the elevation at which the
laser pulse rebounded. Figure 7 shows a schematic of the localization at which the laser pointed. Three zones (‘A’, ‘B’ and ‘C’) are identified within the intertidal zone. The gray section at the far left is a zone where the border ice builds up and usually stays for the entire winter. Historically, border ice extended up to the end of zone C, the end of the plateau, and remained for the entire winter. Today, commercial ships, navigating the area at speed, limit border ice to zone ‘A’. Zone ‘B’ is the zone where the laser was pointing. Within this zone tidal flat ice builds up for a few days, but can be loosened by a passing ship and/or swept away on a spring tide. Figure 8 shows a large ice extent, up to the middle of zone ‘C’ as defined in figure 7.

![Figure 7](image_url)

**Figure 7.** Part of the channel’s cross section and approximate submerged instruments location.

![Figure 8](image_url)

**Figure 8.** Images of (a) the ice canoe and a grounded accumulation within zone ‘A’ to ‘C’ as defined in figure 7; (b) drifting floe (about 20 x 30 m) in the river with ice canoe and crew on it.
Figure 9 shows a typical time series of the elevation detected by the laser and the water levels during this period. It was observed that the laser could not detect water interface and so when there are missing data in figure 9, this could be due to the absence of ice or the presence of water on ice. Continuous photos of the area taken by a camera located beside the laser allowed us to see what the laser was seeing. Figure 9 presents calculated elevations of the tidal flat ice based on laser data. The surface of the tidal flat ice usually follows the tidal induced fluctuations of the water levels until the ice becomes grounded whereafter the levels are relatively stable until water levels rise sufficiently to float the ice once again.

![Figure 9](image_url)

**Figure 9.** Water levels and tidal flat ice surface levels fluctuations over time. Solid lines are the water levels and circles are the tidal flat ice surface levels. Dates are shown as day/ month/year.

Figure 9 demonstrates that, while tidal flat ice is grounded on low tides, its levels are not necessarily perfectly constant. A slight decrease of the measured level could be explained by the following: as the water levels goes down, the ice may bend. As a consequence, longer distances are measured by the laser. On the other hand, the increase of the level does not offer any such easy explanation. It may be caused by either (i) snow accumulation, which, after verification of cases shown in figure 9 was not the case; or, (ii) as the ice bends under its own weight, a possible explanation of the increase, it may present the laser an irregularity at the surface of the ice, such as an ice block.

It was possible to identify the channel bottom elevation at the exact location at which the laser aimed. This was done by inspecting the complete time series, as in figure 9, and by inspecting the series of available images. The bottom geodesic elevation at this location was found to be approximately 0.15 m above Mean Sea Level. An estimate the grounded tidal flat ice thickness was therefore made. Figure 10 shows the computed results. Thicknesses are between 0.4 and 2.2 m throughout the winter. Note that there are many periods when the laser did not acquire measurements – hence the large gaps between points in figure 10.
4. Comparison with available data from Lake St. Pierre

Morse et al. (2003a) characterize brash ice on Lake St. Pierre, located approximately 125 km upstream from the present study area, for three winters from 2000-2003. For these three winters, they measured surface ice keel thickness with an IPS, reporting hourly averaged ice thickness ranging from 10 to 90 cm. For the present analysis, most of the floe keel thicknesses, also hourly averaged, are less than 200 cm, but some even reached 300 cm. Of course, one has to account on the fact that weather conditions were different from winter to winter; however, the ice thicknesses at Quebec City are clearly higher than those at Lake St. Pierre, roughly double or triple the depth.

Concentrations at Lake St. Pierre were estimated using both an IPS and an ADCP (Acoustic Doppler Current Profiler). Morse and his colleagues discussed that the IPS may be much more precise to estimate concentrations. The current study offers an alternate method for estimates of this parameter using still cameras mounted on-shore. The main advantage is its low cost compared to the underwater deployment of an IPS or ADCP. Another advantage may be that the camera offers a larger vision and two dimensional approximation. From data at the two sites, in addition to an in-house analysis of data made by Morse of floe dimensions downstream of Quebec, it is concluded that concentration and floe size depend upon local flow conditions and channel geometry.

Morse et al. (2003a) presented data for unit surface ice discharge ($q_{ice}$) and total surface ice discharge ($Q_{ice}$) at the Lake St. Pierre. Between 2000-2003, they reported values of $q_{ice}$ ranging between 0.1 to 0.5 m$^2$/s, and values of $Q_{ice}$ usually well below 250 m$^3$/s. In Quebec City, except for extreme ice events, the unit discharge was normally under 0.5 m$^2$/s. This is congruent with
Lake St. Pierre data. It seems however that $q_{ic}$ peaks are higher at Quebec City, up to 1.7 m$^2$/s, than at Lake St. Pierre. This is because the contributing area at Québec is much greater than at Lac St. Pierre: ice is thicker, concentrated by the Québec bridge and flows much faster. The estimated total maximum surface ice discharge at Quebec City may reach as much as 2000 m$^3$/s in both directions. This is 10 times higher than the value found 125 km upstream in Lake St. Pierre. However, at Québec, ice going up the river comes back down on the next tide.

5. Discussion

Brash ice characteristics in the St. Lawrence River, near Quebec City, are quantified throughout this paper and some of those are compared to those available at the Lake St. Pierre. The use of an on-shore mounted still camera has proven to be very cost effective and permitted the estimate of numerous parameters, including surface ice floe velocities, identification of different types of ice, surface ice concentration and surface floe dimensions. Combined with a submerged IPS4, the camera could also provide means of estimating surface ice discharge. This estimate proved to be very close to reality since the camera ‘saw’ a much larger part, or even the total width, of the channel than a static submerged instrument might have done. However, additional cameras with different lenses would be an asset to provide alternate and more global information.

The use of the IPS4 for measuring the surface ice keel thickness has generated good results here and in many other publications (e.g., Morse et al., 2003a). A simple correlation between air temperature or $AFDD$ and surface ice thickness has yet to be discovered. Although figure 5 shows some trends, the phenomenon is complex. Solar radiation and local current directions play an important role.

As the instrumentation used throughout this study had allowed only the estimate of surface ice keel, no information about the surface ice sail is available. The use of a laser rangefinder could be a good complement to obtain the sail thickness; however, our experience with two lasers at the site is not conclusive. Also, using multiple image processing skills, multiple on-shore cameras could provide an estimate of the vertical variations of surface ice thickness over the water surface. Further work is needed to obtain these data.

Another factor to consider would have been the deposit thickness under the floes. Suspended frazil ice particles are transported into suspension below the surface floes, depending upon local flow conditions (Morse and Richard, 2008). These particles may deposit under the floes. This eventually forms a porous layer of deposited and compacted particles under the floe’s crust, which is essentially attributable to freezing air temperature. Neither underwater instruments nor on-shore mounted instruments could measure the thickness of such layers. Although Morse et al. (2003a) advance a promising avenue for the IPS to estimate porosity; this would not have provided thickness information. To date, the only way to obtain submerged layer thickness is to proceed to the field and manually measure it. This was done by the authors in March 2008 from an ice canoe. The deposit submerged layer was measured to be about 50 cm thick. This could account for a great proportion of the total thickness of the floe, knowing that the ice thicknesses in March are roughly 100 cm. More work is needed to assess the different layers’ properties within a floe.
6. Conclusions

This study quantified brash ice characteristics at Quebec City and compared them with data at Lake St. Pierre. At the Quebec site, thicknesses could be two to three times greater than those at Lake St. Pierre. Parameters such as ice velocities, ice concentration and ice floe dimensions could be estimated with a relatively high confidence level using off-the-shelf instruments, such as a camera mounted on-shore. At the Quebec site, concentration and thickness are very dependent upon the tidal state, since the flood tide brings in a great deal of ice. Camera-estimates of floe dimensions should use a lens that permits a large enough field of view to measure correctly. Also, special care must be given to the image rectification process, especially if the studied reach is tidal. It has been shown that ice velocities are dependent upon wind velocity and ice concentration. Total surface ice discharge at the Quebec City site is quantified and it was shown that there is a high quantity of surface ice drifting in the area.

7. Acknowledgments

The authors thank the City of Quebec, which provided logistical, financial and moral support to the project and, in particular, Damien Roy and Richard Simoneau for their technical and scientific input. We thank the team at ASL Environmental Sciences that also offered technical support and, at the request of the authors, were willing to modify the IPS in order to sample frazil ice in the water column. At the Université Laval, Donon Bisanswa provided many long hours of image analysis; Annie-Claude Parent, Simon Nolin and Dany Crépault provided field support and Rock Santerre GPS support. At the State University of New York, Cobleskill, Edward Stander provided technical and scientific support. We also greatly appreciate the editorial correction made by James O’Regan. The work was partially supported by a NSERC discovery grant.

8. References


Grasse River Session I: Grasse River ice evaluation
Ice Observation and Ice Thickness Modeling on the Grasse River 2003-2007

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ABSTRACT

The Grasse River flows northeasterly for more than 55 miles in northern New York to the St. Lawrence River. In the lower 7 miles of the river, remediation options are being studied by Alcoa and the U.S. Environmental Protection Agency to address elevated levels of polychlorinated biphenyls (PCBs) in fish, with the PCBs originating from the river bed sediments. In March 2003, an ice jam in the lower Grasse River caused hydraulic scour of a portion of the bed sediments. This ice event was well documented through a series of field and desktop studies. Since the ice scour event, annual observations of river ice conditions have been conducted to better understand ice formation and breakup processes on the river. Observations are documented using still and video photography, both from the ground and the air. Ice thickness is measured at key locations and these data are used to validate an ice growth and decay model.

The information gathered through the ice observation program supports the engineering assessment of ice control alternatives, which are being considered as part of the sediment remediation effort. Data from the program are also used to predict the timing and severity of ice breakup as the season progresses. This paper summarizes findings from the past five winters and presents general observations on the ice formation and breakup processes on the Grasse River.
1.0 Background and Objectives

The Grasse River flows to the northeast approximately 55 miles from a dam located at Pyrites, New York, to its confluence with the St. Lawrence River, approximately 7 miles east of the village of Massena, New York. The upper Grasse River is comprised of long, low gradient sections interrupted by shorter, steeper elevation drops (Figure 1). In the flat lying “lower” Grasse River (approximately 7 miles), remediation options are being studied by Alcoa, Inc. (Alcoa) and the U.S. Environmental Protection Agency (EPA) to address polychlorinated biphenyls (PCBs) in the river bed sediments. At the time of this writing, the findings from the referenced studies are still under review by EPA and should be considered preliminary.

In the summer of 2001, a pilot study was conducted that involved the construction of a subaqueous cap in an approximate 7-acre portion of the river using various combinations of capping materials (e.g., sand, topsoil, clay) and placement techniques (Figure 2). A program was initiated in the spring of 2002 to monitor flow conditions and conduct limited observations of the pilot study area during the annual period of spring ice breakup, when river flows are historically at their highest. Bathymetric studies conducted in the fall of 2002 confirmed that the cap materials remained in-place during the spring 2002 high flow season.

The winter of 2002/2003 was unusually cold, resulting in the formation of a thick ice cover (24 to 30 inches) that persisted until late March. During late March 2003 significant ice floe movement was observed from the upper river (upstream of Massena) through the rapids in Massena (river mile 8) and into the lower river. Photographs on March 28, 2003, captured an approximately 1-mile length of ice pieces compressed against an intact ice cover at near river mile 5.3 (Figure 2), which coincided with the capping pilot study area.

Bathymetric studies in the spring and summer of 2003 indicated that a loss of cap material and, in some areas, underlying sediment had occurred along some portions of the 7-acre pilot study area. The photographic records and information on precipitation, river discharge, and estimated ice thickness allowed a team of river ice experts to conclude that the formation of an ice jam over the pilot study area caused hydraulic scour of the capping pilot study area. A number of studies were undertaken in 2003 and 2004 to better understand the 2003 ice jam and associated bed scouring event, and to determine the past frequency and severity of similar events (Alcoa, 2004).

Since the March 2003 ice jam event, the river ice monitoring program has evolved to better understand the ice formation and breakup processes on the Grasse River and to document the specific conditions each year as they relate to the possible occurrence of ice jams in the lower Grasse River.

Objectives for River Ice Monitoring

The information gathered through the ice monitoring program enhances the understanding of river ice processes and supports the engineering assessment of ice control alternatives, including ice control structures, that are being considered as part of the larger sediment remediation effort conducted by Alcoa and EPA. Data from the monitoring program also help the Grasse River ice evaluation team predict the timing and potential severity of breakup as the winter season progresses.
A hindcasting analysis (Alcoa, 2004) was conducted as a result of the 2003 ice jam event. An 81-year record of meteorological and flow data was utilized to estimate ice freeze-up and breakup dates for the Grasse River and the corresponding river discharges. This information combined with physical evidence of prior ice jams (e.g., tree scar evidence, stratigraphic sediment core evaluation) resulted in the conclusion that there are threshold conditions at which a mechanical ice breakup and resulting ice jam would be more likely to occur. These conditions include:

- A discharge increase of at least 3,500 cubic feet per second (cfs) from freeze-up to breakup (based on daily average discharge), and
- An ice cover thickness at the time of breakup greater than 15 inches.

Reaching these conditions would not necessarily mean that ice jams capable of causing significant sediment scour will form, but these conditions are considered to be the threshold of concern at which a mechanical ice breakup and resulting ice jam are more likely to occur in the lower Grasse River (Shen et al. 2008). The importance of ice thickness at the time of breakup is two-fold: to provide an ice supply sufficient for significant jam formation and to provide a thick and strong ice cover in the lower river that can arrest the floes of ice arriving from upstream and therefore initiate an ice jam.

One of the objectives of the river ice monitoring program is to provide a reliable set of annual data that can be used to verify the relationship between spring breakup conditions and ice jam formation in the lower river. Visual observations are combined with direct measurements in order to further evaluate the breakup conditions each year, and allow the Grasse River ice evaluation team to draw conclusions regarding the possible occurrence of ice jams in the lower Grasse River.

In the winter of 2006/2007, the ice monitoring program was used for decision-making during an ice breaking pilot study conducted by Alcoa. Alcoa had retained an experienced contractor to break a 200- to 250-foot-wide channel within the lower river (typical width of 400 feet), if or when it was determined that ice cover thicknesses would reach and sustain a minimum of 15 inches prior to natural breakup. Ice thickness measurements and computer simulated ice thickness results were used in the decision to proceed with the ice breaking, and in successfully breaking the ice from March 19-24, 2007.

Components of the River Ice Monitoring Program
The river ice monitoring program utilizes multiple components to gather the information necessary for an annual assessment of the ice formation and breakup conditions. These components, discussed in the following sections, include:

- Recording of climatological conditions: air temperature and precipitation data (forecasted and actual) are obtained from online sources, and actual field conditions are noted during visual observations.
- River stage and discharge monitoring: real-time stage and discharge data are downloaded from an upstream United States Geological Survey (USGS) gaging station, and a second staff gage with a continuous recording of stage height is used in the area of the lower river where ice jams are of concern.
- Ice thickness measurement and simulation: ice thickness is typically measured twice per winter at four to five locations; the growth and decay of the ice cover is also simulated with a computer model.
- Visual observation and recording: visual observations are conducted throughout the winter with emphasis on the freeze-up and breakup periods. A photographic record is developed from 16 pre-selected observation points; video footage is provided to record ice movement and accumulation during the breakup period. Weather-permitting, an aerial photo reconnaissance is conducted just prior to the anticipated breakup period.

2.0 Climatological Conditions
Climatological data are obtained from a National Weather Service monitoring station at the Massena International Airport, which is within 1.5 miles of the lower Grasse River. Daily precipitation and maximum, minimum, and average air temperatures are recorded. Precipitation forecasts in the form of rain late in the season are monitored closely, since it is expected that a mechanical breakup and ice run would typically be associated with a sudden and significant rise in river flow associated with rainfall.

3.0 River Stage Monitoring
When the 2003 ice jam event occurred, the best available discharge data for the lower Grasse River were obtained by transforming data from the USGS gage on the West Branch of the Oswagatchie River at Harrisville, New York, to an equivalent flow of the Grasse River at Massena. This was done using correlations developed from flow records collected on the two systems between 1924 and 1977. Since October 2003, stage and discharge monitoring data have been recorded by a USGS gage in the upper Grasse River at Chase Mills, New York (river mile 20 in Figure 1), which is located approximately 11 miles upstream of Massena. Provisional real-time stage height and discharge data for the Chase Mills gage is monitored throughout the winter (http://waterdata.usgs.gov/nwis/uv/?site_no=04265432). These data are used to document the discharge during the freeze-up when the ice cover is typically fixed for the winter, and help predict the timing and characterization (i.e., thermal melt-out or mechanical breakup) of the annual ice breakup.

A staff gage has historically been available at the Alcoa West Plant Outfall 001 (Figure 2), for manual observation of stage height in the lower Grasse River. After the 2003 ice jam event, this gage was equipped with a continuous level recording device and modem to provide additional information on river stage height, especially during the breakup period each spring. The stage height information is automatically recorded every 5 minutes throughout the year. Evaluation of Outfall 001 gage data now allows for the detection of abrupt rises in stage that could be associated with ice jam formation downstream. Photographs during the 2003 ice jam event indicated a stage rise of approximately 9 feet above normal pool in the area of Outfall 001 (Alcoa, 2004), which was an important indicator of the severity of the ice jam that had formed downstream, and the degree to which flow in the river channel had been obstructed.

Based on the limited period of record available for the Chase Mills gage (October 1, 2003 through September 30, 2006), monthly mean discharge during the freeze-up months of December and January (around 1,500 cfs) were moderately less than the breakup months of March and April (1,350 cfs and 2,150 cfs, respectively). However, maximum daily mean values
ranging from 6,000 to 8,770 cfs have been recorded within the March 28 to April 5 timeframe among the 3 years that final (non-provisional) data are available. These dates correlate to the annual ice breakup periods.

A limitation of the Chase Mills gage that has been observed since 2003 is the occasional absence of real-time provisional discharge data during the transition periods of freeze-up and breakup. Discharge is not reported by USGS during these periods when in their judgment, the discharge could be affected by ice accumulation in the river. Real-time discharge reporting is discontinued once the ice cover in the vicinity of the gage is formed. In some instances discharge reporting is discontinued prior to the visual confirmation of ice cover formation in the lower portions of the river. Therefore, the discharge at the time of lower river freeze-up must be estimated. Daily average discharge for the annual freeze-up date and breakup periods since 2003 are summarized in Table 1, which includes final (non-provisional) discharge data that has become available since the actual observation periods.

4.0 Ice Thickness Measurements and Simulation

Ice thickness measurements were collected each winter season to document the intact ice cover thickness in the Grasse River. This effort was augmented in the 2004/2005 winter season with a simulation model (Shen et al 2008, Shen and Yapa 1985) that has been used to forecast ice formation and decay during each winter period since. The model uses actual and forecasted temperature data as collected from the Massena International Airport.

4.1 Ice Thickness Measurement

Each winter, attempts are made to collect two rounds of thickness measurements at pre-established locations on the river. Ideally, one of the measurements is intended for late in the season nearer to the time of breakup. However, due to safety concerns, ice thickness measurements have generally been collected between late-January through late-February. In the lower Grasse River, thickness measurements were collected at or near the following locations shown in Figure 2: river mile 0.6, Route 131 bridge, and near Alcoa Outfall 001. Thickness measurements in the upper Grasse River were collected at or near the Route 37 bridge (Figure 2) and attempted at the Madrid bridge near river mile 30 (Figure 1). Often the ice was deemed unsafe at the Madrid location and thickness measurements were not collected.

At each location, a motorized auger is used to drill multiple 8-inch diameter boreholes across the channel to determine variations in thickness with distance from shore. In order to visually differentiate between solid ice and porous “snow ice” and/or slush, technicians attempt to halt each boring prior to penetration in order to prevent water from rising into the drilled hole and obscuring the snow-ice interface. Photographic documentation is then taken and observations made for each borehole. After completing the borehole, a tape measure or graduated probe is used to hook onto the bottom of the ice cover and measure upward to the top of the borehole.

4.2 Ice Thickness Simulation

Since ice thickness at the time of breakup is important when determining the likelihood of a mechanical breakup and possible ice jam, and because direct measurement of ice thickness late in the season may pose an unacceptable safety risk, ice thickness simulation was added to the river ice monitoring program in the winter 2004/2005 season. From initial freeze-up through
breakup, the growth and decay of the ice cover thickness was simulated using actual and forecasted air temperature data.

Cover thickness simulations started in December or January of each year when an ice cover was visually confirmed for the lower river. A 15-day air temperature forecast was periodically uploaded into the model to generate a graph showing predicted ice cover thickness in relationship to the winter calendar. As the winter progresses, the “predicted thickness” portion of the curve is replaced by a “simulated thickness,” based on the actual temperatures that occurred. A simplified example of the simulated and predicted ice thickness, as of February 27, 2007, is provided as Figure 3. This figure also shows the comparison between the simulated ice thickness and the range of measured thicknesses for the lower river during 2007.

Generally, the ice simulation results have correlated well with the range of actual thickness measurements for the lower river. The predicted date of complete melt-out also has been generally consistent with visual observations of the lower river, as shown in Table 1.

5.0 Visual Observation and Recording

5.1 Monitoring of River Ice Formation and Extent
Since the 2003/2004 winter season, the extent of ice cover on the Grasse River was observed periodically at 16 or 17 pre-established locations along the lower 35 miles of the river. Dated photographs looking both upstream and downstream are taken at these locations. Monitoring is performed at least once per month beginning in early December, and more frequently when the ice cover is initially forming and when the ice cover begins to deteriorate in each season.

Each year, an estimated freeze-up date is determined based on visual observations at the seven monitoring locations within the lower river. The freeze-up date is determined when a stable and essentially complete ice cover is observed in the lower river (excluding areas around bridge piers or outfalls where faster or warmer water may delay ice formation). The freeze-up dates for 2003-2007 are summarized in Table 1. The lower Grasse River is usually fully covered with ice by mid-December, with the exception of the 2006/2007 season, when full ice cover in the lower Grasse River was not observed until mid-January. For 2002/2003, a late freeze-up date of February 13, 2003, was hindcasted using temperature records (Alcoa, 2004). This date was not confirmed with visual observation as was done in subsequent years.

In the lower river, ice cover typically extends to the center of the river through a combination of thermal border ice growth and juxtaposition of frazil ice slush and flow arriving from the steeper, faster flowing upstream reaches. This is the typical mode of ice formation in areas of the Grasse River that have a low flow velocity. In these areas of the river, the ice remains stationary through the winter with little to no visible distortion. In areas of the river with rapids or sharp drops in elevation, namely within Massena, Louisville, and Chase Mills, the ice takes longer to form and typically does not completely cover the river. The mode of ice formation is similar to that described above.
5.2 Monitoring of River Ice Breakup
Visual observations along the Grasse River intensify near the end of March as air temperature begins to increase and/or rainfall is anticipated. Beginning with the 2004/2005 season, an overflight of the Grasse River was made to provide both a written summary and oblique aerial photographs of the ice cover. These observations provide valuable insight into the overall condition of the ice cover, and the potential volume of ice that could move into the lower river should a mechanical breakup occur upstream.

During breakup, field crews are stationed along the Grasse River to visually observe and record the movement of ice floes if/when they occur, with emphasis on the movement of ice through the Massena rapids and into the lower river. In addition to still photography, video documentation is used in the final stages of ice breakup to record ice movement. A diagram is drawn two or more times per day to illustrate the river conditions observed during a given timeframe. Areas of the lower river are color-coded as either open water, ice covered, or as having broken ice pieces, based on visual observations and photos. The diagrams allow for the gradual progression of ice breakup to be illustrated.

Based on the observations from 2003-2007, the Grasse River breaks up and clears itself of ice in an upstream to downstream progression. The shorter, shallower, and steeper sections upstream of Louisville (Figure 1) break up first and progress downstream to the two long and flat pools at the end of the river: Louisville rapids to Massena Main Street Bridge, and Massena rapids to the St. Lawrence River (the lower 7 miles of the Grasse River). In the lower river, the ice cover typically remains intact in the last mile of the river until the very end of the breakup period. The lower river, with average depths of 12 to 15 feet, is deeper than the rest of the river, and is hydraulically influenced as a stable backwater of the St. Lawrence.

6.0 Characterization of Annual Ice Breakup Conditions
The visual observations, ice thickness information, river stage/discharge records, and climatological data are used annually to define a “breakup period” for the river. The data gathered is reviewed by the Grasse River ice evaluation team to characterize the breakup conditions, typically as a thermal melt-out or a mechanical breakup with ice movement. After consideration of the available data, a conclusion is made as to whether an ice jam occurred or was likely to have occurred in the lower river, and whether the ice jam was of sufficient magnitude to significantly disturb the sediments. A summary of pertinent river monitoring parameters and the characterization of the annual breakup conditions is provided in Table 1. Although most of the visual observations during the breakup period are focused on the lower river, the characterization of breakup conditions also considers the upper river, especially the long flat pool between the Louisville rapids and the Massena Main Street Bridge. This pool contains the ice that could most readily be delivered to the lower river in the event of sudden mechanical breakup.

Based on the information collected, the ice breakup periods in the 4 years since the severe ice jam in March 2003 can all be generally characterized as a thermal melt-out, although a minor ice run/jam occurred late in the 2005 season when the highest breakup discharges of the 4 years carried ice pieces into the lower river. The evidence suggests that the threshold conditions for a mechanical breakup and significant ice jam (i.e., a discharge increase of at least 3,500 cfs from
freeze-up to breakup, and ice cover thickness at the time of breakup greater than 15 inches) have not been met any of the 4 years since 2003. Although the ice cover thickness in the lower river in each year typically grew to exceed 20 inches, the cover thickness has generally been in a deteriorated state by the time breakup occurs. The minor ice run/jam observed in 2005 at around river mile 5.9 was described qualitatively as an accumulation of ice pieces which did not undergo the thickening required to create an ice jam sufficient to significantly disturb the underlying sediments, relative to the 2003 event (Alcoa, 2006a). The change in river stage at the Alcoa Outfall 001 gage (river mile 6.3) during the minor ice run/jam in 2005 was around 1 feet, further confirming that the accumulation had not resulted in a jam with thickness that would substantially restrict the flow and raise the river stage, as was the case in 2003.

7.0 Conclusions
The monitoring program of climatological data collection, river stage/discharge reporting, ice thickness measurement and simulation, and visual observations has provided a useful set of data for the Grasse River ice evaluation team to assess the annual breakup conditions, and to draw conclusions regarding the possible occurrence of an ice jam in the lower Grasse River. Additional knowledge has been gained regarding the stage/discharge trends in the river as measured upstream and in the lower river where ice jams are of concern. The river monitoring data developed since the ice jam event of 2003 have provided additional information that has been used to evaluate short-term ice control alternatives (e.g., ice breaking pilot study in 2007). The monitoring program provides information to further enhance the understanding of river ice processes, as they relate to the design of ice control structures that are under consideration for the Grasse River.

8.0 References


Figure 1. Typical Water Surface Elevations for the Grasse River.
Figure 2. Ice Thickness Measurement Locations
Figure 3. Ice Thickness Simulation Results for February 27, 2007

Source:
1. 15-Day Forecast - www.accuweather.com
2. Actual Average Daily Temperature - The Massena International Airport

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<th></th>
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<th>2004/05</th>
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<td>December 5, 2003</td>
<td>December 13, 2004</td>
<td>December 12, 2005</td>
<td>January 17, 2007</td>
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<td><strong>Mean Discharge For Freeze-up Date (cfs)^{(1)}</strong></td>
<td>660</td>
<td>1,600</td>
<td>2,800</td>
<td>780</td>
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<td><strong>Daily Mean Discharge During Breakup Period (cfs)^{(1)}</strong></td>
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<td>740 - 6,020</td>
<td>5,400 - 8,770</td>
<td>1,160 - 1,430</td>
<td>2,980 - 4,710 (provisional data)</td>
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<td>13 (March 25)</td>
<td>8 (March 27)</td>
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<td>April 6</td>
<td>March 29</td>
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<td>Thermal Melt-Out</td>
<td>Thermal Melt-Out (Preceded by Partial Ice Breaking)</td>
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<td>No</td>
<td>Yes - Minor</td>
<td>No</td>
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(1) Flow information for 2002/03 was calculated and is based on USGS data for the West Branch of the Oswegatchie River, Harrisville, NY. Flows reported for 2003/04, 2004/05, and 2005/06 are final (non-provisional) data from gage at Chase Mills, NY (#04265432) as available from USGS on 4/6/08 at http://waterdata.usgs.gov/nwis/uv/?site_no=04265432.

(2) Discharge differential of >3,500 cfs for breakup discharge over freeze-up discharge and greater than 15 inch ice cover at time of breakup.

(3) Freeze-up and breakup dates reported for 2002/03 are based on hindcasting analysis (Alcoa, 2004) and not direct observation of river conditions. The breakup date for 2003 was revised based on updated flow records as described in "Revised Parameters for DynaRICE 100-Year Return Period Sensitivity Analysis Simulations-Technical Memorandum (Alcoa, 2006b).

(4) Estimated value; provisional discharge at Chase Mills gage reported as "ice affected".

(5) A pilot ice breaking study was conducted March 19-24, 2007, clearing a 200-250 ft wide channel through the river prior to natural ice breakup that occurred March 27-29, 2007.

NA - Not Available
Ice Breaking Demonstration Project on the Lower Grasse River Winter 2007

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At the direction of USEPA, Alcoa conducted an Ice Breaking Demonstration Project in March 2007 to evaluate the effectiveness of mechanical ice breaking as an interim measure for avoiding future ice jam-related sediment scour in the lower Grasse River. The work was undertaken based on the occurrence of an ice jam in the river in spring 2003 that disturbed both the cap placed as part of the 2001 Capping Pilot Study and PCB-containing sediments below the cap.

In response to the 2003 event, USEPA and Alcoa evaluated both interim and long-term measures to prevent future ice jam events. Mechanical ice breaking in the lower Grasse River was
subsequently identified as the only potentially feasible non-structural interim measure. Alcoa is continuing to evaluate long-term structural ice control measures.

The Demonstration Project involved mechanical breaking of the intact ice cover in an approximate 7-mile reach of the lower Grasse River with the goal of providing increased conveyance capacity needed to allow the ice floes entering from upstream during natural breakup to be transported through the lower Grasse River without causing a significant ice jam and related scour event.

Ice breaking was successfully completed using two excavators operating from a barge that was moved by a shallow-draft tug. Monitoring activities conducted during the course of ice breaking activities included measurement of ice thickness, tracking of ice breaking progress, turbidity monitoring, noise monitoring, warning sign/banner observation, and photographic documentation. Significant efforts were made in the planning stages and during operations to promote community safety because the section of river involved is used for winter recreational activities including snowmobiling.

Although the project was completed, Alcoa and USEPA have agreed that ice breaking will not be recommended for the future as an interim ice control measure due to the inherent safety risks and unpredictable conditions that can impact the ability to successfully complete the work.
1. Project Overview
Alcoa Inc. (Alcoa) conducted a United States Environmental Protection Agency (USEPA)-directed Ice Breaking Demonstration Project (Demonstration Project) in March 2007 to evaluate the effectiveness of mechanical ice breaking as an interim measure for avoiding future ice jam-related sediment scour in the lower Grasse River located in Massena, New York. The work was undertaken based on the occurrence of an ice jam event in the river in spring 2003 that disturbed both the cap placed as part of the 2001 Capping Pilot Study and polychlorinated biphenyl (PCB)-containing sediments below the cap.

In response to the findings from the 2003 event, USEPA and Alcoa evaluated both interim and long-term measures to prevent future ice jam events. An interim (and potentially longer-term) pier-type ice control structure (ICS) was originally proposed to be a component of the 2005 Remedial Options Pilot Study project; however, this was not pursued due to community concerns associated with the planned location of the ICS, which was several miles upstream of the lower Grasse River in the Town of Louisville. Mechanical ice-breaking in the lower river was subsequently identified as the only potentially feasible non-structural interim measure. Alcoa continues to evaluate long-term structural ice control measures in support of the development of the revised Analysis of Alternatives Report for the river.

Based on the understanding that ice jams capable of causing sediment scour occur approximately once every 10 years, a decision process was put in place in advance of the Demonstration Project to evaluate whether ice breaking was warranted in spring 2007 considering both actual and forecasted ice and weather conditions. Following a review of available data on ice coverage, ice thickness, and air temperature, a recommendation to proceed was made by the Alcoa ice expert group and subsequently agreed with by USEPA.

The Demonstration Project was conducted March 19 through 24, 2007. The project involved breaking the intact ice cover in an approximate 7-mile reach of the lower Grasse River, with the goal of providing increased conveyance capacity needed to allow the ice floes entering from upstream during natural breakup to be transported through the lower Grasse River without causing a significant ice jam and related scour event.

![Figure 1. Lower Grasse River Approximate Ice Breaking Limits](image)
2. Project Objectives
The Demonstration Project was implemented to evaluate the feasibility of ice breaking as an interim measure for mitigating ice jam scour in the lower Grasse River and to develop site-specific information to support an understanding of the impact of site conditions (e.g., ice thickness, weather, river flow) on the schedule, production rate, and associated cost of the ice breaking operation. Another key objective included evaluation of the community notification measures to minimize potential safety issues associated with winter recreational use of the river once ice breaking activities were initiated.

3. Community Outreach and Notification
Safety of the community and personnel involved with ice breaking and monitoring activities, along with protection of the surrounding environment, were critical considerations during the planning and implementation stages of the Demonstration Project. Alcoa committed significant efforts to ensure that safety was the highest priority prior to, during, and after completion of ice breaking activities.

The Demonstration Project was conducted prior to natural ice out from the river, when the river would normally be used for recreational purposes (e.g., snowmobiling, ice fishing). In addition, it was recognized that, during the conduct of the project, portions of the lower river would contain both broken and unbroken ice as a result of the Demonstration Project. As such, it was essential that the community be informed that the lower river was not safe for recreational use. Alcoa, working in conjunction with USEPA, developed and implemented an extensive community notification program and developed emergency planning and response procedures to inform and protect the community.

The community notification program was designed to help inform the public about the project, provide opportunities for the public to ask questions about proposed activities on the river, and to provide an opportunity for the community to make recommendations regarding additional notification activities that should be considered prior to the start of the work. This program was utilized to notify the community about the elements of the Demonstration Project and, most importantly, the associated safety concerns.

Community notification efforts included community meetings, announcements in local papers and on local radio and television stations, distribution and posting of community mailers and flyers, announcements at local community group meetings, schools, local businesses, community centers, and door-to-door visits to residents along the lower Grasse River. Alcoa also installed warning signs and lights at 20 river access/egress points and posted banners on bridges over the lower Grasse River and Raquette River (tributary to the St. Lawrence River approximately 4 miles downstream of the mouth of the Grasse River).
Concerns were expressed during the community outreach efforts regarding the ability to successfully reach all potentially affected parties (e.g., recreational users of the lower river), irrespective of the level of effort employed in the notification process. The potential consequences of a safety incident involving a recreational river user was a major concern both during the conduct of the project and from the conclusion of the project through the natural ice out of the upper Grasse River.

In addition to community notification and safety efforts, environmental and worker health and safety measures were implemented at the onset of the Demonstration Project and were reinforced through its completion. These measures included development of written health and safety plans (HASPs) to establish safe working guidelines and procedures, Alcoa-sponsored site orientation training prior to the start of ice breaking activities, daily safety meetings for all site personnel, and environmental health and safety audits and reviews to promote and maintain safe working conditions.

4. Ice Breaking Activities
A tug and barge were mobilized to the Alcoa East Plant (located on the St. Lawrence River just downstream of the Grasse River mouth) on December 21, 2006 to stage equipment prior to river freeze-up. Additional equipment (e.g., excavators, lighting) was mobilized to the Alcoa East Plant on March 14, 2007 to prepare for ice breaking activities, which began on March 19, 2007.

Ice breaking was performed in an approximate 7-mile reach of the lower Grasse River with two excavators operating from a barge that was moved by a shallow-draft tug. Two crews worked in alternating 12-hour shifts, 24 hours per day to complete the project. Way Point (WP) designations were established for navigation purposes and to track daily progress. The tug was equipped with a Global Positioning System (GPS) that was linked to an electronic bathymetric map of the river. The GPS was also aligned with pre-defined polygons that identified the portion of the river channel targeted for ice breaking. The GPS system was used to check the location of the ice breaking equipment and periodically track progress in consideration of the WPs. The GPS system was verified by oversight personnel routinely checking progress in the river visually and periodically requesting coordinates to check the location on independent mapping. This information was used to determine daily production rates and overall ice breaking progress.
Figure 3. Excavators and Barge during Ice Breaking Activities

Progress during each shift depended on many conditions including ice thickness, river flows, weather (e.g., air temperature, wind speed and direction), and river course (e.g., straight versus curved river sections). These conditions are key factors in being able to clear ice from the channel. Unfavorable situations associated with any or a combination of these conditions impacted the ability to clear ice from the channel once it was broken, and backtracking was required during several shifts to clear and widen channels that had become clogged with ice pieces or where ice had reformed. Observations and results from the Demonstration Project are summarized below.

- Ice thickness: Prior to initiation of the ice breaking, the average ice thickness at the winter monitoring locations was approximately 22 inches. Ice thickness was also measured throughout the project by retrieving ice pieces with the excavators and bringing them to the barge surface for direct measurements of thickness. During the project, ice thicknesses ranged from 8 to 25 inches. The thickest ice was observed within the curves in the river. In general, the thicker the ice, the greater effort required to break and clear the ice from the river.

- River flows: River flows were monitored and recorded during ice breaking using the United States Geological Survey (USGS) gage located upstream of the lower Grasse River. Higher river flows and current aid in movement of broken ice from the cleared channel downstream and out of the river. At the start of the Demonstration Project (March 19), instantaneous river flows were approximately 2,700 cubic feet per second (cfs); however, flows decreased to approximately 1,500 cfs in the middle of the project (lowest recorded during the project), and then began to steadily increase through the end of ice breaking. On several occasions, forward progress for ice breaking ceased in order to clear ice from the previously broken sections of river.

- Weather: Weather conditions fluctuated significantly during the Demonstration Project. Air temperatures ranged from approximately -10 degrees Fahrenheit (°F) to 60 °F. No work was conducted during the day shift of Day 3 due to extremely cold temperatures. Likewise, wind varied in direction and magnitude from calm to strong gusts. In general, wind directions from the southwest and west were favorable for clearing the channel of ice, while winds from opposing directions inhibited ice clearing.
River course: The course of the Grasse River also had an impact on ice movement. Portions of the river with curves, especially the north/south curves (Figure 1), tended to accumulate broken ice that required clearing with the tug and barge to facilitate ice movement downstream. In the straight sections of the river, broken ice typically progressed downstream naturally.

In summary, the site conditions identified above resulted in ice breaking activities taking longer than anticipated. For example, the maximum measured ice thickness coincided with the lowest air temperature, and the thicker ice slowed forward progress in the north/south bends in the river. Also, ice breaking was hampered by decreased river flows, calm winds and/or winds from opposing directions, and river course, and multiple efforts were required to facilitate ice movement downstream with the tug and barge. Ice breaking efficiency increased in the middle of the project due to increased flows, warmer air temperatures, and favorable wind direction.

Figure 4 illustrates daily production rates achieved based on forward progress in breaking new ice each day along with daily average air temperature, average wind speed and direction, and discharge rates.

Note that equipment failures also resulted in some delays, but downtime associated with this work was limited. The original schedule for the work was 3 days, and the actual time to implement the work was 5 days.

Ice breaking operations were completed on March 24, 2007 to a location approximately 2,000 feet downstream of the Alcoa Bridge. The originally proposed upstream extent of ice breaking activities was approximately 500 feet downstream of the Alcoa Bridge, but crew observations of
frazil ice led to the termination of operations short of the planned location. An ice breaking channel width of approximately 200 to 250 feet was achieved throughout the river, with the exception of the final approximate 3,000 feet, where the channel was narrower (approximately 130 to 150 feet) and oriented toward the southern shore of the river. Attempts to break a wider channel resulted in visibly turbid water conditions, presumably due to relatively shallow water depths.

Figure 5. View of Channel with Ice Clog and Cleared

There were no community or worker health and safety incidents during ice breaking operations. On four separate occasions, individuals were observed on the ice and, during one shift, headlights were observed near a boat launch in proximity to the ice breaking operation. In accordance with the established health and safety procedures, operations were stopped until it was confirmed that the river was clear and it was safe to continue operations.

5. Summary of Monitoring Activities

As discussed in Section 4, monitoring activities conducted by the contractor during the course of ice breaking activities included measurement of ice thickness and tracking of ice breaking progress. Additional community-related monitoring included turbidity monitoring, noise monitoring, warning sign/banner observation, and photographic documentation as described here. In addition, the captain and crew of the ice breaking operation, along with shore-side support personnel, monitored for the presence of individuals on the ice in the vicinity of the work. Overall, these activities were conducted to inform site management on progress of the work, identify the impact that site conditions have on ice breaking effectiveness, and to maintain measures implemented to protect the community.

The following bullets summarize results of the turbidity, noise, warning sign/banner, and photographic monitoring activities.

- Turbidity was monitored using a real-time turbidity meter that was enclosed in a metal protective sleeve welded to the barge stern hull. The meter was suspended from the barge in the sleeve with approximately 6 inches of the meter extending below the bottom of the sleeve. The meter was programmed to run continuously and record turbidity levels and temperature every 15 minutes. The meter was retrieved after the first 24 hours of ice breaking, and upon review of the readings, it was clear that the meter was not working correctly (likely due to ice trapped within the guard and frozen to the sensors). At some
point during the second day of ice breaking, the protective sleeve and meter detached from the barge and were lost.

- Noise was monitored approximately every 12 hours at the shoreline or closest receptor locations. Monitoring results indicated that noise levels were not sustained over the standard. Reports of noise-related concerns during night operations from local river residents were received during the follow-up community survey (Section 6).

- The warnings signs and banners were monitored 2 to 4 times per day to observe the condition of signs/banners and identify any needed repairs. Any identified problems were corrected as quickly as possible.

- Photographic documentation included digital still and video images to record all components of the ice breaking activities. Aerial flyovers were also used on five occasions to record river and ice conditions.

6. Post-Ice Breaking Community Surveys and Input
Focus groups and surveys were conducted after the Demonstration Project to understand community reaction to the project and evaluate the community notification measures. There were five major concerns noted by the community including: danger to snowmobiles; noise at night; shortened winter recreational season; environmental disturbance; and worker safety. In general, survey participants noted a major concern about the potential for a snowmobile accident occurring as a result of the project, and extensive public notification was considered the best way to guard against this possibility.

The participants indicated that the notification procedures implemented by Alcoa and USEPA were comprehensive and effective, and the average respondent received notification from at least four different sources. Signage was also noted to be situated in the appropriate locations and was effective.

7. Summary and Conclusions
The ice breaking operation was successfully completed in the lower Grasse River, with an approximate 7-mile channel opened from the confluence with the St. Lawrence Seaway to just downstream of the Alcoa Bridge. Observations and data collected during the Demonstration Project provided useful information regarding implementation-related issues, impacts of site conditions on effectiveness and efficiency, and the effectiveness of community notification efforts.

Spring 2007 ice monitoring activities carried out following the completion of the Demonstration Project indicate that most of the upstream ice melted in place (thermal melt out) and a major ice run from upstream sections of the Grasse River did not occur as part of the spring breakup. Based on these conditions encountered during the natural breakup period, the Demonstration Project did not provide a true test of whether the broken channel created would prevent a significant ice jam from forming.
Significant effort went into planning and preparing for the Demonstration Project such that the work could be performed safely from both a community and worker safety standpoint. Although there were no health and safety incidents, individuals were identified on the river on four occasions. These individuals safely left the ice, but the observations indicate the limitations involved in attempting to restrict access to the river during and following the ice breaking operations.

Factors observed to influence the efficiency and effectiveness of the ice breaking operation included ice thickness, temperature, wind direction, and river flow. A primary lesson learned from the project was that the convergence of a number of inherently unpredictable conditions related to these factors, which can impact both the need to break ice in a given year and the ability to complete the operation in the available timeframe, is required for this approach to meet the intended project objectives of preventing ice jam formation.

Based on the experiences and observations from the Demonstration Project, ice breaking will not be conducted as an interim ice control measure in the future for the lower Grasse River.

8. Acknowledgements
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- William Moon and Treina Puente (Camp Dresser & McKee) – responsible for environmental health and safety and provided support for community notification/survey efforts.
- McKeil Marine Ltd. – served as the primary contractor for ice breaking activities.
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- Quantitative Environmental Analysis, LLC – led work plan development.
Parameters for DynaRICE 100-Year Return Period Frequency Sensitivity Analysis Simulations

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Subsequent to the occurrence of the 2003 scour event in the Grasse River, Alcoa conducted an extensive river ice investigation to obtain the information needed to understand the impact of ice-related scour on long-term remedial options for managing PCB-containing sediments in the river. A pier-type ice control structure (ICS) was one potential effective technology identified during this investigation. Several studies were conducted to help support the evaluation and design of such a structure, including numerical and physical model studies.

The numerical and physical model studies were conducted using the 2003 ice scour event as the design event. This event was considered an appropriate design event for the evaluation of the Grasse River ICS based on the severity of the 2002-2003 winter relative to prior winters, extent of scour observed during the 2003 event relative to historic events, the height of tree scars
resulting from the 2003 ice jam relative to the historic record, and the volume of information that was generated for the 2003 event. In addition to this design event, several combinations of river flow and ice supply with estimated recurrence intervals of approximately 1 in 100 years were tested to account for the inherent year-to-year variability associated with these parameters. The selection of these 100-year events was based on a combined probability analysis performed with over 80 years of site data. Conservatism was incorporated into the design event and 100-year event conditions used in the modeling studies.

This paper details the approach used to define the design event and 100-year event conditions used in the numerical and physical modeling analyses of ice control alternatives for the lower Grasse River. The findings presented in this paper represent work in progress only, as the final remedy at the Grasse River site has yet to be chosen.

1. Introduction

Subsequent to the occurrence of the 2003 ice scour event in the lower Grasse River, Alcoa conducted a comprehensive river ice investigation to obtain the information needed to understand the relevance of ice jam-related sediment scour on long-term remedial options for managing polychlorinated biphenyl- (PCB-) containing sediments in the river (Alcoa, 2004a). Several potential ice control alternatives were evaluated as part of this investigation, with three options surfacing as viable options that warranted further consideration: (1) a pier-type ice control structure (ICS); (2) integration of ice control into the design of a hydropower dam that is currently being considered on the Grasse River by the Massena Electric Department (MED); and (3) an armored cap designed to withstand the forces associated with ice jams. Figure 1 shows the location of the 2003 ice jam event, as well as the proposed locations of the pier-type ICS and the MED hydropower dam. Several studies were conducted to help support the evaluation and design of these structures, including numerical and physical modeling studies. The numerical DynaRICE model developed by Clarkson University (Shen et al., 2000), which simulates the dynamic transport and jamming of surface ice in river systems, was applied to evaluate the stability of ice accumulations and resultant backwater levels upstream of the ICS, as well as to assist in the design of the physical model study. The physical modeling study was conducted to evaluate the overall reliability of the ICS design for ice retention, and was performed by the U.S. Army Corps of Engineers (USACE) at its Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, New Hampshire.
2. Ice Breakup Design Conditions

In order to design and construct an operationally reliable ICS, the characteristics of a “design” ice breakup event must be understood. Important ice jam characteristics include the ice floe thickness, volume of the upstream ice supply, and the nature and sequence for breakup (Tuthill and Lever, 2006). In the Grasse River, breakup typically occurs as a series of jams and releases. As a result of upstream jam formation and melting during breakup, a large portion of the pre-breakup ice volume never reaches the Village of Massena. For example, the ice volume of the 2003 ice jam that was estimated to reach the lower river represents only a small fraction (about 10%) of the total pre-breakup ice volume estimated for the approximate 64-kilometer (km) (40-mile) long potential source reach of the river upstream (Alcoa, 2007).

The 2003 ice jam event has been identified as an appropriate design event for the evaluation of potential ICSs for the Grasse River. The basis for this selection includes:

- The winter of 2003 produced an unusually thick (about 0.6 to 0.8 meters (m) (24 to 30 inches)) ice cover that was amongst the thickest estimated for the historic record (Alcoa, 2004a).

- The extent of the sediment scour experienced during the 2003 event was greater than that attributed to three historic ice jam events determined to have occurred in the lower river since the 1970s (Alcoa, 2004d).
An approximate 2.7 m (9 foot) increase in stage was observed approximately one mile upstream of the ice jam that formed during the 2003 event. This rise in stage upstream of the 2003 jam damaged an Alcoa outfall structure on the river; the damage caused by the 2003 was the worst noted by Alcoa employees during their 30 years of employment at the plant (Alcoa, 2004a).

The 2003 event resulted in the highest tree scars in the lower Grasse River over the approximate 40-year period of record captured by tree scars. Tree scars have been used on other river systems to confirm the occurrence of past ice runs and jams, and estimate the maximum river stage that occurred during an event (Alcoa, 2004c).

The volume of site- and ice jam-specific information that exists for the 2003 event far exceeds that available for historic events on the river. This information base provides the best and most comprehensive characterization of a severe ice jam event for use in the evaluation of an ICS on the lower Grasse River.

For the 2003 event, the river flow at the time of breakup (termed the breakup flow) was approximately 178 meters per second (m/s) (6,303 cubic feet per second (cfs)) and the peak flow during the breakup jam was about 242 m/s (8,559 cfs). The total solid ice volume delivered to the lower Grasse River was estimated at approximately 303,000 cubic meters (m³) (10.7 million cubic feet (ft³)). This ice supply volume was determined through the calibration of the DynaRICE model to various data collected immediately after the 2003 ice event, including: (1) observed water surface elevations measured upstream of the ice jam location; (2) ice jam length, as determined through photographic records of the 2003 event; and (3) tree scar survey data collected upstream of the jam location (Alcoa, 2004e). This solid ice volume (303,000 m³ or 10.7 million ft³) was used for the pier-type ICS evaluation given the proposed proximity of the ICS relative to the location of the 2003 ice jam in the lower Grasse River. The 2003 design event conditions are presented in Table 1.

3. 100-Year Return Period Event Conditions

The two primary input parameters common to both the numerical and physical modeling activities include river flow and ice supply. To account for the inherent year-to-year variability associated with these parameters, a sensitivity analysis was conducted in conjunction with the analysis of the 2003 design case in order to evaluate the performance of the pier-type ICS and MED hydropower dam under a range of river flow and ice supply conditions. A series of six sensitivity analysis cases were identified to cover a range of flow conditions and ice thickness with an estimated recurrence interval of approximately 1 in 100 years (i.e., combinations that have a 1% chance of occurring in any given year). To accomplish this, a hindcasting analysis was performed to estimate the likelihood of ice jam occurrence in the Grasse River during past winters.

The hindcasting analysis consisted of the review and analysis of river discharge change from freeze up to breakup, air temperature and ice cover thickness for each winter over an 81-year period between 1922 and 2003 (Shen et al., 2008). River flows during each winter were defined using stream gage records collected over the period of record (Alcoa, 2001). Ice cover
thickness was determined using air temperature records from the Massena International Airport and the unified degree-day method described by Shen and Yapa (1985). The river discharge records and estimated ice cover thicknesses for each winter were compared to determine which of four breakup conditions likely occurred that winter: no ice jam; ice jam not likely; possible ice jam; and ice jam (Shen et al., in press). Two mid-winter jam scenarios were also considered. Historical Grasse River breakup dates reported in the local Massena Observer for 1928 through 1973 and 1977, observed breakup dates on the nearby St. Regis River, Hogansburg, New York for 1990 through 2003 (Haehnel and Gagnon, 2003), and data on ice jam occurrences from tree scars along the Grasse River (Alcoa, 2004c) were used to validate the likely breakup conditions determined for each winter. The results of this analysis are presented in Figure 2.

![Figure 2](image)

**Figure 2.** Ice jamming potential as a function of discharge change and ice cover thickness at the time of breakup. Numbers represent years of possible or known ice jams.

Using Weibull plotting position (Ponce, 1989), the probability and return period for each winter were then estimated as:

\[
\frac{1}{T} = P = \frac{m}{n+1}
\]

[1]

where:

- \( T \) = return period
- \( P \) = probability of occurrence
\[ m \text{ = rank of each event in descending order with the largest value equal to 1} \]
\[ n \text{ = number of values in the series} \]

Since river flow and ice supply are independent variables (i.e., the value of one is not dependent on the other), a combined probability analysis of river flow and ice cover thickness was performed to determine the recurrence intervals for various combinations of river flow and ice cover thickness (Alcoa, 2006; Shen et al., 2008). The combined probability of river flow \((Q)\) and ice cover thickness \((h)\) was calculated as

\[ P(Q \cap h) = P(Q) \times P(h) \]  \[\text{[2]}\]

and the associated return period calculated as

\[ T = \frac{1}{P(Q \cap h)} \]  \[\text{[3]}\]

where:

\[ Q \text{ = river discharge} \]
\[ h \text{ = ice cover thickness} \]

The results of this analysis are shown in Figure 3. The sensitivity analysis cases developed from this analysis and subsequently used in the evaluation of the pier-type ICS and MED hydropower dam are summarized in Table 1.

\textbf{Table 1.} 2003 design event and sensitivity analysis conditions used in pier-type ICS and MED hydropower dam evaluation.

<table>
<thead>
<tr>
<th>Case</th>
<th>Breakup Discharge m³/s (cfs)</th>
<th>Modeled Peak Discharge m³/s (cfs)</th>
<th>Cover Thickness m (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>178 (6,303)</td>
<td>242 (8,559)</td>
<td>0.59 (23.25)</td>
</tr>
<tr>
<td>Ice A</td>
<td>300 (10,608)</td>
<td>408 (14,405)</td>
<td>0.10 (4)</td>
</tr>
<tr>
<td>Ice B</td>
<td>281 (9,936)</td>
<td>382 (13,492)</td>
<td>0.25 (10)</td>
</tr>
<tr>
<td>Ice C</td>
<td>259 (9,139)</td>
<td>351 (12,410)</td>
<td>0.41 (16)</td>
</tr>
<tr>
<td>Ice D</td>
<td>209 (7,372)</td>
<td>283 (10,011)</td>
<td>0.56 (22)</td>
</tr>
<tr>
<td>Ice E</td>
<td>190 (6,724)</td>
<td>259 (9,131)</td>
<td>0.61 (24)</td>
</tr>
<tr>
<td>Ice F</td>
<td>93 (3,289)</td>
<td>126 (4,466)</td>
<td>0.73 (28.7)</td>
</tr>
</tbody>
</table>
In addition to ice floe thickness and discharge (e.g., “ice supply”) required by both the physical and numerical models, the numerical DynaRICE model also requires input of the river flow hydrograph through the period of ice jam formation. The approach for specifying the flood hydrograph to be used for sensitivity analysis simulations was to proportionally adjust the 2003 design event hydrograph according to the following equation:

\[
Q(t)^{100} = Q(t)^{03} \times \frac{Q_{bk}^{100}}{Q_{bk}^{03}}
\]  

[4]

where:

- \(Q(t)^{100}\) = time dependent (instantaneous) discharge at upstream model boundary for the 100-year event
- \(Q_{bk}^{100}\) = breakup (daily average) discharge for 100-year return period sensitivity case
- \(Q(t)^{03}\) = time dependent (instantaneous) discharge at upstream model boundary for 2003 event
- \(Q_{bk}^{03}\) = breakup (daily average) discharge for 2003 event (178 m\(^3\)/s or 6,303 cfs)

**Figure 3.** Return period analysis for combinations of breakup discharge and ice thickness.
Similarly, ice supply for the various 100-year return period scenarios was proportioned to that from the 2003 event according to the following equation:

\[ V_{ice}^{100} = V_{ice}^{03} \times \frac{h_i^{100}}{h_i^{03}} \]  \[5\]

where:

- \( V_{ice}^{100} \) = ice supply for the 100-year event
- \( V_{ice}^{03} \) = solid ice supply volume for the 2003 event (303,000 m\(^3\) or 10.7 million ft\(^3\) for the pier-type ICS; 238,000 m\(^3\) or 8.4 million ft\(^3\) for the MED hydropower dam)
- \( h_i^{100} \) = ice cover thickness at breakup for each 100-year return period sensitivity case
- \( h_i^{03} \) = ice cover thickness at breakup for the 2003 event

The numerical DynaRICE model simulates an approximate 30-hour period of ice jam formation, based on the 2003 design event hydrograph (Alcoa, 2004e). For the sensitivity analysis simulations, this same 30-hour portion of the hydrograph was scaled up or down based on Equation 4.

4. Conservatism in Modeling Approaches

The performance of the pier-type ICS and the MED hydropower dam was then assessed through numerical and physical model studies. The combinations of river discharge and ice cover thickness (i.e., 2003 design event and six 100-year return period scenarios; see Table 1) used during these modeling studies were considered conservative for several reasons, as discussed below.

For the numerical DynaRICE modeling, the use of the 2003 event hydrograph (scaled according to the appropriate 100-year event breakup flow condition) for the sensitivity analysis simulations is considered conservative since:

- The shape of the 2003 hydrograph is atypical of most other jam events that occurred or may have occurred historically on the river, and could be characterized as an “extreme” event. The 2003 event hydrograph is considered atypical since it has an extended period (i.e., several days) of higher flows after the hindcast breakup date; in all but two of the remaining 13 known or possible historical ice jam events determined in the hindcasting analysis, a declining hydrograph is exhibited after (or shortly after) the hindcast breakup date (Alcoa, 2006).

- Scaling the atypical 2003 hydrograph results in modeled peak flows (used in the numerical DynaRICE model), which are based on instantaneous flows measurements collected during the 2003 event, that are considerably higher than the daily average breakup discharges determined from the historic flow record during the hindcasting analysis (see Table 1). Comparison of instantaneous (i.e., 15-minute) and daily average flow records for the Grasse River at Chase Mills flow gage indicates that
peak instantaneous flows are, on average, about 5% higher than the reported daily average flow (Alcoa, 2006).

- The occurrence of a more extreme event (i.e., one with higher discharges or larger ice supplies than a 1-in-100 year event) is possible, but the current understanding of river ice dynamics, coupled with field observations, supports the existence of an upper limit on the magnitude of the possible combinations of ice supply and river flow. This is because very high discharges are the result of combined rainfall and snow melting, both of which are associated with warm temperatures above the freezing point. These warm temperatures also result in high melting rates of the ice, making it unlikely for both very high discharges and very high ice supplies to occur simultaneously. The coincidence of rapid decreases in ice volume due to melting and increases in discharge has been noted during prior studies (Tuthill and White, 2004; Lever and Daly, 2003; Lever et al., 2000).

The use of the 2003 ice supply as the basis for scaling ice supply volumes for the sensitivity cases (as per Equation 5) provides additional conservatism to the modeling studies conducted for the pier-type ICS, since the 2003 ice supply includes ice present on the river downstream of the proposed location. Further, for these analyses, DynaRICE did not consider overbank flow and, thus, predicted backwater levels are higher than if overbank flow was simulated (Kolerski and Shen, 2007).

In the physical modeling, a variable discharge hydrograph was not simulated over the short duration of the ice jam formation period for practical reasons of model operation (Tuthill, 2007). Therefore, a single discharge was chosen to represent the ice jam formation period. For the physical modeling of the 2003 design event, a constant discharge of 215 m$^3$/s (7,600 cfs) was used, which represents the average for the 21-hour ice jam formation period for the 2003 event. This value is similar to the average of the breakup discharge (178 m$^3$/s or 6,303 cfs) and peak discharge (242 m$^3$/s or 8,559 cfs) for this event (210 m$^3$/s or 7,431 cfs; see Table 1). This design discharge was then bracketed with the higher and lower flows used in the sensitivity analysis (Ice B and Ice F in Table 1). As was done for the 2003 design event, the average of the breakup and peak discharges were computed for these two sensitivity analysis cases yielding discharges of 332 m$^3$/s (11,714 cfs) and 110 m$^3$/s (3,878 cfs), respectively, and applied during the physical model testing. This approach was considered conservative since, similar to the numerical DynaRICE model simulations, it results in the use of a discharge in the physical model that is higher than the breakup discharge determined from the hindcasting analysis (see Table 1).

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Grasse River Session II: Evaluation of structural ice control alternatives
Evaluation and Design of an Ice Control Structure on the Lower Grasse River

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This paper describes studies conducted by Alcoa Inc. (Alcoa) to evaluate the expected performance and support the design of a pier-type structure to control ice jams on the lower Grasse River located in Massena, New York. We build upon numerous studies and investigative work conducted following the 2003 ice jam event on the lower Grasse River. The information presented herein provides a basis for the evaluation and design of a pier-type ice control structure (ICS) at “T6.75” a site located 0.75 miles downstream of the Massena Rapids. Specifically, this paper: 1.) Identifies a range of options considered for controlling ice jams in the river, and 2.) Summarizes the results of the studies evaluating the technical feasibility and design development of a pier-type ICS to control ice jams and ice related sediment scour in the lower Grasse River. The findings presented in this paper represent work in progress only, as the final remedy at the Grasse River site has yet to be chosen.
1.0 Background and Introduction

Alcoa conducted a Capping Pilot Study (CPS) in the lower Grasse River in summer 2001 that involved placement of capping materials in a seven-acre section of the river (Figure 1) using various combinations of cap materials and placement techniques (Alcoa, 2002). Post-placement monitoring conducted through fall 2002 indicated that the cap had remained in place and was functioning as designed (Alcoa, 2003). As part of the routine monitoring work conducted in the CPS area in spring 2003, a loss of cap material and in some cases underlying native sediments was observed. As a result of these observations, an extensive follow-up investigation was initiated to understand the cause of the changes in the CPS area, the mechanisms involved, and the overall impacts of the event on the distribution of polychlorinated biphenyls (PCBs) in the river. The loss of cap material and native sediments was caused by high water velocities and turbulence underneath the toe of an ice jam that formed during the spring 2003 ice breakup (Alcoa, 2004).

Figure 1. River segments historically susceptible to ice jam-related scour.

The follow-up investigation of the nature and causes of the ice jam event revealed the following information (Alcoa, 2004):

- Ice jams occur in the lower Grasse River when an ice run encounters an intact ice cover on the lower river that has sufficient strength to arrest the ice run. Contributing conditions include a thick ice cover on the lower river at the time of ice breakup in the upper river, coupled with a significant rise in the river flow driven by a precipitation and/or snowmelt event that triggers mechanical breakup of the upstream ice cover;
Ice jams that are significant enough in size to cause sediment scour occur on the order of once a decade;

Available information supports that the reach of the lower river that is vulnerable to ice jam-related sediment scour is the upper 1.8 miles of the lower Grasse River (Figure 1), and;

Post-event monitoring indicated that the 2003 jam did not significantly affect PCB concentrations in sediments, water, and fish.

The occurrence of the spring 2003 ice jam event resulted in the need to both update the conceptual site model of PCB fate and transport processes for the lower Grasse River, and to evaluate potential options for addressing ice-related scour. This paper discusses the approach taken to the identification and evaluation of potential short term and long term ice management options for the lower Grasse River.

The evaluation and preliminary design of ice management options was performed in coordination with the United States Army Corps of Engineers (USACE) Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, New Hampshire under a Cooperative Research and Development Agreement. In support of this effort, ice control structure (ICS) performance at the sites under consideration was evaluated at Clarkson University using the computer simulation model DynaRICE and through physical model studies at CRREL.

2.0 Preliminary Screening of Ice Control Options

A range of options for mitigating ice jams and ice jam scour on the lower Grasse River was evaluated. Table 1 lists the options considered, and comments on their relative effectiveness and ease of implementation. Structural options considered included dams, including integration of ice control into a proposed hydropower project sponsored by the local public utility (MED project), restoring the breached Massena Weir, ice booms, and pier-type ice control structures. Non-structural methods screened included mechanical ice breaking, thermal melting using warm effluent and surface treatment to melt ice, i.e. dusting with a dark material to increase absorption of solar radiation.

Dams

River dams are extremely effective ice retention structures, particularly those with underflow spillway gates. Provided the reservoir has sufficient depth and capacity, the entire ice run from upstream can be stored while flow passes beneath the stable ice accumulation and out the gates. Dam technology is time-tested and reliable, but obtaining the necessary regulatory approvals can be a complex process. Although the cost is relatively high, a dam can also provide benefits by producing hydroelectricity, which can offer a payback over time, and can provide recreational as well as waterfront enhancement opportunities.
Table 1. Preliminary assessment of ice control options.

<table>
<thead>
<tr>
<th>OPTIONS</th>
<th>EFFECTIVENESS</th>
<th>IMPLEMENTABILITY</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STRUCTURAL</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dam upstream with underflow gates</td>
<td>HIGH: Retains all upstream ice.</td>
<td>Technical: Uses existing technology</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Administrative: Strong local support. Complex permitting process. Two to four year licensing process for hydroelectric facility.</td>
</tr>
<tr>
<td>Restore old weir in Massena downstream of Main St. Bridge</td>
<td>LOW: Helps stabilize ice cover. May induce upstream jamming and delay arrival of breakup ice run to lower river.</td>
<td>Technical: Uses existing technology</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Administrative: Strong local support. Requires permits.</td>
</tr>
<tr>
<td>Series of ice booms upstream to delay upper river breakup</td>
<td>LIMITED: Ice passes booms at higher discharges.</td>
<td>Technical: Existing technology requires annual installation, removal, and maintenance.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Administrative: Requires permit; may encounter opposition to new structure in a river.</td>
</tr>
<tr>
<td>Pier structure upstream of jam-prone area</td>
<td>HIGH: Greatly reduces ice supply to lower river jams, or delays ice arrival into falling limb of breakup hydrograph.</td>
<td>Technical: Technology available; requires model testing. Administrative: May encounter local opposition to new structure in a river. Requires permits.</td>
</tr>
<tr>
<td><strong>NON-STRUCTURAL/OPERATIONAL</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mechanical: Ice breaking/cutting</td>
<td>MODERATE: May be effective if implemented just before jam events; this timing is difficult to predict.</td>
<td>Technical: Used elsewhere; equipment availability potential issue; requires annual effort.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Administrative: May require permit.</td>
</tr>
<tr>
<td>Thermal melting</td>
<td>LIMITED: Effective only if very large thermal source such as power plant cooling water available (none known).</td>
<td>Technical: Existing technology; cost to provide adequate thermal source very high.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Administrative: Requires permit.</td>
</tr>
</tbody>
</table>

**Restore Massena Weir**

Strong local support exists for restoring the existing but deteriorated weir located in Massena, approximately 183 m (600 ft) downstream of the Main Street Bridge (see Figure 4). This would increase the base flow water depth by about 0.6 m (2 ft) and improve conditions for recreational boating between Massena and Louisville. Re-establishing the historic water level would decrease water velocity and therefore stabilize the upper river ice cover, possibly delaying upper river breakup, but it would not prevent ice jams on the lower river. This belief is based on two observations: (1) a 1995 video shows a breakup ice run passing the intact Massena Weir; and (2) tree scar and sediment stratigraphy data indicate that several ice jams of sufficient magnitude to cause sediment scour occurred on the lower river before the weir was breached (Alcoa, 2004).

**Booms**

One vendor contacted as part of the evaluation proposed installing a series of steel pipe ice booms on the upper river to retain the breakup ice run and mitigate jams in the lower river. Although the concept has proven successful at several Canadian sites, it is unlikely that even multiple booms would reliably retain the ice run on the upper Grasse River due to the dynamic nature of the breakup, and associated high discharges and high water velocities.
Pier-Type ICS
The pier-type ICS is a natural evolution from earlier structures that utilizes a weir with piers to retain ice. Pier-type structures have proven to be reliable, are more cost effective, and are less environmentally obtrusive than a weir with piers. These structures function by retaining the breakup ice run at a safe location upstream of the area to be protected from jams. The state-of-the-art breakup ICS design consists of a row of cylindrical piers spaced 3 to 4.6 m (10 to 15 ft) apart across the river channel. The piers retain a significant portion of ice in the main channel and use an adjacent floodplain to pass relief flow around the jam created by the structure. Since most of the relief flow goes around the ice jam rather than beneath it, upstream water level rise and under-ice water velocity are greatly reduced as is the potential for ice passage between the piers and destabilization of the ice jam formed behind the structure.

Ice Breaking
A number of mechanical ice breaking options were explored including trenching the ice cover into a pattern of large floes and breaking up the lower river ice cover using conventional ice breakers, barge mounted excavators, amphibious excavators, and hovercraft or air cushion vehicles (ACVs).

Thermal Methods
The use of thermal effluent to melt the ice cover on the lower Grasse River was ruled out due to the lack of an economical source for the large volume of warm water that would be required. Surface treatment or dusting was ruled out as a reliable technique because, in the northeastern U.S., one usually cannot rely on sufficient late winter sunshine to deteriorate the ice cover in advance of breakup.

Preliminary Screening Evaluation - Summary
The preliminary screening of ice control options led to the identification of structural ice retention as a viable long-term approach for mitigating ice jams and ice jam-related scour on the lower Grasse River. Specifically, this would include the installation of a stand-alone pier-type ICS or integration of ice control into the design of the proposed MED hydroelectric project. Pier-type ICSs, which represent the state-of-the art in breakup ice control, are viable options for the Grasse River due to their reliability, relatively low cost and low environmental impact. The dam structure would provide reliable ice control for the lower river in addition to providing the benefits of hydropower production and enhanced recreational benefits and waterfront improvement opportunities for the local community. Ice breaking was also retained for further evaluation as a potential interim option.

3.0 Structural Ice Control Site Considerations

3.1 Technical Factors

Dam Structures
Simple weirs or small dams can in some cases retain a breakup ice run, but most of the ICSs built in the last three decades involve a weir with vertical piers added for ice retention. Technical factors in the siting of an ice retention weir or dam are similar to the factors considered for a pier-type structure (see below). A weir or dam designed specifically for ice control, however,
must also consider issues such as fish passage and the management of solids that may accumulate behind the dam.

Pier-Type Structures
Important factors in pier-type ICS structure site selection include hydraulic conditions, channel morphology, the ice regime, and the potential effects on upstream lands.

For the expected breakup discharge range, hydraulic conditions near an ICS must be sufficiently mild that a stable ice accumulation can exist upstream of the piers. Also, the under-ice water velocity must be low enough to avoid ice erosion and piping beneath the jam, which can lead to ice passage between the piers and jam failure. A number of successful breakup ICS designs take advantage of an adjacent floodplain area to bypass water flow around the jam that forms in the main channel. This relief flow mechanism limits upstream stage rise and prevents excessive water velocity in the ice jam toe region. Recent field observations and laboratory experiments indicate that the presence of an overbank relief flow channel approximately doubles the maximum discharge at which a pier ICS can retain ice (based on unpublished results of lab experiments at USACE CRREL and Laval University).

Other important site selection factors relate to pre- and post-breakup ice volumes, and the nature of the ice breakup. These factors are collectively referred to as the ice regime. The ICS must be located close enough to the problem area so that it retains sufficient ice to mitigate downstream ice jam flooding or under ice scour, depending on the purpose of the structure. This requires a good estimate of the maximum probable ice supply and knowledge of where the upstream ice originates, as well as the nature and sequence of breakup. For example, on some rivers, breakup may progress very rapidly down a long reach of river to form a single large jam at the downstream end. In this case, the portion of the total ice supply that melts or deposits along the banks (referred to as overbank deposition) may be relatively small. At the other extreme, breakup may occur as a downstream progressing series of jams and releases with significant en-route ice losses due to melting and overbank deposition. This latter regime is what has been observed for previous ice breakup events on the Grasse River.

Upstream effects are a critical issue, as the location selected for the ICS could result in upstream water levels in excess of pre-project conditions during the ice breakup period in years when a natural mechanical breakup occurs. Numerical and physical ice-hydraulic models are useful in analyzing upstream effects, as well as issues of ice accumulation stability and ice jam volumes, both at the ICS and at the original or natural ice jam location.

3.2 Non-Technical Factors

Beyond technical feasibility, constructability, and operational reliability, there are a number of non-technical factors which are considered in evaluating potential structural ice control locations, including but not limited to:

- Potential impacts to recreational river users, including safety considerations related to snowmobile use;
- Environmental impacts;


- Availability of land for ICS construction and for flowage easements;
- Overall community acceptance;
- Upstream impacts to land and structures due to river rise during an ice jam;
- Visibility and aesthetic impact of the ice control structure;
- Cost of construction and operation/maintenance; and
- Ability to obtain necessary regulatory permits.

3.3 Design Event for Grasse River ICS

The design event for an ICS consists of a conservative combination of river flow and ice supply. The March 28, 2003 ice jam was used as the design event in the engineering analysis of ice retention alternatives for the Grasse River. A sensitivity analysis using various combinations of ice thickness and river flows was conducted for the two structural ice control options (MED project and lower river pier-type ICS) that are currently under consideration. The basis for the design event and sensitivity analysis cases is described in Quadrini et al. (2008).

4.0 Site-Specific Assessment of ICS Locations for the Grasse River

4.1 Summary of Pier-Type ICS Locations Evaluated

Table 2 lists ICS sites and cases considered. Of the five upper river sites, the most favorable location from a technical standpoint is Site 1, located approximately 1 km (0.6 miles) upstream of the Route 37 Bridge. Interactions with the community related to the Site 1 location identified a number of concerns with the siting of a structure on this stretch of the river. Site 1 attributes include a relatively straight undeveloped stretch of river for retaining ice, and a broad floodplain to pass relief flow. Numerical simulations indicated that the ICS would retain a stable ice accumulation and about 70% of the flow would bypass the jam via the floodplain, avoiding excessive upstream stage rise or high under-ice water velocities. The simulations predict that for the design event (a 2003-like event scenario), an ICS at this location would retain sufficient ice volume to significantly lessen the likelihood of a scour-producing ice jam on the lower river.

For the lower river, an area between river transects T1 and T9 (Figure 1) was evaluated for potential ICS sites. A major difference between the lower and upper river sites is the lack of a natural floodplain for bypassing flow around the lower river structure. Without overbank flow relief, most of the water flow must pass beneath the ice accumulation, with a smaller portion passing through the jam as seepage flow. Preliminary simulations of lower river ice retention produced partially grounded jams with sufficiently high water velocities to erode the jam underside, a condition that would likely lead to piping and ice blowout between the piers. As a solution, the ice evaluation team developed the concept of an in-channel flow relief, which consists of a longitudinal row of piers to maintain an open water area along one side of the river channel for bypass flow. A row of piers across the channel would be joined to the longitudinal piers, forming an “L” shaped configuration.

The initial location considered on the lower river was at transect T9, approximately 1.05 km (0.65 miles) upstream of the location of the 2003 ice jam. DynaRICE simulations were also conducted upstream at transects T6 and T3. Based on the numerical simulations and in...
consideration of the above criteria, the general vicinity of location T6 offered the best compromise of the various siting criteria. In general terms, this location is the furthest upstream location which has the necessary physical characteristics to provide the required ice retention without causing potentially unacceptable backwater effects.

DynaRICE simulations at the T6 site found that both 100- and 200-m long flow relief channels reduced the under ice water velocities to non-erosive levels. Therefore, a relief channel length of 100 m (328 ft) was chosen as an appropriate design. The addition of in-channel flow relief had no effect on the upstream water surface profile beyond a short distance upstream of the structure. A final simulation at transect T6.75 (approximately 122 m (400 ft) downstream of T6) yielded similar results as the T6 case. The T6.75 site was chosen as the best candidate location in the T1 to T9 section of the river because its in-channel flow relief configuration takes advantage of a natural widening of the river along the north bank, which is expected to enhance its performance. A conceptual plan and cross section for an ICS at “Site T6.75” are presented as Figures 2 and 3. A detailed evaluation of the performance of a pier type ICS at this location was conducted through a physical model study conducted at CRREL and related DynaRICE modeling runs conducted at Clarkson University, the results of which are described in Tuthill et al. (2008a) and Kolerski et al. (2008), respectively.

Figure 2. Aerial view of the T6.75 ICS.
Table 2. ICS sites and locations evaluated.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DESCRIPTION</th>
<th>MODEL</th>
<th>RESULT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline simulation of 3/28/06 ice jam at T16</td>
<td>Design event</td>
<td>DynaRICE, HEC-RAS</td>
<td>Both models matched observed ice jam water levels reasonably well.</td>
</tr>
<tr>
<td><strong>UPPER RIVER SITES</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Site 1 (~1 km upstream of Route 37 Bridge)</td>
<td>Piers across channel, broad floodplain on left for bypass flow</td>
<td>DynaRICE, HEC-RAS</td>
<td>Stable ice accumulation in channel. Under-ice water velocity moderate. 70% of flow bypasses jam via floodplain.</td>
</tr>
<tr>
<td>Site 2 (~0.6 km downstream of Rod &amp; Gun Club)</td>
<td>Ice retained in main channel, adequate floodplain on left for bypass flow</td>
<td>DynaRICE, HEC-RAS</td>
<td>Stable ice accumulation in channel. Under-ice water velocity higher than Site 1. 70% of flow bypasses jam via floodplain.</td>
</tr>
<tr>
<td>Site 3 (1.1 km upstream of Rod &amp; Gun Club)</td>
<td>Ice retained in main channel, narrow floodplain on right for bypass flow</td>
<td>DynaRICE, HEC-RAS</td>
<td>Ice accumulation stability questionable. Under-ice water velocity extremely high.</td>
</tr>
<tr>
<td>Site 4, Louisville Bridge</td>
<td>Structurally enhance bridge's tendency to retain ice</td>
<td>Not modeled</td>
<td>Would not sufficiently reduce ice volume reaching lower river jam to make positive difference.</td>
</tr>
<tr>
<td>Restore Massena Weir to former height</td>
<td>Open water profiles with and without weir compared at low and high flows</td>
<td>HEC-RAS</td>
<td>In all cases, backwater extends to foot of Louisville rapids. At breakup flow, profiles nearly the same. Weir has little ice retention capability.</td>
</tr>
<tr>
<td>Ice retention with dam</td>
<td>Ice retained in main channel, no relief flow channel</td>
<td>DynaRICE</td>
<td>Stable ice accumulation in dam reservoir. Under-ice water velocities low. Storage capacity adequate for 2003 ice supply scenario.</td>
</tr>
<tr>
<td><strong>LOWER RIVER SITES</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ice retention at T9</td>
<td>Ice retained in main channel, no relief flow channel</td>
<td>DynaRICE, HEC-RAS</td>
<td>Similar ice jam profile to baseline simulation at T16, but displaced upstream. High under-ice water velocities near toe. Ice erosion likely.</td>
</tr>
<tr>
<td>Ice retention with piers at T9</td>
<td>Actual pier geometry included in model grid, no relief flow channel</td>
<td>DynaRICE</td>
<td>Similar to above case. Extremely high under-ice water velocities immediately upstream of piers. Ice erosion near piers likely.</td>
</tr>
<tr>
<td>Ice retention at T6</td>
<td>Ice retained in main channel, no relief flow channel</td>
<td>DynaRICE</td>
<td>Partially grounded jam, high under-ice water velocities, ice erosion likely. Jam stability questionable.</td>
</tr>
<tr>
<td>Ice retention at T3</td>
<td>Ice retained in main channel, no relief flow channel</td>
<td>DynaRICE</td>
<td>Partially grounded jam, high under-ice water velocities, ice erosion likely. Jam stability questionable.</td>
</tr>
<tr>
<td>ICS at T6 with relief channel on south bank</td>
<td>60, 100, and 200-m-long relief flow channel compared</td>
<td>DynaRICE</td>
<td>Partially grounded jams in all cases. Upstream water surfaces profiles nearly identical. Moderate under-ice water velocities in the 100- and 200-m-long relief channel cases.</td>
</tr>
<tr>
<td>ICS at T6.75 with relief channel on north bank</td>
<td>100-m-long flow relief channel to take advantage of river widening on north bank</td>
<td>DynaRICE</td>
<td>Very similar result to previous case with 100-m-long relief channel along south bank.</td>
</tr>
</tbody>
</table>
4.2 MED Project Siting Considerations

MED evaluated several locations and combinations of pool elevations in a one mile reach of the river between the site of the old Massena Weir just below the Main Street Bridge and the Massena Wastewater Treatment Plant (see Figure 4). The siting of their hydroelectric dam took into consideration the balance between potential power generation, pool size, and the costs of construction and land acquisition. The ability of the structure to store and retain ice during a breakup event for the locations under consideration was evaluated after the initiation of discussions on the potential to integrate ice control into the project.

A preliminary analysis indicates that the expected ice volume from the 2003 design ice jam event could be retained behind the proposed dam without a significant incremental rise in water levels, if underflow gates were provided. A detailed evaluation of the performance of a pier type ICS at this location was conducted through a physical model study conducted at CRREL and related DynaRICE modeling conducted at Clarkson University, the results of which are described in Tuthill et al. (2008b) and Kolerski and Shen (2008), respectively.
5.0 Non-Structural Ice Control Options

Nonstructural methods of ice control include mechanical, thermal, and chemical measures. Of these three methods, only mechanical measures of ice control (i.e. “ice breaking”) were seriously considered for use in the lower Grasse River, since thermal and chemical methods were determined to be less feasible and/or reliable.

Ice breaking was identified as a potential interim ice jam mitigation measure. The approach would be to break up the cover ice on the lower Grasse River in advance of the natural breakup on the upper river, which typically occurs between mid March and early April. Because of its low gradient, low water velocity, and northerly location, the lower Grasse River ice is thicker than the upstream ice cover and tends to stay in place longer. It is believed that ice jam formation in the lower river is largely associated with the presence of this intact ice cover. If no competent ice cover existed on the lower river, the arriving ice from upstream is expected to travel through this reach to the St. Lawrence River with little, if any, significant jamming, and the hazard of ice jam-related scour to sediments would be significantly alleviated.

A number of mechanical ice breaking techniques were evaluated including ice trenching, use of conventional icebreakers and tugboats, amphibious hydraulic excavators, barge mounted excavators, and ACVs. Trenching, which consists of cutting the ice cover into a pattern of large floes, was ruled out because it would not guarantee an open water condition on the lower river at the time of upper river breakup. Conventional screw-driven icebreakers and tugboats pose the concern of the vessel draft and the potential for the propeller wash to disturb the riverbed sediments.

Following an extensive investigation of potential ice breaking equipment and contractors, an ice breaking demonstration project was conducted in the spring of 2007 using conventional
excavators mounted on a barge. While the project was successful in clearing the ice from the majority of the targeted reach of the lower Grasse River, community safety considerations coupled with the number of uncontrollable factors that must converge for the activity to be successful resulted in a decision to not use ice breaking as an interim measure in future years. A detailed description of the Ice Breaking Demonstration project is provided in VanDewalker et al. (2008).

6.0 Summary and Conclusions

The evaluation of potential ice management options to prevent ice jam related sediment scour in the lower Grasse River identified two long-term structural ice control alternatives (pier type ICS and integration of ice management into a proposed hydroelectric dam). Armored capping is also under evaluation as a means by which to prevent ice jam-related sediment scour, with the cap designed to withstand ice jam-related forces.

An analysis of potential siting options indicated that suitable locations existed for both of the structural ice control options within the general vicinity of the lower Grasse River. Community input related to siting considerations in the upper Grasse River resulted in the need to focus on candidate sites in the lower Grasse River and develop an integrated flow bypass system for the pier type ICS in view of the lack of a natural floodplain in the vicinity of the lower River locations that were considered. Detailed evaluations of the ice management capabilities of these structures were conducted through both physical model studies conducted at CRREL and numerical DynaRICE modeling conducted by Clarkson University. The results of these studies indicated that both structural ice control options would be effective in mitigating ice jam related sediment scour on the lower Grasse River under design event conditions.

Ice breaking using barge mounted excavators was identified and tested as a possible interim measure for ice jam scour prevention. Results of a demonstration study indicated that while ice breaking could be successfully conducted, community safety considerations coupled with the number of uncontrollable factors that must converge for the activity to be successful resulted in a decision to not use ice breaking as an interim measure in future years.

References


DynaRICE Modeling to Assess the Performance of an Ice Control Structure on the Lower Grasse River

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This paper describes a two-dimensional numerical study using a dynamic ice transport model to analyze the design and effectiveness of an ice control structure on the Grasse River, Massena, NY. The structure was designed as an L-shape barrier with a flow relief channel along the northwest bank of the river. The effect of the length of the flow relief channel was first examined to assist in the selection of channel length. The ice retention capability of the ice control structure as well as the under ice current and the backwater effect of the ice accumulation were studied. A sensitivity analysis was carried out for several combinations of breakup flow and ice supply conditions with a 100-year return period. The effect of possible ice jam formations downstream of the control structure resulting from the possible ice leakage through the ice control structure was also examined. The findings presented in this paper represent work in progress only, as the final remedy at the Grasse River site has yet to be selected.
1. Introduction

Due to a severe breakup ice jam that occurred in March 2003, a portion of the riverbed was scoured in the vicinity of Alcoa’s capping pilot study area of the lower Grasse River (Alcoa 2004). To control further ice jams and to prevent the potential erosion of PCBs from the river bottom, a pier-type ice control structure (ICS) located upstream of the capped river section has been evaluated as one possible mitigation option. This paper presents the DynaRICE numerical simulation results on the ice retention capability and associated backwater effects of an ICS on the lower Grasse River at river transect T6.75, which is located approximately 0.7 km downstream of the Alcoa Bridge (approximately 1.6 km upstream of the pilot cap area; Figure 1). The model domain covers the 15.3 km reach upstream of the river mouth. The bathymetry in the vicinity of the ICS is given in Figure 2. All of the river level data are presented based on the USLS35 datum. The model has been calibrated with the 2003 breakup jam that occurred on the lower Grasse River (Shen and Liu 2005). The ICS was designed as L shape barrier with a flow relief channel along the northern bank of the river to reduce the under-ice water velocity. In addition to the simulations on the effectiveness of the ICS, the effect of possible ice jam formation downstream of the structure resulting from possible ice leakage through the ICS (Tuthill, et al. 2008) was also examined.

![Figure 1. Grasse River in Massena.](image)

2. Ice Control Structure Performance Analysis

Initial DynaRICE test runs indicated that ice retention without a relief channel could have relatively high under-ice currents with a velocity of about 2 m/s. Further analyses with 60-, 100-, or 200-m-long relief channels showed the under-ice flow velocity was significantly reduced when the channel length is 100 m or more. Based on these results, a relief channel length of 100 m was selected.
In all simulation cases, the ICS is assumed a fixed barrier, i.e. 100% retention of the ice entering the site from upstream was assumed and, thus, provides conservative estimates of water stage increase upstream of the ICS. In these cases, flow is passed under, around, and to a lesser extent through the ice accumulated in the vicinity of the piers. The model enables the assessment of whether or not an ice jam induced by the ICS was stable, but is not capable of assessing whether ice may ‘leak’ through the ICS between piers. The leakage potential was evaluated by the physical model (Tuthill, et al. 2008). The effects of ice leakage through the ICS on potential downstream jam formation are discussed in Section 3.

The input parameters for the cases simulated for the ICS were prepared based on discharge and ice supply parameters presented in Table 1 (Quadrini et al. 2008).

**Figure 2.** Bathymetry in the vicinity of the modeled ICS (In meters, USLS35).

**Table 1.** Breakup discharge and ice supply for sensitivity analysis simulations

<table>
<thead>
<tr>
<th>Case</th>
<th>Breakup Discharge m³/s (cfs)</th>
<th>Modeled Peak Discharge M³/s (cfs)</th>
<th>Cover Thickness m (inches)</th>
<th>Total Ice Supply Volume m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>178 (6,303)</td>
<td>242 (8,559)</td>
<td>0.59 (23.25)</td>
<td>237,888</td>
</tr>
<tr>
<td>Case A</td>
<td>300 (10,608)</td>
<td>408 (14,405)</td>
<td>0.10 (4)</td>
<td>39,648</td>
</tr>
<tr>
<td>Case B</td>
<td>281 (9,936)</td>
<td>382 (13,492)</td>
<td>0.25 (10)</td>
<td>101,952</td>
</tr>
<tr>
<td>Case C</td>
<td>259 (9,139)</td>
<td>351 (12,410)</td>
<td>0.41 (16)</td>
<td>164,256</td>
</tr>
<tr>
<td>Case D</td>
<td>209 (7,372)</td>
<td>283 (10,011)</td>
<td>0.56 (22)</td>
<td>223,728</td>
</tr>
<tr>
<td>Case E</td>
<td>190 (6,724)</td>
<td>259 (9,131)</td>
<td>0.61 (24)</td>
<td>246,384</td>
</tr>
<tr>
<td>Case F</td>
<td>93 (3,289)</td>
<td>126 (4,466)</td>
<td>0.73 (28.7)</td>
<td>294,528</td>
</tr>
</tbody>
</table>
2.1 Simulation results
Simulated water levels for all cases at hour 26, when the ice jam is fully developed, are summarized in Figure 3. The maximum water levels predicted to occur during the entire simulation period are summarized in Figure 4. The FEMA (1980) 100-year open water flood level is also shown in Figures 3 and 4 for comparison. Note the FEMA flood level limits cover only a portion of the numerical model domain, and included the effect of the old dam near the Main Street Bridge, which was not included in the ICS simulations.

Figure 3. Simulated water levels for all cases at hour 26

Figure 4. Maximum water levels predicted during the simulation period for all cases
Figures 3 and 4 shown that the predicted water level is a result of both water discharge and ice supply. Cases B and C with relatively high water discharge and ice supply produced the highest water levels. Figures 5 and 6 show the jam profile, water depth, and velocity underneath the jam for Cases A and F. These figures demonstrate the interaction between the water discharge, ice supply, and channel bathymetry. In Case A, the high water discharge and low ice supply resulted in a short jam with thick jam toe. In Case F, the low water discharge and high ice supply resulted in a long ice jam with much smaller jam toe thickness.

Figure 5. Case A – Simulated jam profile, and water depth and velocity under the ice jam at hour 26.

Figure 6. Case F - Simulated jam profile, and water depth and velocity under the ice jam at hour 26.

3. Effects of Ice Release through the Ice Control Structure
To evaluate the effects of possible ice jam formations resulting from the ice leakage through the ICS on the lower Grasse River, three jam locations downstream of the ICS that were susceptible of potential ice jam formation and bed scour were selected. These three locations were T8, T13, and T16. T8 was selected to examine the possible effects of a jam formed close to the ICS, T13 was selected because of the shallow channel depth, and T16 was selected because it was the location of the 2003 breakup jam. Ice was supplied from the ICS located at transect T6.75, which is the upstream boundary of the ice leakage effect simulations. The flow hydrograph of the 2003
ice jam event were used as the upstream flow boundary condition. The following ice leakage amounts through the ICS are used based on the physical model studies (Alcoa 2007):

1. Discharge associated with the 2003 event and an ice supply of 3,965 m$^3$ (140,000 ft$^3$) (solid ice volume) distributed over the duration of the simulation with ice jam initiation locations at T8, T13, and T16.

2. Repeat these three simulations with a 50% increase in the total ice supply to 5,947 m$^3$ (210,000 ft$^3$).

The 3,965 m$^3$ of ice volume is considered a conservative leakage volume because:

- It was the highest leakage volume observed in the physical models tests #5 through #15.
- The leakages predominantly occurred only at the quite high discharge of 337.4 m$^3$/s (11,914 cfs), which is nearly 40% greater than the peak discharge associated with the 2003 event.
- Melting during the experiments probably facilitated leakage episodes because of thinning and rounding of the ice pieces.
- While the ice supplies in the physical model tests were less than those associated with the cases in the numerical simulations due to limitations on the amount of ice that could be produced in the model facility, the clear trend in the experiments was less leakage occurred with higher ice supplies as shown in Figure 7.

![Figure 7. Total ice leakage observed in the experiments](image)

3.1 Simulation results
For both ice leakage cases, the model did not predict grounding for any of the three ice jam locations. The final ice jam profile and corresponding water velocity and depth underneath the jam at hour 26 are presented for the three cases with a 5,947 m$^3$ ice leakage volume. The velocity distribution at the time when the maximum velocity occurred is also presented in Figures 8, 9, and 10. Figure 11 shows the simulated water level profile at hour 26 for the case with a 20 cm thick continuous ice cover downstream of the ICS. The increases in water level at the ICS due to ice jam formations over the case with the continuous ice cover are less than 5 cm.
Figure 8. 5,947 m$^3$ leakage - Simulated ice jam profile at transect T8 at hour 26.

Figure 9. 5,947 m$^3$ leakage - Simulated ice jam profile at transect T13 at hour 26.

Figure 10. 5,947 m$^3$ leakage - Simulated ice jam profile at transect T16 at hour 26.
Table 2 summarizes the maximum velocity in the study reach and under the ice jam. For jams at T8 and T16 the maximum water velocity under the jam toe occur around hour 6 due to the occurrence of peak water discharge at that time. For jams at T13 the maximum water velocity under the jam toe occur at hour 16 and 18 for 3,965 and 5,974 m$^3$ ice leakages, respectively.

Table 2. Maximum water velocity in the study reach and under the jam

<table>
<thead>
<tr>
<th>Ice Volume m$^3$ (ft$^3$)</th>
<th>Jam Location</th>
<th>Maximum water velocity upstream of jam m/s (ft/s)</th>
<th>Maximum water velocity under jam m/s (ft/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,965 (140,000)</td>
<td>T8</td>
<td>0.60 (1.96)</td>
<td>0.49 (1.62)</td>
</tr>
<tr>
<td></td>
<td>T13</td>
<td>0.68 (2.23)</td>
<td>0.66 (2.16)</td>
</tr>
<tr>
<td></td>
<td>T16</td>
<td>0.82 (2.68)</td>
<td>0.53 (1.75)</td>
</tr>
<tr>
<td>5,974 (210,000)</td>
<td>T8</td>
<td>0.59 (1.94)</td>
<td>0.49 (1.61)</td>
</tr>
<tr>
<td></td>
<td>T13</td>
<td>0.75 (2.46)</td>
<td>0.75 (2.46)</td>
</tr>
<tr>
<td></td>
<td>T16</td>
<td>0.83 (2.71)</td>
<td>0.53 (1.74)</td>
</tr>
</tbody>
</table>

4. Summary
This paper presents the results of a numerical model study to evaluate the stability of ice accumulations upstream of an ICS at T6.75, backwater levels upstream of the ICS, and the effects of ice release through the ICS on the potential formation of ice jams downstream of the ICS. The performance of the T6.75 ICS was evaluated through simulations of the 2003 design event and several combinations of ice supply and river discharge with estimated recurrence intervals of 1-in-100 years. The simulation results show that the ICS would produce stable ice accumulations upstream of the structure under the range of conditions tested. The simulation results also show, for all cases, ice jams will be partially grounded upstream of the ICS, assuming all supplied ice is retained by the structure. Additionally, the ice accumulation just upstream of the ICS had a characteristic shape of a thickened triangular region with the apex at the corner of the ICS and a resultant channeling of the flow towards the upstream end of the
relief channel piers and the south side of the river. This behavior is virtually identical to that observed in the physical model study (Tuthill et al. 2008).

Comparison of the maximum water levels predicted by DynaRICE show that Cases B and C, with the relatively high discharges and medium ice thicknesses and ice supplies, yielded the highest water levels just upstream of the ICS. Case F, with a relatively low discharge but the highest ice supply, yielded the highest water levels at about river km 11.8 but only a little higher than the other cases. Case F also had the furthest upstream progression of the ice jam induced by the ICS, which is due to the ice accumulation having a lesser thickness and thus greater upstream extent. Even though rather large variations in ice supply and discharge were used in combinations in the various cases simulated, the water levels are similar, suggesting that the water levels are not sensitive to the different kinds of events.

Comparisons to the FEMA 100-year open water flood levels show that the maximum predicted backwater levels produced by the ice accumulations exceed the 100-year open water flood level in the approximately 1.9 km (1.2 mile) stretch of river upstream of the ICS (i.e., upstream to about km 11.8 [river mile 7.3]). Upstream of this location, the maximum predicted backwater levels are lower than the 100 year open water flood level.

The evaluation of the effects of ice release through the T6.75 ICS on the potential formation of jams downstream of the ICS was conducted in response to observations made during the physical model testing, which indicated that some of the supplied ice passed through the ICS piers under certain test conditions. Model simulations were made based on conservative ice leakages of 3,965 and 5,974 m$^3$ (solid ice volume) distributed over the duration of the simulation with ice jam locations at T8, T13, and T16. The water discharge is assumed to be the same as the 2003 design event.

For both ice leakage cases the model did not predict any grounding for all three jam locations. Water level at the ICS with ice jam formation is very close to the water level for a 20 cm (7.9 in.) thick continuous ice cover downstream of the ICS. The increases in water level at the ICS due to ice jam formations over the case with the continuous ice cover are less than 5 cm (2 in.). In all cases, the velocity in the immediate area underneath the jam is lower than 0.69 m/s (2 ft/s), except for the T13 ice jam site where the water depth is relatively shallow. The current velocities underneath the jam toe at T13 have a maximum value of 0.66 m/s (2.16 ft/sec) and 0.75 m/s (2.46 ft/sec) for 3,965 and 5,974 m$^3$ ice leakages, respectively. These velocities are slightly higher than the case without a jam, which has a maximum velocity of 0.62 m/s (2.03 ft/sec). Outside the jam toe area, there are areas with water velocity exceed 0.69 m/s (2 fps). However, they are not caused by the ice jam. For all cases at T8 and T16 water velocity reaches its maximum around hour 6, which coincides with the peak flow discharge. For the ice jam at T13, the maximum velocity occurred under the jam toe at hours 16 and 18 for ice leakage volumes of 3,965 and 5,974 m$^3$, respectively. The ice jam accumulation produced some backwater effect, which can cause a slight reduction of water velocity from it peak time outside the jam toe area.

5. Acknowledgements
This study was funded by Alcoa and overseen by Camp Dresser and McKee (CDM), Quantitative Environmental Analysis (QEA), and a team of ice experts (George Ashton,
Guenther Frankenstein, and Andrew Tuthill). The authors thank all team members for their valuable comments on this study.

References


Grasse River Ice Control Structure, Physical Model Study

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This paper describes a 1:30-scale physical model study with real ice to assess performance of a pier-type ice control structure (ICS) located at T6.75 in the lower Grasse River. The new type of ICS design consists of a row of transverse piers across the river channel and a second row of longitudinal piers parallel to the left bank. This L-shaped configuration doubles the total length of piers, reducing under-ice water velocity and providing relief flow around the jam that forms at the transverse piers. The model structure retained ice for a conservative range of ice and hydraulic conditions. Results of interest included non-uniform ice thickness distributions found upstream of the ICS and the development of preferential high-velocity flow channels beneath and through the ice accumulation. Near-field physical model results compare reasonably well to parallel DynaRICE simulations. The physical model study results, supported by the numerical simulations, increased confidence in the anticipated performance of this new type of ICS design. The findings presented in this paper represent work in progress only, as the final remedy at the Grasse River site has yet to be chosen.
1. Introduction

The lower Grasse River below the Village of Massena, New York, is the subject of environmental investigations by Alcoa due to the presence of polychlorinated biphenyls (PCBs) in river sediments and fish. In 2001, a pilot test cap was constructed over a 230-m length of the lower Grasse River to study the feasibility of subaqueous capping of the PCB-containing sediments. A severe ice jam on March 28, 2003, caused hydraulic scour of the pilot cap and, in some instances, the underlying sediments. Because the phenomenon of ice-jam-related sediment scour in the lower Grasse River was not known prior to this event, the pilot cap was not designed to withstand the associated forces. Subsequent efforts analyzed the magnitude of the 2003 event, and investigated possible mitigation measures (Alcoa 2004). Initial results from these studies indicated that similar ice jam scour events could be mitigated by retaining the breakup ice run at a location about 1.6 km upstream of the pilot cap area (Figure 1).

Simulations using the DynaRICE ice-hydraulic numerical model (Shen et al. 2000) indicated that a pier-type ice control structure (ICS) would create a nearly grounded ice jam at location T6 (Figure 1), and the 1.4-km-long channel from there upstream to the foot of the Massena Rapids could store the estimated 303,000-cubic-meter ($m^3$) ice volume of the March 2003 jam\(^1\). Three design considerations that could not be fully evaluated in the numerical model simulations were 1) the three-dimensional ice-structure interaction and the potential for ice release between the piers; 2) calculated under-ice water velocities that were above the assumed ice erosion threshold; and 3) the absence of natural floodplains near the site to convey relief flow around the jam in the main channel. The latter design consideration is critical since successful pier-type ICS designs to date rely on this relief flow mechanism to prevent excessive ice jam thickening, limit under-ice water velocities, and maintain ice accumulation stability. Physical model tests were therefore warranted to more fully evaluate the performance characteristics of an ICS in this river section and improve confidence in the results of the numerical simulations.

Because of the lack of natural floodplains in the proposed ICS area, the conceptual design included an engineered relief flow channel in conjunction with transverse piers across the river channel. This new concept consisted of a longitudinal row of piers extending upstream from the structure approximately parallel to the left bank. Flow would escape the ice-filled main channel to pass between the longitudinal piers, and exit via the open water relief flow channel (Figure 2). Initial DynaRICE tests found ice retention at T6 without relief flow to be questionable because of relatively high under-ice water velocities (about 2 m/s) and partial ice jam grounding upstream of the piers. Subsequent numerical simulations of ice retention with 60-, 100-, and 200-m-long relief channels found under-ice water velocities significantly reduced for the latter two cases. Based on these results, a relief channel length of 100 m was selected for detailed evaluation in the CRREL physical model study. The ICS location was moved about 100 m downstream of T6 to a location called “T6.75” to take advantage of a natural widening of the channel (Figures 1 and 2).

A 1:15-scale flume study preceded a 1:30-scale full width “river model.” The purpose of the larger scale model was to examine in detail ice accumulation stability and ice retention characteristics in the immediate vicinity of the piers without the effect of flow relief. The flume

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\(^1\) The 304,000 $m^3$ refers to the volume of solid ice in the jam $V_i$. Assuming an ice jam porosity $e$ of 0.4, the total ice jam volume $V_j = V_i/(1-e) = 506,700$ $m^3$. 

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study validated the initial DynaRICE simulations, finding that, under the discharge and ice conditions of the March 28, 2003, ice event, the stability of the ice accumulation retained behind a transverse row of piers alone would be marginal. The observed lower threshold for erosion of ice pieces from the model jam underside fell in the 1.2- to 1.8-m/s (prot.) range. Ice piece size distribution and ice strength did not greatly affect the ice retention capacity of the model piers. For the cases where the ice was retained, ice accumulation thickness and under-ice flow depth were fairly uniform across the flume width.

2. Approach

This paper focuses on the larger 1:30-scale river model, which evaluated the performance of the proposed ICS design for a conservative range of ice and hydraulic conditions. The model domain encompassed the entire river width, including the relief flow channel, and covered a channel length of about 1 km (Figures 1 and 3). Important issues investigated included the ice holding capacity of the piers, the stability of the retained ice accumulation, the flow conveyance of the relief channel, and ice forces on the piers. Also considered were possible failure modes, upstream water level rise, and under-ice water velocities and depths as they relate to ice erosion and the design of bed and bank protection. Important modeling parameters include river discharge, ice discharge and ice jam volume, as well as ice thickness, ice piece size distribution, and ice strength. Table 1 summarizes the range of discharge and ice conditions tested.

The March 28, 2003, ice jam, because of its high magnitude and low frequency was considered the design event in the physical model testing of the ICS design. Model tests used a steady discharge of 215 cms which was the average flow during the 21-hour ice jam formation period and also very close to the average of the breakup and peak discharges for the day of the event. Conservatism was added by bracketing the design discharge of 215 cms with the averages of the breakup and peak discharges for cases F and B (110 and 332 cms respectively) from the 100-year recurrence interval sensitivity analysis (Quadrini et al., 2008).

Additional conservatism was gained through the use of an open water initial condition (IC). The manner in which the breakup ice run impacts the piers is critical. At one extreme, ice floes may move through open water before impacting the piers, and their retention will depend on ice arching or grounding. Much more likely, the advancing breakup front will fracture the sheet ice upstream of the ICS and push large floes against the piers. These broken sheets will help retain the smaller floes in the upstream ice accumulation. Both of these initial conditions were tested.

Figure 3 shows the layout of the physical model. The model surface consisted of a 3-cm-thick layer of concrete formed over sand fill between plywood templates. Model bathymetry came from existing-conditions multibeam sonar data below the water level and ortho-photo contour mapping above. Existing conditions were modeled, except for the relief channel, which was deepened slightly to increase conveyance capacity. The addition of stone armor raised the existing bed elevation by 0.6 m in the vicinity of the ICS. The physical model ultimately had 24 rectangular transverse (T) piers (6.1 m high, 0.91 m wide, and 3.0 m long) and 26 1.5-m-diameter cylindrical longitudinal (L) piers, all with 3.66-m gaps in between (Figure 2). Pier height was initially set at 48.8 m USLS 1935, based on ice accumulation top elevations in the DynaRICE simulations.
During the tests, water flow entered the model through a large plastic ice-making tank. Because ice strength is proportional to the length scale, the model ice was weakened by adding 1% by weight urea to the water. Prior to testing, the room was cooled to –12ºC to form an ice sheet of the desired thickness. Before each test, this ice cover was mechanically broken into a target piece-size distribution. The room temperature was then raised to 1.7ºC to represent breakup conditions, and beam tests were done to determine the ice strength. Flow was increased sufficiently to convey the ice pieces from the tank into the river channel where they were retained behind a boom, or against the leading edge of the sheet ice, depending on the initial conditions. At the start of the open water initial conditions tests, the boom was removed, allowing the floes to drift toward the piers. An array of six pressure transducers collected time series water level data, and instrumented load cells on piers 1, 6, 11, 16, and 21 measured overturning moments. Spot water velocities were measured at critical locations using a miniature Marsh-McBirney electromagnetic flow meter.

3. Model Tests

Table 1 summarizes test conditions and results. Summary data include test duration, maximum water level rise, maximum ice accumulation thickness, maximum forces on the piers, and maximum measured water velocities. The discussion of results below focuses on tests (12–15) since they best represent extreme design conditions. Of the final four tests, Tests 12 and 14 had the highest starting ice volume and Test 13 was the longest in duration. Detailed results are presented for Tests 14 and 15. These two tests illustrate ice accumulation processes for the open water and broken sheets initial conditions, respectively.

Test 14 represents an extreme case with the open water initial condition, weak (230 kPa) thick (41 cm) ice, and the highest starting ice volume of 160,000 m³. Test 14 included a surge rather than a gradual increase from 215 to 332 cms. Figure 4 presents time series water level and discharge data. At 110 cms, the ice accumulated behind the piers and thickened to 2.3 m. Following the flow increase to 215 cms, the ice accumulation shove-thickened to approximately 3.4 m. After the surge to 332 cms, the ice floes grounded in the triangular area formed by the transverse and longitudinal piers, diverting the bulk of the water flow to each side. Figure 5 shows a photo and Figure 6 plots the ice accumulation profile at the time of peak water levels.

Test 15 represents an extreme case for the broken sheet initial condition. The ice is relatively weak at 450 kPa with a thickness of 30 cm, comparable to the 2003 ice event. At 110 cms, the ice floes accumulated against the broken sheets, subsequently thickening to about 3.0 m. Following the flow increase to 215 cfs, the accumulation shove-thickened to 3.8 m, and floes began to erode from the jam underside, transport, and deposit beneath the broken sheets as shown in Figure 7. At 215 cms, approximately half the water flow shifted left toward the upstream end of the relief flow channel as ice floes accumulated beneath the sheets in the triangle area. Following the increase to 332 cms, a third channel eroded through the mass of accumulated floes to convey water down a path to the right of the L-piers. Figure 8 plots the ice accumulation profile at the time of peak water levels. Note the greatest water level rise is about 2.1 m, which is 0.24 m less than the stage increase in open water IC Test 14. The maximum under-ice water velocities were on the order of 1.5 m/s, comparable to those observed in the Flume Model.

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2 Piers are numbered left to right facing downstream.
An interesting result was the development of high-velocity flow paths beneath and through the ice accumulation. These features developed as the upstream ice accumulation shove-thickened and additional floes were entrained beneath the broken sheets. As one channel became blocked with ice floes, the water and ice would seek a different path in a process analogous to channel shifting in a braided river. Zabilansky et al. (2002) observed a similar process during ice formation on the upper Missouri River. This phenomenon of shifting flow paths was seen in most of the broken sheets IC tests.

Maximum water velocity data from Tests 12 through 15 are also listed in Table 1. The highest surface water velocities were measured in the relief flow channel (≤ 3.4 m/s) and downstream of the right-hand three T-pier gaps (≤ 3.0 m/s) at a flow of 332 m/s. The highest mid-depth velocities of about 2.5 m/s were measured during Test 14 near the upstream end of the T-piers. Relatively high water velocities were also estimated from tracking ice particles moving beneath the broken ice sheets during Test 13 (≤ 2.4 m/s).

4. Conclusions

In the model tests, the proposed ICS reliably retained ice under ice and hydraulic conditions considerably more severe than the March 28, 2003, ice jam. The 2003 event occurred at an average discharge of 215 cms with an intact ice cover on the lower river. The model ICS performed acceptably at discharges as high as 332 cms, with the much more conservative open water initial condition upstream of the piers. Based on previous analyses, the probability of a breakup flow as high as 332 cfs, or an open water initial condition on the lower river, is very low, and the combination of the two conditions during breakup is much lower. For these reasons, the design of the model test schedule is considered to be conservative.

In cases of limited ice supply, or following periods of prolonged melting, some bleed-through of small ice floes occurred. Short-duration ice releases were also observed in higher ice volume tests, following ice shoves against the piers. In both cases, the volume of the ice passing the piers represented a small fraction of the starting ice volume (< 5%).

For the range of conditions tested, ice strength did not appear to affect the ice retention capacity of the piers. Similarly, the initial range of ice floe dimensions had no discernible effect on ICS performance.

Unlike the flume study where ice thickness and under-ice flow depth were relatively uniform, these two parameters were quite variable in the River Model. Typically an area of thick or grounded ice would occupy the triangular area formed by the two rows of piers, diverting the faster water flow to the right side of the main channel and also toward the upper end of the flow relief channel. These paths of fast under-ice flow would shift in location as they became ice-congested. As a result of this flow-shifting phenomenon, it is difficult to predict where the maximum water velocities will occur. Therefore, for the portions of the bed area within about 180 m upstream of the T-piers where armoring would be necessary, the armor layer would need to be designed to withstand flow velocities as high as 2.4 to 3.0 m/s with the added turbulence resulting from ice cover roughness.
Maximum measured surface water velocities were on the order of 3.0 m/s, the highest of which occurred in the flow relief channel (3.4 m/s) and downstream of the right-hand T-piers. Maximum under-ice water velocities of 2.4 m/s were estimated by tracking small ice particles moving beneath the broken ice sheets. The highest surface water velocities observed along the banks upstream of the ICS were about 1.8 m/s. From water velocity and depth data, it is estimated that about three-quarters of the total river discharge will pass through the relief channel at the 332-cms flow with the open water initial condition. For the same discharge and the broken ice sheet initial condition, a little over half the total river flow is expected to pass through the relief channel.

The originally proposed pier alignment, spacing, and height were close to optimal, requiring only minor changes. Six heightened piers were added to the upstream end of the L-piers to prevent ice flanking and overtopping at the upstream end of the relief flow channel.

The maximum calculated downstream force of 695 kN occurred at a T-pier near the right side of the channel. The maximum ice load on a pier for the open water initial condition tests was about half the above value at 365 kN. Based on the model results and preliminary ice crushing force calculations, ice loads on the ICS due to ice retention are expected to be less than those resulting from other design conditions, such as crushing of large floes against the piers or thermal expansion of the pre-breakup ice sheet.

5. References


6. Acknowledgements

This study was funded by Alcoa and overseen by Camp Dresser McKee (CDM), Quantitative Environmental Analysis (QEA), and a team of ice experts including Hung Tao Shen and Guenther Frankenstein. The authors thank all team members for their guidance in this study.
They also acknowledge the support of CRREL Engineering Research Branch members in the construction and operation of the physical models.

Table 1. Test Summary

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<th></th>
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<td>110 215 332</td>
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<td>110 215 332</td>
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<tr>
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<td>110 215 332</td>
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<td>4.6</td>
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<tr>
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<td>110</td>
<td>110 215 332</td>
<td>4.3</td>
<td>1.5</td>
<td>4.6</td>
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<tr>
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<td>1.0E+05</td>
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<td>—</td>
<td>—</td>
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</tr>
<tr>
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<td>110 215 332</td>
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<td>—</td>
<td>—</td>
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<td>9.9</td>
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<td>110 215 332</td>
<td>9.7</td>
<td>2.1</td>
<td>1.8</td>
<td>695</td>
<td>2.1</td>
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</table>
Figure 1. Site Map

Figure 2. Conceptual plan of pier ICS with in-channel relief flow.
Figure 3. Plan view of the CRREL Refrigerated Research Area showing model layout. Pressure transducers are numbered in red.

Figure 4. Test 14 water levels and discharge.
Figure 5. Ice accumulation at ICS; Test 14 following surge to 332 cms.
Open water initial condition.

Figure 6. Ice accumulation profile and peak water levels; Test 14 at 332 cms.
Open water initial condition.
Figure 7. Ice floes transporting and depositing beneath broken sheet ice in Test 15.

Figure 8. Ice accumulation profile and peak water levels; Test 15 at 332 cms. Broken sheets initial condition.
Numerical Modeling of Ice Retention and Upstream Effects of a Small Hydroelectric Dam on the Grasse River

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In this paper, the ice retention capability and effectiveness of a proposed small hydroelectric dam on the Grasse River at Massena, NY were assessed using the dynamic ice transport numerical model DynaRICE. The ice retention capability as well as the under ice current and the backwater effect of the ice accumulation of the MED Dam were studied. A sensitivity analysis was carried out for several combinations of breakup flow and ice supply conditions with a 100-year return period. The results showed the dam to be an effective barrier to the breakup ice run and a possible means of mitigating ice jams and ice jam scour on the lower Grasse River. The findings presented in this paper represent work in progress only, as the final remedy at the Grasse River site has yet to be selected.
1. Introduction

Severe ice jams, such as the one that occurred in 2003, have the potential to scour PCB-containing sediments from the upper 2.9 km of the lower Grasse River (Alcoa 2004). Ice control measures aimed at preventing ice jam-related scour in the lower river are currently being examined. One possible approach is the integration of ice control into a small hydroelectric dam being considered in the Village of Massena by the Massena Electric Department (MED) (MED, 2006). This paper presents the DynaRICE (Shen et al. 2000, Liu et al. 2006) numerical simulation results on the ice retention capability of the proposed MED hydropower dam at a location 30.5 m (100 feet) upstream of the existing Foot Bridge (Figure 1) on the Grasse River in Massena, New York.

The simulation model domain includes the 4 km (2.5 mile) reach upstream of the proposed dam site, as shown in Figure 2. Downstream of the Main Street Bridge is the old Massena Weir, which was intact until the mid-1990s when its breaching was initially observed (Alcoa 2001). The breached weir remains in place today, but does not retain water to any significant extent.

Figure 1. Grasse River MED dam site.
The modeling was based on normal pool elevation provided by MED, referenced to the North American Vertical Datum of 1988 (NAVD 1988). Since the numerical model DynaRICE used in this study is a two-dimensional model, the detailed dam and gate geometry are not modeled. The “boom option” in DynaRICE is used to simulate the ice control. The DynaRICE simulations used the following 4 criteria to control the ice jam simulation with a barrier. If any one of these criteria is exceeded, the model will allow ice to pass the barrier.

- Critical Froude number for ice entrainment – in all cases Fr = 0.09 was used;
- Ice erosion velocity – in all cases the under jam ice erosion velocity, V_{eros} = 1.5 m/s (4.92 ft/s), was used;
- Critical line load – in all cases this parameter was assumed to have a very large value, i.e. a fixed boom/barrier is assumed, and there will be no ice spill over the top of the boom;
- Boom/barrier depth – 3.35 m (11 ft) was used. When bottom of the jam toe is above the lower edge of the barrier, no ice erosion will occur at the barrier.

For a 3.35 m (11 ft) barrier depth and a pool level of 54.3 m (178 ft), the lower edge of the barrier, i.e. the top of the gate opening level is at 50.9 m (167 ft).

2. 100-Year Open Water Flood Condition
The water surface profile for 100-year open water flood was calculated by FEMA in a Flood Insurance Study for the Village of Massena (FEMA 1980). This flood condition serves as a reference for examining the backwater effect of ice accumulation behind the MED dam. For the FEMA study, the 100-year discharge for Grasse River in Massena was estimated at 427 m$^3$/s (15,080 cfs), and water levels were calculated for the 3.04 km (1.89 mile) long reach from the Railroad Bridge up to Town Line Road. For the present study, the two-dimensional DynaRICE model was calibrated to match the FEMA 100-year open water level prior to the simulation of ice dynamics in the river.
2.1 Model calibration (with original weir in place)
For this calibration the DynaRICE model domain contains a 15.3 km (9.5 mile) long reach of the lower Grasse River from its mouth up to the Route 37 Bridge. For the calibration to the 1980 FEMA condition, the old intact weir was located at km 12.65 with a top of weir elevation of 54.05 m (177.34 ft). The model was then calibrated by adjusting the bed Manning’s coefficient until agreement with the FEMA water level profile was obtained. The bed Manning’s coefficient and the water surface profile for this calibration are shown in Figure 3. In most of the model domain the roughness coefficient is 0.025, except for the short rapid section near and downstream of the weir where the Manning’s coefficients are higher.

![Diagram showing water level and velocity profiles with different Manning’s coefficients.](image)

**Figure 3.** 100-year open water flow for 1980 FEMA condition

2.2 100-year open water flood for current condition (with breached weir)
The 100-year open water condition for the current weir configuration (i.e., breached weir) was calculated using the same bed roughness coefficients determined from the 1980 condition calibration. The current breached weir configuration was included in both the 100-year open water flood, as well as for all of the subsequent ice simulations. The result of the simulation is shown in Figure 4. Simulation of the breached weir results in lower water levels upstream of the weir relative to the intact weir condition in 1980. In the vicinity of the weir, water level decreased by about 2.1 m (7 ft) due to the effect of the breaching. At the upstream end of the domain (2.4 km or 1.5 mile upstream of the weir) the water level for current condition is about 0.3 m (1 ft) lower than that of the case with the intact weir in place.
3. Ice Retention Analysis
The simulations conducted on the ice retention capability of the proposed MED dam were prepared based on the discharge and ice supply parameters presented in Quadrini et al. (2008). The ice model parameters were calibrated with the breakup ice jam that occurred in the lower Grasse River in 2003 (Shen and Liu 2005). Table 1 summarizes the cases simulated in this paper. The modeled discharge hydrographs for Cases A through F representing the ice jam formation period were constructed by scaling the March 2003 calibration period hydrograph. All simulation cases were for a design pool level of 54.3 m (178 ft).

Table 1. Cases simulated for MED ice retention capability analysis

<table>
<thead>
<tr>
<th>Cases</th>
<th>Ice Volume m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Peak Flow &amp; Ice thickness)</td>
<td></td>
</tr>
<tr>
<td>2003 (242 m³/s : 59.1 cm)</td>
<td>238,000</td>
</tr>
<tr>
<td>Case A (408 m³/s : 10.2 cm)</td>
<td>40,000</td>
</tr>
<tr>
<td>Case B (382 m³/s : 25.4 cm)</td>
<td>102,000</td>
</tr>
<tr>
<td>Case C (351 m³/s : 40.6 cm)</td>
<td>164,000</td>
</tr>
<tr>
<td>Case D (283 m³/s : 55.9 cm)</td>
<td>224,000</td>
</tr>
<tr>
<td>Case E (259 m³/s : 61.0 cm)</td>
<td>246,000</td>
</tr>
<tr>
<td>Case F (126 m³/s : 72.9 cm)</td>
<td>295,000</td>
</tr>
</tbody>
</table>

3.1 Simulation results
Simulations for Cases A through F and the 2003 “Design Case” for a pool level of 54.3 m (178 ft) were conducted for a duration of 26 hours, when the ice jam is fully developed. Ice jam and
water surface profiles are obtained for each case when maximum water levels are reached. Also included in these results are water surface levels at three distinct comparison points referred to herein as: west side of Tamarack Street, west side of Water Street, and the upstream model boundary. These locations are approximately 0.39 km (1,270 ft), 1.37 km (4,490 ft), and 3.89 km (12,750 ft), upstream of the MED dam site, respectively, as shown in Figure 2.

In all simulated cases the entire ice volume was retained by the dam, as shown in Table 2, and the water velocity underneath the jam toe did not exceed the erosion velocity limit of 1.5 m/s (4.92 ft/s). Table 3 summarizes the maximum water levels reached at any time during the respective modeling runs at the three comparison point locations.

**Table 2.** Comparison of ice volume in model domain at hour 26

<table>
<thead>
<tr>
<th>Cases (Peak Flow &amp; Ice thickness)</th>
<th>Entering model domain</th>
<th>Remaining behind dam</th>
<th>Passed dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003 (242 m$^3$/s : 59.1 cm)</td>
<td>238,000</td>
<td>238,000</td>
<td>0</td>
</tr>
<tr>
<td>Case A (408 m$^3$/s : 10.2 cm)</td>
<td>40,000</td>
<td>40,000</td>
<td>0</td>
</tr>
<tr>
<td>Case B (382 m$^3$/s : 25.4 cm)</td>
<td>102,000</td>
<td>102,000</td>
<td>0</td>
</tr>
<tr>
<td>Case C (351 m$^3$/s : 40.6 cm)</td>
<td>164,000</td>
<td>164,000</td>
<td>0</td>
</tr>
<tr>
<td>Case D (283 m$^3$/s : 55.9 cm)</td>
<td>224,000</td>
<td>224,000</td>
<td>0</td>
</tr>
<tr>
<td>Case E (259 m$^3$/s : 61.0 cm)</td>
<td>246,000</td>
<td>246,000</td>
<td>0</td>
</tr>
<tr>
<td>Case F (126 m$^3$/s : 72.9 cm)</td>
<td>295,000</td>
<td>295,000</td>
<td>0</td>
</tr>
</tbody>
</table>

**Table 3.** Comparison of simulated maximum water levels

<table>
<thead>
<tr>
<th>Cases (Peak Flow &amp; Ice thickness)</th>
<th>Water level (m, NAVD 1988) and hour of occurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>West Side of Tamarack Street (390 m upstream of dam)</td>
</tr>
<tr>
<td></td>
<td>ft</td>
</tr>
<tr>
<td>2003</td>
<td>55.01</td>
</tr>
<tr>
<td>Case A</td>
<td>55.17</td>
</tr>
<tr>
<td>Case B</td>
<td>56.05</td>
</tr>
<tr>
<td>Case C</td>
<td>56.16</td>
</tr>
<tr>
<td>Case D</td>
<td>55.30</td>
</tr>
<tr>
<td>Case E</td>
<td>55.33</td>
</tr>
<tr>
<td>Case F</td>
<td>54.29</td>
</tr>
</tbody>
</table>

In all cases, a thick ice jam toe is located at the MED Dam, except for Case F, where thicker ice accumulation occurred in the Water Street area. This situation is due to the lower flow velocity caused by the smaller water discharge and the larger channel depth in the vicinity of the Parker Avenue Bridge (about 0.8 km or 0.5 mile upstream of the MED dam). Figures 5, 6, and 7 show the effect of different combinations of water discharge, ice supply, and channel bathymetry on ice jam and water level profiles.
Figure 5. Case C-Simulated ice jam profile at hour 26.

Figure 6. Case E-Simulated ice jam profile at hour 26.

Figure 7. Case F-Simulated ice jam profile at hour 26.
**Figure 8.** Simulated water level (NAVD 1988) for all cases at hour 26

**Figure 9.** Simulated maximum water level (NAVD 1988) for all cases
Table 4. Predicted average under-ice velocities for each numerical simulation

<table>
<thead>
<tr>
<th>Case</th>
<th>Breakup Discharge (m$^3$/s)</th>
<th>Modeled Peak Discharge (m$^3$/s)</th>
<th>Cover Thickness (cm)</th>
<th>Average Under-Ice Velocity at Jam Toe (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>178</td>
<td>242</td>
<td>59.1</td>
<td>0.52</td>
</tr>
<tr>
<td>Case A</td>
<td>300</td>
<td>408</td>
<td>10.2</td>
<td>0.73</td>
</tr>
<tr>
<td>Case B</td>
<td>281</td>
<td>382</td>
<td>25.4</td>
<td>0.67</td>
</tr>
<tr>
<td>Case C</td>
<td>259</td>
<td>351</td>
<td>40.6</td>
<td>0.55</td>
</tr>
<tr>
<td>Case D</td>
<td>209</td>
<td>283</td>
<td>55.9</td>
<td>0.49</td>
</tr>
<tr>
<td>Case E</td>
<td>190</td>
<td>259</td>
<td>61.0</td>
<td>0.52</td>
</tr>
<tr>
<td>Case F</td>
<td>93</td>
<td>126</td>
<td>72.9</td>
<td>0.34</td>
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</tbody>
</table>

Figure 8 shows the water level profiles at hour 26 for all cases. The maximum water level that occurred at any time during each simulation is presented in Figure 9. Table 4 shows the average under ice water velocity that occurred in the vicinity of the MED Dam. This average velocity was calculated in the area where ice accumulation is thicker than 0.5 m (20 inches) and for the hour when the velocity reached its maximum at the jam toe.

4. Summary
This study assessed the ice retention capability and effectiveness of the MED structure using the dynamic ice transport numerical model DynaRICE. The ice retention capability as well as the under ice current and the backwater effect of the ice accumulation of the MED dam were examined. A sensitivity analysis was carried out for several combinations of breakup flow and ice supply conditions with a 100-year return period. The results showed the dam could be an effective barrier to the breakup ice run and a possible means of mitigating ice jams and ice jam scour on the lower Grasse River. Further physical model study to validate the numerical model results and evaluate near-field conditions in the close proximity of the MED dam is presented by Tuthill et al. (2008).

5. Acknowledgements
This study was funded by Alcoa and overseen by Camp Dresser and McKee (CDM), Quantitative Environmental Analysis (QEA), and a team of ice experts (George Ashton, Guenther Frankenstein, and Andrew Tuthill). The authors thank all team members for their valuable comments on this study.

References


Physical Modeling of Ice Retention at a Small Hydroelectric Dam on the Grasse River

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Ice retention capability and upstream effects of a small hydro dam on the Grasse River at Massena were assessed in a physical model study with natural ice. For a conservative range of ice and hydraulic conditions, the model tests showed the dam to be an effective barrier to the breakup ice run and a possible means of mitigating ice jams and ice jam scour on the lower Grasse River. The lab tests verified ice retention assumptions used in parallel DynaRICE simulations such as under-ice erosion velocity and gate opening depth. An interesting result was that the dam retained ice in situations where water velocity exceeded 1.5 m/s and the bottom of the ice accumulation a short distance upstream was below the tops of the gate openings. Comparisons between physical and numerical model results are presented and discussed. The findings presented in this paper represent work in progress only, as the final remedy at the Grasse River site has yet to be chosen.
1. Introduction

The lower Grasse River downstream of the Village of Massena, New York, is the subject of ongoing environmental investigations by Alcoa Inc. (Alcoa) due to the presence of polychlorinated biphenyls (PCBs) in river sediments and fish. This 11-km long reach of river, which lies in the backwater of the St. Lawrence, is 120 to 180 m wide and 3 to 8 m deep. In 2001, a pilot test cap was constructed over a 230-m length of riverbed about 3 km below Massena to study the feasibility of subaqueous capping as a remedy for containing the PCB-containing sediments. A severe ice jam on March 28, 2003, caused hydraulic scour of the pilot cap and, in some locations, the underlying sediments. Because the phenomenon of ice-jam-related scour in the lower Grasse River was not known prior to this event, the pilot cap was not designed to withstand the associated forces. Subsequent analyses were performed to understand the magnitude of the 2003 event, as well as ways to mitigate the effects associated with ice jam scour events in the lower Grasse River. Initial results from these studies indicated that similar ice jam scour events could be mitigated through retention of the breakup ice run using a pier-type ice control structure (ICS). The effectiveness of a stand-alone pier-type ICS near the upstream end of the lower Grasse River designed for this purpose was evaluated through physical model tests at the U.S. Army Corps of Engineers (USACE) Cold Regions Research and Engineering Laboratory (CRREL) and numerical modeling simulations using DynaRICE (Alcoa 2007).

The Massena Electric Department (MED) has developed preliminary plans for a 7.9-m-high hydroelectric dam near the foot of the Massena Rapids. Subsequent discussions between Alcoa and MED led to the investigation of whether ice control could be incorporated into the project design. On this basis, Alcoa used DynaRICE to evaluate the ice retention capacity of the structure and the resulting ice jam profiles. This preliminary numerical analysis identified discharge, ice volume, and under-ice erosion velocity as important factors in ice retention capacity and upstream water levels. In view of these considerations, physical model tests of these specific aspects of the MED Dam were carried out in conjunction with the ongoing tests for the stand-alone pier-type ICS. The results of the study were also used to evaluate near-field backwater effects in the vicinity of the proposed dam structure.

2. Approach

Physical model tests investigated in detail ice transport processes in the immediate vicinity of the dam gates. These tests were carried out in a 2.3-m-wide × 36-m-long refrigerated flume located at the USACE CRREL in Hanover, New Hampshire (Figure 1). At an undistorted 1:20 scale, the experiments considered a 48-m-wide segment of the entire 134-m-wide channel width at the proposed dam site. Froude number similitude was used to calculate scaling relationships for length, velocity, and discharge. Figure 2 shows the segment of the prototype river cross section modeled, and Figure 3 shows a longitudinal profile of the flume with an ice accumulation retained upstream of the dam. At 1:20 scale, the 36-m-long flume represented a prototype channel length of 730 m, 570 m of which was upstream of the dam and 160 m downstream.

\[134 \text{ m is the approximate hydraulic width at the proposed dam site. The hydraulic width is the cross-sectional area divided by the average depth, assuming a water surface elevation of 54.25 m North American Vertical Datum (NAVD) 1988.}\]
The tests used freshwater ice containing 1% by weight urea to scale for ice strength. Ice strength scales with the prototype-to-model length ratio of 1:20. Ice flexural strength and ice piece size distribution were measured at the start of each test. Model ice floe dimensions were compared to field and lab-observed natural ice piece size distributions. Model ice floe thickness ranged from 20 to 34 prototype cm and ice strength ranged from 280 to 1380 KPa.

Previous sensitivity analyses determined a range of breakup discharges and ice volumes associated with a 100-year recurrence interval ice jam event on the lower Grasse River (Alcoa 2006). DynaRICE simulations (Kolerski and Shen 2008) demonstrated ice retention at the dam for Cases A through F listed in Table 1. Total ice volumes reaching the MED impoundment were scaled according to the ratio of the calculated pre-breakup ice thickness and the 2003 event ice thickness of 59 cm, with the volume reduced 20% from the 2003 event ice volume due to the further upstream location of the proposed dam. Based on calculation of pre-breakup ice volumes, this portion was assumed to be 80 percent of the 2003 event ice jam volume of 304,000 m$^3$, or 238,000 m$^3$. In the simulations, it was assumed that the dam would retain a stable ice accumulation provided an under-ice erosion (water) velocity of 1.5 meters per second (m/s) and Froude number of 0.09 were not exceeded. The dam was numerically modeled as a 3.4-m-deep barrier with underflow across its entire width. If the ice accumulation at the barrier thickened beyond the 3.4-m depth, that ice was assumed to pass downstream. Upstream of the barrier, the model allowed the ice accumulation to thicken beyond the 3.4-m depth, as long as the ice erosion velocity and Froude number for ice entrainment were not exceeded. Table 1 lists maximum ice accumulation thicknesses and under-ice water velocities from the DynaRICE simulations. These maximum ice thicknesses typically occurred a short distance upstream of the dam since the submerged depth of the accumulation at the dam was limited to 3.4 m.

The MED Dam physical model study focused on mid-range ice and discharge conditions represented by Cases C through E of the numerical simulations. Here, DynaRICE projected maximum ice accumulation thicknesses near the dam of 6.3 to 4.1 m, with a corresponding decrease in under-ice water velocity of 1.3 to 0.6 m/s, respectively. These mid-range cases were considered the most critical since they produced the highest under-ice water velocities (1.34 m/s for Case C) and contained sufficiently high ice volumes that a major ice release at the dam would have the potential to produce an ice jam on the lower river.

Initial physical model tests used a constant discharge $Q$ of 259 cubic meters per second (cms), which equals the breakup discharge for Case C, and also the modeled peak discharge for Case E in the sensitivity and MED Dam numerical simulations. In the flume model, once ice retention was demonstrated at 259 cms, discharge was increased to 351 cms. This flow was considered a reasonable upper limit since it exceeds all the breakup discharges listed in Table 1 and equals the modeled peak discharge for Case C of the numerical simulations of mid-range ice and discharge conditions. The range of flows modeled are conservative since Case E, representing the lower end of the discharge range, is similar to the flow conditions of the March 2003 ice jam, which is considered the design event for ice jam mitigation alternatives under consideration for the lower Grasse River.

Because of the limited domain and width of the flume model, it was not possible to model the entire length and ice volume of full-scale ice jams at the MED Dam. The study therefore focused
on ice processes in the near vicinity of the dam and relied on DynaRICE for the simulation of far field ice and hydraulic processes. As a result of the limited flume channel length, the smaller ice volume, and the reduced flume width, the scaled thickness of modeled jams forming naturally was about half the ice thickness one would expect in the prototype channel. For these reasons, the ice accumulations near the model dam were manually thickened based on target values of 4.1 and 6.3 m for the 259- and 351-cms discharges, respectively (Table 1).

Three gate geometries were tested, ranging from a conservative underflow gate design similar to the barrier used in the DynaRICE simulations, and progressing to the actual rectangular gate design proposed by MED. It was thought that locating the gate openings as low as possible along the dam would minimize the potential for ice passage. For this reason, the 3.0-m by 4.9-m rectangular gates proposed by MED were initially tested at an invert elevation of 48.46 m (NAVD 1988), and then raised to the recommended invert elevation of 49.38 m (NAVD 1988).

The continuous underflow gate extended across the entire flume width. Calculated gate opening heights for the various discharges were fine-tuned by trial and error to maintain a constant water depth of 6.4 prototype m on the upstream side of the dam. For the 259-cms discharge, a gate opening of 0.24 m prototype was used, and for the 351-cms prototype, the gate opening was increased to 0.30 m.

The preliminary design provided by MED consists of eight 3.0-m-wide by 4.9-m-high vertical lift underflow gates with the gate sills at 49.38 m (NAVD 1988) (Figure 3). These gates would occupy the central portion of the dam with 6.1-m spaces in between. The 34% channel width modeled in the flume was best represented using three of the eight gates proposed for prototype conditions. These three gates were spaced symmetrically with respect to the flume channel centerline. Based on calculations and fine tuning in the model, to pass the equivalent prototype discharge of 259 cms and maintain a 6.4-m upstream pool depth required 1.31-m gate opening. For the 351-cms prototype flow, the gate opening was increased to 1.84 m. The initial four flume tests used a more conservative sill height of 48.46 m (NAVD 1988) in an effort to minimize the potential for ice passage. A final test was conducted with the recommended sill height of 49.38 m, which increased the gate openings slightly to 1.43 and 1.95 m for the 259- and 351-cms full river flows, respectively.

During the physical model tests, water velocity was measured at selected locations beneath the ice accumulation using a miniature Marsh-McBirney electromagnetic velocity probe. An array of five pressure transducers recorded water levels at locations 12, 90, 183, 305, and 427 prototype m upstream of the dam. Ice thickness was measured visually through transparent sections of flume wall, or from the top of the accumulation using a hooked ruler. Still and video photography also documented test results.

A major difference between the flume and the DynaRICE model is that the numerical model simulated ice, hydraulic, and geometric conditions of the full river width rather than just the 34% portion of the width modeled in the flume. Also, the flume was flat-bedded and the domain limited to 730 m in length.
It was anticipated that the reduced width of the flume might result in a “narrow river” type ice jam of limited thickness as opposed to a “wide river” type jam, which would be expected on the full-width Grasse River. Pariset and Hausser (1961) defined jams on “narrow rivers” as ice accumulations whose thickness during upstream progression is defined by the entrainment and deposition behavior of individual floes under the associated hydraulic forces at the head of the accumulation, and jams on “wide rivers” as those where the thickness results from internal collapse (or “shoving”) of the accumulation as a consequence the streamwise forces on the accumulation. The section of the Grasse River upstream of the proposed MED dam is sufficiently wide that one would expect shove thickening as a result of water drag, channel slope, and gravity. By this process, the ice jam, in its mid-section, would attain an “equilibrium thickness” sufficient to transfer downstream forces through the ice material to the shorelines where they are resisted by bank friction. DynaRICE inherently includes the mechanics associated with a “wide river” jam.

Flume ice jam thicknesses and water surface profiles were compared to similar DynaRICE runs. It was expected that, because of the flume’s limited ice supply, reduced width, and flat bed, the flume ice jam profiles would be similar to the computer-simulated profiles only in the ice jam toe area, defined as two or less river widths upstream of the dam. Beyond this distance, one would expect lower water surface elevations in the flume compared to the numerical model. Tests 1 and 2 used the continuous underflow gate condition, whereas the remainder used the three rectangular gates. Table 2 summarizes test starting conditions and results.

3. Results

The model dam reliably retained ice for the full range of test conditions. Parameters used to evaluate performance included maximum ice accumulation thickness, maximum under-ice water velocity, maximum ice accumulation length, and maximum upstream water level rise (Table 2).

Continuous Underflow Gates

In initial testing, ice was allowed to drift into the flume and accumulate upstream of the dam as it would in a natural river. Because of the reduced width of the flume (34% compared to the actual river), the flume’s limited length, and limited ice supply, Test 1 produced flume ice thicknesses in the 1.2- to 1.8-m range, less than half of that predicted by DynaRICE (Kolerski and Shen 2008). These differences between the physical and numerical model ice accumulation thicknesses are explained in part by ice jam theory, in particular the effects of channel width and the differences between “narrow” and “wide” jams, as previously discussed.

In the initial flume tests, some ice shoving was observed. Calculations of equilibrium ice jam thickness for the reduced channel width help explain the thinner ice accumulations observed in the initial flume tests. White (1999) provides equations used to calculate equilibrium ice thickness for the full river width of 134 m and the 46-m-wide channel modeled in the flume. For full-width hydraulic conditions (similar to DynaRICE Case E), the calculated equilibrium ice jam thickness of 4.8 m was very close to the mid-section thickness of the DynaRICE ice jam. Holding all other parameters the same but reducing the width to 46 m decreased the calculated equilibrium ice jam thickness to about 1.8 m. In addition to the “narrow river” phenomenon, the
thinner flume ice accumulations may occur because the jams are too short to shove to their full equilibrium thickness. To account for the width difference and the other factors discussed above, the flume ice was manually thickened in subsequent tests to attain values close to those predicted by the DynaRICE model (Tables 1 and 2).

The results of Tests 1 and 2 showed the model dam with a continuous underflow gate to be an effective ice retention structure. Test 2 provided an extreme case with a flow of 351 cms and a partially grounded ice accumulation (manually thickened to 6.1 m) upstream of the dam (Figure 4). Under these conditions, the volume of ice that passed beneath the gate was negligible. Test 2, with a relatively high starting ice volume of 23,000 m$^3$, caused a maximum upstream water level rise of 0.51 prototype m. The nearly grounded ice upstream of the gate resulted in the highest measured water velocity of the study at 2.0-m/s prototype at a location about 15 m upstream of the gate.

Three Rectangular Gates

In Tests 3 through 7, which evaluated the ice retention characteristics of a dam with three rectangular gates, no appreciable ice passage was observed. Recall that in Test 7 the gate inverts were raised 0.91 prototype m from 48.46 to 49.38 m (NAVD 1988). In the rectangular gate tests, under-ice water velocities were on the order of 1.5 m/s for the 351-cms discharge, with a maximum of 1.7 m/s measured during Test 3. In Tests 5 and 7, the ice was manually thickened to below the top of the 1.85-m-high gate opening. In these thick ice cases, the water would erode the ice underside leading to the gate. These eroded channels were limited in area, about the same width as the gate (3.0 m) with top elevations about even with the gate opening. The eroded channel extended only about 15 prototype m upstream of the dam, beyond which the level of the ice bottom could be well below the elevation of the gate opening.

As the testing evolved, more detailed water velocity and ice thickness measurements were made, particularly during Tests 6 and 7. Figure 3 shows a longitudinal profile for Test 6 at the time of maximum water surface elevations. Test 6, with a high starting ice volume of 30,000 m$^3$ and an accumulation thickness of 4.9 m, produced the greatest water level rise of all tests of 0.60 prototype m. Test 3, with the same starting ice volume but a jam thickness of 4.0 m, produced only 0.27 m of water level rise at the 259-cms discharge.

Figure 5 compares the results of Tests 6 and 7. Test 6 has the gate inverts at 48.46 m with a discharge of 259 cms, while in Test 7 the invert is 49.38 m and the discharge is 351 cms. Ice accumulation thickness and upstream water levels are comparable in both tests. Under-ice water velocity is slightly higher in Test 7 as a result of the higher water discharge. Raising the inverts of the rectangular gates from 48.46 to 49.38 m caused no loss in ice retention performance. The results of Test 7 were very similar to Tests 3 through 6 in that no appreciable ice passed the gates, even when the bottom of the upstream ice accumulation was lower than the top of the gate openings.
Comparisons of Physical Model Water Surface Profiles to DynaRICE Simulations

Tests 6 at the 259-cms discharge, with a 4.9-m-thick ice accumulation, is roughly comparable to the Case E DynaRICE simulation, with a peak discharge of 259 cms and an average ice accumulation thickness of 4.1 m near the dam (Kolarski and Shen 2008). The DynaRICE Case E water surface and ice jam profiles plotted in Figure 6 are similar to the Test 6 physical model results for the first 300 m above the dam, at which point the flume water levels fall below the computed values. The difference results in part from the limited ice supply, narrower width, and flat bed of the flume compared to the DynaRICE model, which uses the prototype ice supply and the actual river geometry, as previously discussed.

4. Conclusions

The flume model tests showed the proposed MED dam to be an effective ice retention structure for a conservative range of test conditions. Ice was retained at a full-scale river discharge of 351 cms, which has a probability of occurrence of less than 1% during the breakup period (Alcoa 2006).

The dam retained near-grounded ice accumulations up to 6.1 m in thickness with measured under-ice water velocities as high as 2.0 m/s. In cases where the bottom of the upstream ice accumulation was lower than the top of the gate opening, limited erosion would enlarge the under-ice flow paths leading to the gates while the surrounding and upstream ice mass remained stable. Once a short distance upstream of the gates, the under-ice water velocities were less than the commonly used 1.5-m/s ice erosion threshold value. The likely reason that the ice accumulation near the gates withstood velocities higher than 1.5 m/s is the added resistance to ice movement provided by the dam. Also, ice erosion velocity is better expressed as a range rather than a single value since many factors are at play. These include the ice piece size distribution, and ice jam porosity, turbulence, and the added resistance to ice movement provided by the bed, banks, or river structures.

Horizontal and vertical velocity distributions were fairly uniform, as was ice accumulation thickness and under-ice flow depth upstream of the gates. In other words, spatial variations in ice thickness were not sufficient to cause areas of concentrated high velocity flow.

Flume and DynaRICE-simulated ice and water surface profiles were fairly similar for the 259-cms flow, within the 4.0- to 4.9-m ice thickness range, for the first 300 prototype m above the dam. Because of factors of limited ice supply and flume model geometry (reduced width and flat bed), flume water levels were much lower than the comparable numerical simulations when the flow was increased to 351 cms.

The flume model results provide confidence in the ice retention capability of the proposed MED dam, answering questions about ice erosion and ice accumulation stability in the near vicinity of the dam gates. The physical model underpredicted upstream water level rise at significant distances upstream of the dam because of limitations in ice supply, reduced channel width, zero bed slope, and the limited model domain.
References


Table 1. Breakup Discharge and Ice Thickness from Sensitivity Analysis and DynaRICE-Predicted Ice Jam Thickness and Under-Ice Water Velocity.

<table>
<thead>
<tr>
<th>Case</th>
<th>Breakup Discharge $Q_{b,k}$ (cms)</th>
<th>Modeled Peak Discharge (cms)</th>
<th>Pre-Breakup Ice Cover Thickness $h_i$ (cm)</th>
<th>Pre-Breakup Ice Volume Upstream of MED Dam (m$^3$)</th>
<th>DynaRICE MED Dam Maximum Ice Accumulation Thickness (m)</th>
<th>DynaRICE MED Dam Maximum Under-Ice Water Velocity (m/s)</th>
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<tr>
<td>2003</td>
<td>178</td>
<td>242</td>
<td>59</td>
<td>2.38E+05</td>
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<td>Q-Ice-A</td>
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<td>10</td>
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<td>25</td>
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<td>1.11</td>
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<td>Q-Ice-C</td>
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<td>351</td>
<td>41</td>
<td>1.64E+05</td>
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<td>Q-Ice-D</td>
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### Table 2. Test Conditions and Results.

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<tr>
<th>Test</th>
<th>Gate Type</th>
<th>Discharge (m³ prot)</th>
<th>Gate Opening Height (m prot)</th>
<th>Ice Floe Thickness (cm prot)</th>
<th>Ice Strength (KPa prot)</th>
<th>Starting Ice Volume (m³ prot)</th>
<th>Maximum Ice Accum. Thickness (m prot)</th>
<th>Maximum Under-Ice Water Vel. (m/s prot.)</th>
<th>Max. Accum. Length (m prot)</th>
<th>Max. Water Level Rise (m prot)</th>
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<tbody>
<tr>
<td>1</td>
<td>Continuous Sill @ 48.46 m</td>
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<td>320</td>
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<td>780</td>
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<td>0.64</td>
<td>560</td>
<td>0.38</td>
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<tr>
<td>3</td>
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<td>259</td>
<td>0.24</td>
<td></td>
<td>780</td>
<td>2.26E+04</td>
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<td>1100</td>
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<td>7</td>
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<td>1.31</td>
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<td>4.27</td>
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<td>200</td>
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Figure 1. MED Dam model in CRREL refrigerated research area.
Figure 2. Section of proposed MED Dam showing cross section modeled in CRREL Flume.

Figure 3. Test 6. Ice jam profile.
Figure 4. Test 2. Continuous underflow gate passing 351-cms prot. with nearly grounded ice upstream of gate.

Figure 5. Ice jam profiles, gate openings, and under-ice water velocities, Tests 6 and 7.
Figure 6. Test 6 ice jam profile compared to DynaRICE Case E simulation.
Special Session: Remote sensing
Satellite Detection and Monitoring of Sea Ice Rubble Fields

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Abstract
RADARSAT-1 ScanSAR, Fine and Extended High images and QuickBird satellite imagery were visually compared to evaluate their respective ease of use in tracking the initial formation, development and decay of rubble fields and sea ice ridges, in the Canadian Beaufort Sea. Over a period of eight months, 35 RADARSAT Fine and Extended High images and 11 QuickBird images were collected. Available RADARSAT ScanSAR images that roughly coincided with the other modes’ collection dates were obtained through the Canadian Ice Service. The Extended High mode was the most successful at accurately identifying a known target and the most useful for visually monitoring target development. Fine mode images could provide a greater amount of detail, but visual identification of new features could be difficult. While the ScanSAR mode was not useful in detecting unknown features during the freeze-up and winter months, it was able to adequately detect decaying rubble fields after the landfast ice had broken up and open water conditions existed. The QuickBird images, at 0.6 m resolution, provided astounding levels of detail of sea ice features; however, the satellite’s limitations in terms of amount of available light and cloud cover limits its use as a reliable tool for tracking the development of sea ice features on a routine basis. The use of satellite imagery to detect and track sea ice features can be a useful tool for hazard identification, shipping, and identifying sea ice features of interest.
1. Introduction

Year-round vessel traffic in the Canadian Arctic, generally for the development of oil and gas reserves, is becoming increasingly viable. Chan et al. (2005) and Kubat et al. (2007) discuss that year-round shipping of Liquefied Natural Gas (LNG) from the High Arctic is feasible in the near-future with an Arctic Class 7 vessel or Canadian Arctic Class 2 vessel, or higher. Should offshore oil and gas production platforms become operational in the Canadian Beaufort Sea, this will entail supply vessel access to sites with personnel, as well as potential emergency rescue vessels and icebreaking support access. With this potential increase in vessel traffic and offshore development in this region of Canada, there will be a need for increasingly detailed sea ice information for operators and regulators.

In the 1980’s, when numerous oil and gas exploration platforms were operating in the Canadian Beaufort Sea, aerial imagery provided valuable information for those operators, identifying possible hazards to navigation (during the freeze-up and break-up periods) as well as potential ice features that could be hazardous to the platform. Today, satellite data has the potential to provide this same information. It can also be used in scenarios where large-scale reconnaissance may be needed, such as search and rescue, where selecting the optimal route for minimizing time to rescue could be important, and for the routine monitoring of sea ice features that change with time, such as the landfast ice edge and rubble fields.

Satellite-based synthetic aperture radar (SAR) has been used on the east coast of Canada for detecting point targets such as icebergs, and is regularly used during the Arctic shipping season to provide forecasts of sea ice conditions and to identify the presence of multiyear ice. However, it has not been extensively used for routine examination of Arctic sea ice features such as rubble fields and ridges. This is partly related to the relatively small amount of activity currently in this region, which limits the need for regular monitoring. However, should greater monitoring become necessary, greater confidence in the detection of sea ice features of interest will be required, at levels of detail not presently examined on a routine basis.

Optical sensors such as the commercial QuickBird satellite (owned and operated by DigitalGlobe Inc.) can also provide valuable information, at much higher resolution levels than currently possible with SAR systems such as RADARSAT-1. By knowing certain characteristics of an optical satellite during the period of image acquisition, it is possible to calculate information on sea ice elevations, based upon shadow length calculations. Melt ponds and developing cracks in the spring may also be identified. However, the use of optical sensors in the Canadian Arctic is limited, due to extensive periods of darkness and cloud/fog cover.

2. Satellite Detection of Rubble Features

Over a period of eight months, the Canadian Hydraulics Centre (CHC) collected 35 RADARSAT Fine and Extended High (EH) images and 11 QuickBird images for the purpose of monitoring rubble field development in the Canadian Arctic, and for ground-truthing other features of interest. Ground-truthing satellite data is an important activity, as misinterpretation of RADAR data especially can have severe consequences for shipping routes and loading on offshore structures.
The use of SAR imagery for the examination of sea ice has been documented in a number of useful texts, including those by Rees (2006), Hall (1998), Haykin et al. (1994), Hallikainen and Winebrenner (1992) and Larson et al. (1981). These texts document considerations for examining sea ice using SAR, which will not be described in detail here, such as:

- **Target scattering properties** – The presence of water or snow on top of the ice surface, and impurities within the ice will all affect how the RADAR signal is scattered upon reaching the surface.

- **Sensor characteristics** – The frequency/wavelength, polarization, incidence angle, number of looks and the repeat cycle of the RADAR sensor will all influence the backscatter return of the signal. For example, a larger incidence angle has the potential to detect rough surfaces such as ridges more readily than smaller angles, where the backscatter signal is closer to vertical.

- **Viewing angle** – The angle of the satellite with respect to the object of interest will also affect the backscatter return. For example, ridges running perpendicular to incident microwaves are more visible than those that are parallel. The viewing angle may also have effects such as shadowing, foreshortening and layover.

The human visual system is sensitive to a wide range of SAR image texture and is adept at resolving the signature ambiguities often found in SAR sea ice imagery. As such, the use of SAR imagery is still predominantly based on visual interpretation. In this analysis, Radarsat-1 data are visually analyzed and compared, and relative differences in tone and texture of sea ice features are discussed. To preserve relative differences between images, they were collected, as much as possible, given conflicting orders and/or location of the features of interest, with the same viewing geometry. For example, an effort was made to collect most of the images in the satellite’s descending mode, with a large incidence angle (for whichever mode was being collected).

For examining sea ice, there are two primary types of studies that can be done: searching for an unknown sea ice target (for example, attempting to find a rubble field or a ridge embedded within landfast ice) or tracking a known sea ice target (for example, monitoring a rubble field that forms with some degree of regularity at the same location every year or tracking the progression of the landfast ice edge). These two studies have different needs when it comes to using satellite imagery. For the latter, finer resolution images are valuable for studying details such as timing of formation and decay, size analysis, and so on. For the former, when a target’s location is not known, it becomes necessary to examine a large swath of sea ice in order to pinpoint a feature of interest. However, using a coarse resolution beam mode, which may cover a substantial aerial region and have a resolution of 100 m, for example, may not provide the level of detail required for sea ice features such as rubble fields and ridges, depending on the diameter of the former and the length of the latter.

An important distinction also needs to be made with respect to analysis goals of satellite images. Operational use of such images by organizations such as the Canadian Ice Service (CIS) requires routine collection of data, as well as a quick turnaround time for end-users. The same may also be said of industrial users, such as oil and gas platform operators, who may require rapid interpretation of data in order to examine sea ice features that may be a hazard for a structure. However, industrial applications are also such that more highly detailed information may be
required than what can be provided by typical operational data. For example, the characterization of grounded rubble fields and the formation of sea ice ridges are important in this context. Ground-truthing of such data is extremely important, in order that satellite imagery of non-routine features may be interpreted correctly, and to examine whether microwave returns from other features could completely mask features of interest.

From a scientific research point of view, it would be extremely useful to determine if satellite image modes routinely collected by the CIS (i.e. ScanSAR mode), the priority user of RADARSAT-1 data, are capable of detecting and monitoring smaller-scale sea ice features such as rubble fields and ridges. If proven, then the CIS operational RADARSAT data could become a regular, reliable source of data for studying such features at lower cost.

3. Satellite Image Analysis

The focus of this paper is the detection and tracking of rubble fields in the Canadian Arctic offshore with satellite image data. A rubble field is a pile-up of broken ice that forms due to either discontinuities on the sea bed (such as an increase in seabed elevation), that induce deep ice keels to scour and ground on the sea floor, or due to ice impinging on an immobile structure, such as an offshore drilling platform. Rubble fields may be floating or grounded, and their formation may vary from location-to-location and year-to-year. There are numerous remnant berm sites in the Beaufort Sea north of the Mackenzie Delta region, many of which continue to generate rubble fields, despite having been abandoned for over twenty years. For example, during an April 2007 field expedition, the authors observed seven rubble fields at such sites, ranging from 100 m wide to almost a kilometre. The locations examined here were the site of exploration drilling platforms, the first in 1985-1986, at Minuk I-53, which was located at 69.70964°N, 136.45886°W (NAD27 co-ordinates). A dredged, submarine berm remains at the site, rising from the surrounding seabed depth of approximately 15 m to 4 m below waterline. This berm often triggers the formation of a rubble field. The second location is the Tarsiut N-44 site at 69.896139°N, 136.19347°W (NAD27 co-ordinates), where a caisson structure and research station was located on a smaller submarine berm from 1981-1983. All images of these sites in this paper are from 2006-2007.

3.1 Freeze-Up

For scientific purposes and vessel traffic operations (rubble fields are a marine hazard), it is useful to know when a grounded pile-up of ice has formed during the freeze-up period. However, Johnston et al. (2000) have shown through ground-truthing that sea state and other ice features can greatly limit the detection of individual targets such as ridges and smaller ice rubble features. When searching for a feature with an unknown location, this task could be extremely difficult at this time of year, if not futile, no matter which SAR beam mode is used. The use of various algorithms in order to identify features that do not change over a certain period of time could make this type of task less difficult. Using optical satellite imagery could aid in pinpointing a feature, but with great time and expense.

It is far more straightforward to characterize a rubble field at a specific geographic location such as at a remnant submarine berm or around an offshore structure. It is, however, challenging to compare the advantages or disadvantages of different beam modes for images taken during the freeze-up period in the Arctic, as the pack ice dynamics may change so rapidly, day-to-day.
Again, analysis of stationary features could be of use in order to identify grounded ice features. As shown in Figure 1, only the Extended High beam mode image (Fig 1b) identifies the Tarsiut N-44 rubble field located at the centre of the image. This image shows a bright return at the centre, the rubble field, with a large, level ice flow (dark return signal) to the west of the rubble field fracturing due to interaction with the grounded rubble. Neither the ScanSAR image nor the Fine beam mode image readily identified the rubble field.

Figure 1. Comparison of a) Fine resolution beam mode (December 17, 2006), b) Extended High beam mode (December 16, 2006) and c) ScanSAR beam mode (December 15, 2006) images, for the same sea ice target location, the Tarsiut N-44 rubble field.
3.2 Landfast Ice

Once a sea ice feature such as a rubble field becomes landfast, it can become even more challenging to locate due to the roughness of the surrounding sea ice. Visual analysis of the backscatter seems to indicate numerous rough features, so it is difficult to pick out a feature of interest for an unknown location.

If the location of the feature is known, analysis with the finer resolution SAR modes and the QuickBird optical imagery is straightforward. Table 1 compares the details of the images used in this study. Figures 2 through 5 show images of the Minuk I-53 rubble field taken between March 10 and March 15, 2007, while the field was at the edge of the landfast ice. These images show that the ice from the northeast through the south of the field was landfast, while open water and moving pack were present at the remaining sides.

Figure 2 shows the RADARSAT-1 ScanSAR image from March 10, 2007. In this image, the rubble field and some nearby ridges are visible, but without knowing what these features were prior to studying the image, it is unlikely that the rubble field would be identified correctly. In the RADARSAT-1 Fine mode image (Figure 3), the return signal is not very strong; however the field is well delineated, at a fine resolution. In the RADARSAT-1 Extended High image (Figure 4), the rubble field has a very bright return, indicative of the roughness of the field. In the QuickBird image (Figure 5) the rubble field is very sharply defined. The relief of the field is unmistakable and shadows created by high ridges are evident. While these higher resolution images (i.e. RADARSAT fine and QuickBird) provide better detail, as indicated in Table 1, these modes have very limited swath width and thus coverage compared to ScanSAR.

The three higher resolution images show some interesting features within the field, such as an almost circular area of low backscatter return (or reflectivity in the case of the QuickBird image) at the upper NE side of the field. This area was ground-truthed and was shown to be a region of very level, relatively thin (0.4 m) ice, most likely a floe imbedded in the field upon its formation. The ridge features that generated large shadows at the bottom of the QuickBird image are also evident in the Fine and Extended High RADARSAT-1 images, although visually their return signal is no greater than that of much of the rubble field. However, these ridges turned out to be 12 m high, near-vertical shear walls. This again indicates the importance of ground-truthing RADAR imagery, in order to ensure that extreme features such as these are properly identified.

Table 1. Image specifications for March 10, 14 and 15, 2007 acquisitions.

<table>
<thead>
<tr>
<th>Satellite</th>
<th>RADARSAT-1</th>
<th>RADARSAT-1</th>
<th>RADARSAT-1</th>
<th>QuickBird</th>
</tr>
</thead>
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<tr>
<td>Beam Mode</td>
<td>ScanSAR Wide</td>
<td>Extended High 4</td>
<td>Fine Near 4</td>
<td>-</td>
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<tr>
<td>Incidence Angle(°)</td>
<td>31-39</td>
<td>54-57</td>
<td>43-45</td>
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<td>0.7</td>
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<tr>
<td>Swath Width (km)</td>
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<td>16.5</td>
</tr>
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<td>Look Direction</td>
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<td>Descending</td>
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<td>March 14, 2007</td>
<td>March 14, 2007</td>
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<td>Acquisition Time (UTC)</td>
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<td>13:01:42</td>
<td>13:31:01</td>
<td>20:53:21</td>
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</table>
Figure 2. RADARSAT-1 ScanSAR image from March 10, 2007 showing the Minuk I-53 rubble field.

Figure 3. RADARSAT-1 Fine mode image from March 14, 2007 showing the Minuk I-53 rubble field.
Figure 4. RADARSAT-1 Extended High beam mode image from March 15, 2007, showing the Minuk I-53 rubble field.

Figure 5. QuickBird optical satellite image from March 14, 2007 showing the Minuk I-53 rubble field, with landfast ice in the southeast quadrant of the image and open water and pack ice in the remaining areas.
3.3 Break-Up and Open Water

Larger grounded rubble fields can remain in place well past the break-up of the landfast ice, into the open water season. For example, tracking of the Minuk I-53 rubble field with satellite images indicated that the field did not break-up and disappear until July 14, 2007. Given its formation in mid-December, this feature was present for seven months, with little change in configuration during that time frame. Large features such as these that exist during what may become a typical shipping season can obviously be a navigational hazard if not properly identified. It is equally useful to know how long such features may last, for scientific purposes, as rubble fields have proven in the past to be effective at reducing ice loading on offshore structures.

Figures 6 and 7 are a ScanSAR and a QuickBird image (respectively) of the Minuk I-53 rubble field, the former from July 9, 2007 and the latter from the following day, July 10, 2007. Similar to the freeze-up period, it can be difficult to compare images that are not taken on the same day, due to the moving pack ice. However, as the rubble field was well-established and in a state of decay at this point in time, it becomes easier to identify non-moving targets that are likely to be grounded ice features. It can be seen that there is good visual agreement between the coarse resolution ScanSAR image and the extremely fine resolution QuickBird image at this time of year.

4. Summary

This study has shown that during the freeze-up period or when landfast ice is present, it would be difficult to identify a rubble field using satellite technology without prior knowledge of where such a feature is likely to form. This is especially true with the coarse resolution ScanSAR mode, and, conversely, the Fine mode images that have a low incidence angle. With specific algorithms designed to search for certain patterns of backscatter return, it may be possible to identify rubble fields with less difficulty; however the roughness of the surrounding ice could still impede straightforward identification. This may be especially true for the identification of narrow targets such as ridges (which were not discussed here) where orientation of the ridge with respect to the microwave return could be important. Automated change detection analysis could also be valuable for monitoring changes in landfast ice and rubble field development over the winter.

Optical satellites are extremely useful for detailed, site-specific information. However, the trade-off in resolution is a small swath size (compared to SAR imagery), which would result in costly and time-consuming efforts to pinpoint a sea ice feature of unknown location. Optical sensors require clear sky conditions (no obscuring clouds) and sunlight to provide useful data. In the Canadian Arctic, this effectively eliminates use of such sensors from the beginning of November through the beginning of March (polar winter). These sensors are also hindered by the persistent cloud over the Arctic in summer months. For routine operational use, these images would not be practical unless industrial users such as oil and gas operators were willing to risk cloud cover obscuring areas of interest for periods when sunlight was sufficient. For one-time, less time sensitive applications, such as examining a grounded rubble field around a structure, or for scientific investigations, optical imagery is extremely useful.
**Figure 6.** RADARSAT-1 ScanSAR image from July 9, 2007 showing the Minuk I-53 rubble field surrounded by pack ice and open water.

**Figure 7.** QuickBird optical satellite image from July 10, 2007 showing the Minuk I-53 rubble field surrounded by pack ice and open water.
Table 2 presents a summary of recommendations, where the various satellite modes may be of the most benefit. It can be seen that there are advantages and disadvantages to each system, depending upon the user’s requirements. Overall, when it comes to visually identifying and monitoring sea ice features, the RADARSAT-1 Extended High beam mode images and the QuickBird optical images seemed to provide the best information in this respect.

Table 2. Recommendations for use of various beam modes, and trade-offs/benefits for each.
(With digital analysis of backscatter, such as pattern recognition for areas of ice that have not changed between images, some tasks may be less difficult than visual inspection would imply.)

<table>
<thead>
<tr>
<th>Satellite</th>
<th>Identification of Sea Ice Feature, Unknown Location, Freeze-Up</th>
<th>Identification of Sea Ice Feature, Unknown Location, Landfast or Pack Ice</th>
<th>Identification of Sea Ice Feature, Unknown Location, Open Water</th>
<th>Identification of Sea Ice Feature, Known Location, Freeze-Up</th>
<th>Identification of Sea Ice Feature, Known Location, Landfast or Pack Ice</th>
<th>Identification of Sea Ice Feature, Known Location, Open Water</th>
<th>Likelihood of Acquiring Image on Specific Date</th>
<th>Ease of Routine Monitoring (e.g. weekly)</th>
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<tbody>
<tr>
<td>Beam Mode</td>
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<td>Extended High</td>
<td>Fine</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RADARSAT-1</td>
<td>Difficult</td>
<td>Potentially difficult</td>
<td>Difficult</td>
<td>Time-consuming, but possible</td>
<td>Time-consuming, but possible</td>
<td>Time-consuming, but possible</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RADARSAT-1</td>
<td>Difficult</td>
<td>Potentially difficult</td>
<td>Difficult</td>
<td>Time-consuming, but possible</td>
<td>Time-consuming, but possible</td>
<td>Time-consuming, but possible</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RADARSAT-1</td>
<td>Potentially difficult</td>
<td>Potentially difficult</td>
<td>Potentially difficult</td>
<td>Time-consuming, but possible</td>
<td>Time-consuming, but possible</td>
<td>Time-consuming, but possible</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>QuickBird</td>
<td>Straightforward</td>
<td>Potentially difficult</td>
<td>Potentially difficult</td>
<td>Straightforward</td>
<td>Straightforward</td>
<td>Straightforward</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Likelihood of Acquiring Image on Specific Date</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Fair (cloud cover and darkness factors)</td>
<td></td>
<td></td>
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<tr>
<td>Ease of Routine Monitoring (e.g. weekly)</td>
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<td>Good</td>
<td>Good</td>
<td>Poor</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

In 2008, data from the recently launched RADARSAT-2 satellite will become available. RADARSAT-2 provides all the capabilities of its predecessor RADARSAT-1, but also adds new high resolution modes and channel polarization. The new Ultra-Fine mode, with its 3 m resolution, has the potential to improve mapping of rubble field detail. The recently added full polarimetric modes have the potential to provide a wealth of information regarding surface roughness and feature detail through the use of advanced polarimetric analysis.
Acknowledgements
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References


Accuracy Estimation of GPS Measurements on the Russian Drifting Stations
North Pole-33 and North Pole-35.

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Abstract
There are difficult to estimate realistically the distance and velocity errors of GPS measurements in the conditions of the drifting sea ice. The presented paper includes some developments for solution this problem. At the summer 2005 the detail GPS measurements in the high latitudes of Arctic Ocean are carried out. Two civil GPS receivers were deployed into the consolidated ice floe on the distance about 181 meters and fix a position every minute. The statistical analysis of the field data allows investigate 2D distributions of pseudo-distance vector between measured points as well as the standard error of drift speed and acceleration estimations. The Gaussian PDF and confidence ellipses of concentration for interpretation of the 2D pseudo-position cloud are used. The error scattergram includes separate strips of the pseudo position points along the longitude lines witch with the rounding of calculation are connected. We have found some empirical relations between standard error, sampling interval and size of averaging window. The potential sources of measured errors as well as optimization of measurements are discussed. The error analysis allows calculate the confidence estimations of kinematical and dynamical events for the measured ice floe. Probably in situ calibration of GPS measurement and error monitoring will be useful for the planning of experiments and observations in the high latitude conditions.
Introduction
The field measurements using satellite navigation systems find successful applications in the geophysics (Steblov et al., 2005), glaciology (Hvidberg et al., 2001) and natural researches. Applying to problems of the sea ice geophysics there are interesting to measure 3D coordinates, drift speed, rotation and acceleration of the ice floes as well as the interaction forces in the sea ice cover. Some of these measurements during the Arctic Buoys International Program were carried out (Rigor et al., 2003) however their efficiency was limited by the satellite traffic and power. For the manned drifting station these factors are not critical that allows to investigate the potential efficiency of the measuring system and to work near to the borders of capability. New methods of kinematical and dynamical measurements based on the satellite navigation are perspective for improvement and extension of traditional measurements based on the contact inertial equipment (Smirnov, 1996).

A principal problem of GPS applications is defining of the real measurement accuracy and selection of the useful information on the background of natural and technical noises. Such calibration is not compound when measurement point is stationary (Antonovitch, 2005) however on the drifting ice of Arctic Ocean the development of indirect calibration methods is required. One of developed methods uses two GPS receivers that are employed on the same ice floe. In this case the real distance between GPS receivers is not changed and variations of the calculated pseudo distance are connected with measurement errors of two GPS channels. The investigation of statistical and spectral properties of pseudo distance variations allows estimate the measurement errors of distance, drift speed and accelerations as well as optimize and testing of the GPS measurement channels. The independent estimations of the measurement error were obtained using regression analysis for data series of two relatively independent GPS channels. A method based on defining of the reference trajectory of the measured ice floe was used for the estimation of the GPS position error in the frequency range more than 0.25mHz for each channel separately. This method would be useful for individual calibration and comparison of the GPS receivers on the drifting ice floe. The error analysis allows calculate the confidence estimations of kinematical and dynamical events that are demonstrated at the final.

Field Measurements
Firstly in the practice of the Russian drifting stations the two-point GPS measurements with one minute fix interval are carried out on the ice floe of North Pole-33 with drift track is presented on the figure 1(d). The measurements were performed at the summer 2005 between May 28 and August 18 using two civil GPS receivers - “GARMIN-GPS-8” and “GARMIN-MAP-76” that are installed as shown in figure 1(a). Visual observations as well as GPS measurements indicate the distance between GPS points was constant during the field experiments. At the same time the thermal and dynamical ice deformations were insignificant in comparison with measurement errors. At July 12 the laboratory home with equipment was displaced from the ice pedestal formed by the surface ice melting. You can see this event on the pseudo distance record in figure 1(c). Before and after the movement the distance between GPS points was 181.29m and 188.23m respectively. Please note the mean values of the pseudo distance are explained statistically because 95% confidence error of these estimations is about 0.02m. A histogram in figure 1(b) shows that about 70% of the pseudo distance variance is concentrated in the frequency range with period less than 1 hour.
Figure 1. (a) - chart of GPS receiver installation on the measured ice floe, (b) – frequency histogram of the pseudo distance data series, (c) - segment of pseudo distance data series with change of distance by the reinstallation one of the GPS antennas, (d) - drift track of the ice floe North Pole-33, summer 2005.

The data of two GPS receivers are fixed by the field computer every minute, but at different moments of the time. This generates an asynchronous error in the relative estimations with magnitude was compared in average with error of measurements. Before the analysis the original GPS data were synchronized for elimination of the asynchronous errors. Using a standard function of the linear interpolation the original data series were transform to the integer minutes with accuracy about one second. The comparison of the ice floe drift track before and after interpolation shows satisfactory quality of the transformation algorithm. A little bit side effect of the synchronization is the smoothing of drift track because of weight averaging between measured points.

Data Analysis
The estimations of the distance and azimuth between measured points based on synchronized data series of the two GPS receivers were obtained. The calculations are performed using standard mathematical functions in approximation of the great circle (the shortest path between two points). An example of the calculated pseudo distance you can see in figure 1(c). The variance of these estimations is based on practically simultaneous GPS measurements on the distance witch length is about $10^{-5}$ of the route length of the navigation radio signals. Therefore in predominance this accumulates the noises of the radio signal reverberation in the near zone of the GPS antenna, cable as well as algorithmic noises and weakly uncorrelated noises on the route of the radio waves. For such conditions the detailed place selection of GPS antenna and field calibration of the GPS measurement channels is a main of factors for quality experimental data. Usually applying to the GPS measurements the following indexes of accuracy are used (Hofmann-Wellenhof et al., 2001): Circular Error Probable (CEP), Horizontal Root Mean Square (HRMS) and Twice Distance Root Mean Square (2DRMS). These indexes show a horizontal
distance to the true position in which borders 50%, 68% and 95% of points are located, respectively. At the time the spatial anisotropy of the measurement errors and symmetric shape of the measured point cloud are supposed. However, an elliptic approximation was used by the analysis because of anisotropy clouds are observed on the drifting ice in the high latitudes (see fig.2(a)). The grey ellipse based on the two-dimensional probability distribution includes a correlation of the orthogonal measurements. One has two principal semi-axes with size 6.2m and 4.0m respectively which characterize 2DRMS estimation along the main directions. The white ellipse is based on the two partial probability distributions in the orthogonal coordinates. One has two semi-axes with size 5.0m and 3.6m which characterize 2DRMS estimation for each of partial distributions that you can see in figure 2(b,c). Please note the first estimation in average is little bit more than the second that reflects a known effect of variance summation for independent random processes (Kramer G., 1975).

![Figure 2](image)

**Figure 2.** (a) – Daily scattergram of the GPS-2 points in the local Cartesian coordinates centred on the GPS-1 position, 2D and partial probability distributions are reflected respectively by the gray and white ellipses of 95% concentration, $\sigma_{xy}$ – partial/principal standard deviations, (b,c) – frequency histograms of the partial pseudo distance distributions with normal approximations and partial standard deviations.
An effect of periodic grouping of the scatter points along the latitude is presented in figure 2(a,b). Usually the two strip systems with spatial period about 1.9m are observed that is close to rounding errors of the geographical coordinates. The second generation of GPS receivers has a rounding error about one thousandth of a minute or 1.852m along of latitude. The length of one longitude minute in the area of the drifting station varied from 49m on the latitude 88.5º to 113m on the latitude 86.5º (see fig.1d) and corresponding rounding error along the longitude varied from 5 to 11cm that is significantly less than GPS measurement errors. Thereof the rounding errors are generated only by the latitude calculations. For the third generation of GPS receivers the rounding accuracy is increased on the one order that automatically eliminates the problem of the rounding errors as well as corresponding problems at the calculations of velocity and acceleration. The elliptical shape of the scatter cloud reflects a non-stationary condition on the analysis interval because of the ice floe rotation with speed about 2.5º per day that was indicated by the pseudo azimuth data. Thereof the average position of GPS-2 in the local coordinates has permanent displacement and summary scatter cloud was strained in the cross direction to the rotation radius. In such conditions the small semi-axis of the scatter cloud with size about 4m is defined as representative 2DRMS error of GPS measurement for the presented sample.

A spectral analysis of the pseudo distance fluctuations indicates that the main part of the measurement error variance is localized in the frequency range from 0.5 to 2 mHz. (fig.3d). The attempts to find corresponding harmonics in the spectrum of the drift speed for each of GPS receivers gave unexpected result - the power spectra of the drift speed by the GPS-1 and GPS-2 are very different – figure 3(e, f).

Figure 3. Data series - (a,b,c) and Power Spectral Density (d,e,f) of the pseudo distance – (a,d), of the drift speed for GPS-1 (b,e), of the drift speed for GPS-2 (c,f).
The spectrum of GPS-1 can be identified as spectrum of non-stationary random process with dominance of the low frequency harmonics, which reflects variance distribution of the natural processes in the ocean, atmosphere and ionosphere. An increase of power in the frequency range between 0.5 and 2 mHz of the GPS-2 spectrum occurred that probably connected with technical problems of the unstable receiving of the radio signals from navigation satellites. At the time the power of the pseudo distance spectrum that associated with the technical noises is about 50% of the summary power which has proportional influence on the scatter variance as well as on the estimation of the measurement accuracy. In particular this means that the obtained 2DRMS estimations can be better after removing of the technical noises.

For quantitative comparison of the different GPS measurement channels a method of the reference trajectory was developed. Comparing with onshore observations its feature is that we do not know the real position of the measured point and we have no a reference point for estimation of the measurements errors. However in the conditions of stationary drift we can determine a smooth trajectory of the ice floe and use it as reference for estimation of the position errors. For calculation of the reference trajectory the segments of records without of obvious perturbations were selected. These segments were processed by a low pass Butterworth filter of the 6-th order with border period about one hour. Further by spatial-temporal interpolation the reference position for every point of the original data series were defined. The GPS measurement error was defined as pseudo distance from the original point of the data series to calculated reference point. According to offered algorithm such estimation can be defined as estimation of absolute positioning errors in the limited frequency range (more than 0.3 mHz). The offered estimation includes both variance of technical noises in the antenna, cable, receiver, in configuration of the navigational satellites, and natural noises with period less than one hour. Because of dominance of the technical noises further this estimation is called as technical error.

As you can see from figure 1(b) the summary variance of the technical noises is about 70% of the total pseudo distance variance.

Figure 4(a) presents a frequency histogram of GPS-2 technical error estimations as well as Least Squares Estimations of technical error distributions for GPS-2 and GPS-1. Similar calculations were performed using partial distributions along the orthogonal geographical coordinates in the normal approximation and the obtained result is shown in figure 4(b) in the form of a two-dimensional elliptic diagram. The internal ellipse with principal semi-axes 1.66m on longitude and 2.28m on latitude corresponds to 95% technical error estimations for the GPS-1. The external ellipse with principal semi-axes 3.58m on longitude and 3.54m on latitude corresponds to 95% technical error estimations for the GPS-2. The comparison of the medians of the Gamma distributions as well as the two-dimensional diagrams of the technical errors shows that measurement channel of GPS-2 makes a great noise and after removing of the superfluous noise we can expect a proportional reduction of the 2DRMS error from 4m to 2.5…3m, and reduction of the technical noise variance from 75% to 40% of the total variance. The indirect comparison of the estimations obtained using the reference trajectory method with result of the inshore measurements gives a quality agreement by the two positions. In the first place both measurement distributions are Gamma distributions. In the second place the estimations of the Gamma distribution with median about 0.96m for technical error are agreeable with estimations of the total error obtained on the Germany Antarctic Station “Neumayer” with median of the Gamma distribution about 2.8m (personal message). We note that the meridian semi-axis of the internal ellipse of the GPS measurement errors in figure 4(b) is about 2 m which is closed to rounding errors along the latitude for the GPS receivers of the second generation.
Figure 4. (a) – Frequency histogram of GPS-2 technical error estimations with approximations by Gamma distribution for GPS-2 (median 1.69m) and GPS-1(median 0.96m), (b) – diagrams of the technical errors for GPS-2 (gray) and GPS-1 (white).

The low frequency component of the pseudo distance variance with period more than one hour is connected mainly with fluctuations of the natural geophysical fields and their influence on the GPS. Figure 5(a,b) presents ten-diurnal fragments of the pseudo distance including low frequency component obtained using Butterworth filter of the 6-th order. Here you can see both diurnal and semi-diurnal components, fluctuations with period from one hour to six hours and separate non-periodic peaks with deviation up to 5m. The investigation of the influence of the natural processes on the GPS measurement accuracy requires long data series as well as application of the statistical tools for analysis of the non-stationary random processes.

Figure 5(c,d) presents a standard error of the pseudo distance estimations depends of the smooth interval of the original data series witch total size is about 80 days. The smoothing of data was carried out using the low pass Butterworth filter of the 6-th order witch frequencies are varied in the wide range from 2 to 60 minutes – figure 5(c) and from 1 hour to 40 hours – figure 5(d). Practically all analyzed data series had a normal distribution which stable statistics. It allowed use a standard deviation of the normal distributions as an estimation of the measurement errors. A 95% confidence interval for the estimations of the standard deviation is about 1cm. The analysis of the dependence allowed selecting separate segments of diagram with homogeneous properties. In particular at the smooth interval less than 10 minutes the measurements error is about 1.6m and does not change because of strong noises correlation. At the smooth interval from 10 to 30 minutes a standard error decreases to 1.1m that is connected probably with the fluctuations of the satellite configuration. At the smooth interval from 30min to 28hrs taking place an exponential reduction of the standard error up to 0.3m with small deviation in the area of diurnal and semi-diurnal harmonics. At the smooth interval more than 28hrs a standard error has uniform reduction with rate about 2.5cm per day.
Figure 5. 10 days long segments of the pseudo distance series, (a) – 21..30 of June 2005, (b) – 4..13 of August 2005, gray and black lines – before and after of low pass filtering respectively, (c,d) - standard error estimations depends of the critical frequency of low pass filter, (e) - daily variations of the pseudo distance.

The presented diagrams allow to forecast the accuracy of the GPS measurements and to plan field experiments in the conditions of the slow kinematics as for example at the investigation of glacier ice. In particular for reaching a decimeter accuracy of the relative GPS measurements it is necessary to measure on the diurnal interval and for reaching centimeter accuracy - during of one week. Probably these estimations can be improved using calibrated GPS receivers of the third generation and significantly improved using the professional equipment. The presented analysis allows select a diurnal variance of the pseudo distance estimations that is shown in figure 5(e). The diagram indicates the difference between night and morning measurements of the pseudo distance is about 0.5m.

The analysis of the measurement errors presented above is based on synchronous measurements by two GPS receivers with a spatial shift. At the measurements of the drift speed two sequent measurements of one GPS receiver are used with a shift both in space as well as in the time. The speed measurement errors are generated in dominance by the measurement error of the distance between sequent points because the satellite navigational system has the highest time standards. However the time factor has indirect influence through the fluctuations of the system parameters.
Other way the GPS receiver can to measure a drift speed by the effect of Doppler however in the presented paper this method is not discussed because of small resolution.

Figure 6(a,b,c) presents examples of calculation of the drift speed and direction on the drifting station NP-35 when the measured floe was destructed at the fast return of the ice drift. Figure 6(g) demonstrates a statistical simulation of the drift speed measurements by the GPS, where each of sequent measurement points is modeled by random series with empirical Gamma distribution and the displacement between them equals double median of distribution. At the time the pseudo distance between pair of points has a Gamma distribution at the small displacements which transforms to the normal distribution at the increase of the displacement. A variance of the drift speed estimations decreases because of proportional increase of the measurement interval. The estimations of the drift speed in figure 6(a) correspond to model in the Fig. 6(g) in the scale 1:12 because at the mean drift speed about 5 cm/sec the displacement at the measurement interval (1 min) is about 3m which is approximately equal two medians of the empirical Gamma distribution in figure 4(a).

The estimations of drift speed and direction in figure 6(b,v) are obtained by the measurement interval 8 min that is correspond to a displacement about 16 medians of the empirical Gamma distribution. A comparison of the estimations shows 8 times decrease of the model error as well as signal fluctuations in the frequency band of 0.5–2 mHz which earlier were hided. The presented estimations of the drift speed also contain error of calculations rounding which appear in the discrete amplitudes.

![Figure 6](image)

**Figure 6.** An example of the drift speed (a,b) and direction (c) calculations November 28, 2007, North Pole–35, time interval of calculation (a) – 1min, (b,c) – 8min, (d) - a statistical simulation of the drift speed measurements using two Gamma distributions with median - 18m, variance - 144m² and displacement – 36m.
Figure 7. A distribution of accelerations along the ice floe drift track, (a) – fracture of the ice floe NP-35 on November 28, 2007, (b) – ice pressure, NP-33, on July 16, 2005, (c) – drift speed and acceleration of the measured ice floe, NP-33, on July 16, 2005.

A discrete step depends both of the measurement interval and of the drift direction and has maximum value for the drift speed in direction of north-south and for the azimuth in direction west-east. As discussed earlier, the error of the latitude rounding is 1.852 m and discrete steps of the drift speed are calculated by simple relations. For example a discrete step for the drift speed estimations by the sampling interval 1 min is 1.852 m/60 sec = 30.9 mm/sec and by the sampling interval 8 min - 1.852 m/60*8 sec = 3.85 mm/sec. The corresponding discrete step for the azimuth estimations mainly depends of the drift speed and is for presented conditions about 5 degrees. Please note that in the cross direction the discrete effect is significantly weaker however in any case there is an important element of the precision measurements.

For estimation of the non-correlated measurement errors of the drift speed and their dependence of sampling interval a regression analysis of the two GPS measurement channels at the distance about 181 m was performed. In this case the distance between sequent measurement points on the sampling interval is used as input data of the regression and a standard error of the regression is used as estimation for the drift measurement error.
The table below presents the estimations of the standard error of regression, non-correlated errors of the drift speed measurements and errors of the calculation rounding that are obtained both the different measurement intervals and the different drift speed and direction. These estimations show a significant decrease of the non-correlated noises occurs at the decrease of the measurement interval less than 10 min. Probable this relation is connected with an increase of the stability of the navigational satellite configuration with reduction of the measurement interval. The table indicates also the rounding errors are dominated at the increase of the measurement interval over 10 min.

**Table 1.** Sampling interval influence on the empirical error estimations.

<table>
<thead>
<tr>
<th>Sample interval, min</th>
<th>1</th>
<th>2</th>
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<td>1.49</td>
<td>1.62</td>
<td>1.72</td>
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<tr>
<td>Standard error of speed, cm/sec</td>
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<td>0.95</td>
<td>0.75</td>
<td>0.62</td>
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The detail analysis of the pseudo distance estimations allows obtaining the statistically significant estimations of the drift speed and ice floe accelerations as well as determining of the way to the estimations of inner forces witch acting in the ice cover. After low frequency filtering of the original data series and replacing of the fluctuations related to the variance of satellite configuration we could obtain the standard error of the acceleration measurements to 2.5mkm/sec^2 or 0.25mGal that allows investigate in first approximation the distribution of external forces along the ice floe drift track. Figure 7(a,b) presents a diagram of the acceleration distribution along the “NP-33” drift track during the ice pressure on July 16 as well as the acceleration distribution along the “NP-35” drift track during the floe fracture on November 28 2007. Figure 7(c) presents the acceleration and drift speed data series for the first event. As both diagrams shown, at the active dynamic events a chaotic distribution of internal ice forces in the ice cover are dominated and the power of the local accelerations significant exceeds the power of accelerations related to large scale influences of the wind and currents. Probably small scale and mezzo scale internal forces, their redistribution and self-organization in the discrete environment are responsible for evolution of the failure processes in the ice cover. Such ice cover dynamics is explained and typical for the discrete hierarchical environments (Sadovskiy 1989).

We hope the presented analysis and some results will be useful for the optimal planning of field experiments using satellite navigation systems as well as for the field investigations in the area of the sea ice geophysics.

**Acknowledgments**

We would like to thank the management and the members of Russian Drifting Station projects for supporting the measurements and field data collection. We also wish to thank Dr. Rudolf A. Balakin of Arctic and Antarctic Research Institute as well as Dr. Martin J. Doble of Cambridge University for useful discussions.
References
Operational Integration and Use of Satellite SAR-Derived Information in the CIS and International Ice Patrol Iceberg Programs

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Abstract:
The CIS (CIS), a division within the Meteorological Service, and the International Ice Patrol (IIP) of the US Coast Guard, take joint responsibility for monitoring icebergs off North America’s east coast. Iceberg information is provided primarily to promote safe navigation of the Northwest Atlantic Ocean during the period when danger of iceberg collision exists. The primary means by which the CIS and IIP acquire data is through airborne reconnaissance from fixed wing aircraft. Other data come from iceberg sightings from ship and shore observations of opportunity. Data from all sources are integrated manually at each center into the Berg Analysis and Prediction System (BAPS), a synchronized database of the current observed and/or model-drifted locations of all known icebergs.

Over the last decade, the CIS and IIP in conjunction with C-CORE, have been investigating the use of satellite synthetic aperture radar (SAR) imagery to detect icebergs. Based on hundreds of SAR images and field observations of icebergs (and vessels) collected over the last 10 years, C-CORE has developed (and continues to enhance and improve) the Iceberg Detection Software (IDS) which automatically detects potential iceberg targets in SAR imagery. Output from the IDS is quality assessed by expert analysts at C-CORE and standard MANICE (Manual of Ice Reporting) coded messages are generated and transmitted to the CIS and IIP. The SAR-derived target information is then reconciled against and integrated manually with the current BAPS database. Presently, this information is being used on a semi-operational basis, nominally twice per month. In addition, annual iceberg surveys are done in Baffin Bay and Davis Strait in the fall (October) using SAR imagery to obtain an estimate of the iceberg population expected to drift down the east coast the following year. The goal of this work is to complement and perhaps eventually replace the costly airborne surveillance programs with satellite SAR monitoring. However, there still remain several challenges and issues to be taken into consideration when using the SAR-derived iceberg information. In this paper, we discuss the pros and cons and describe the steps being taken to bring information from the SAR imagery and the IDS into an operational environment.
1. **Introduction:**
In the spring of 1995, the Canadian Ice Service (CIS) started to investigate point target detection from various sources of radar ranging from ground wave to airborne to space borne and comparing the output from multiple sources over a common area. At that time, the Department of National Defense (DND) was also investigating point target detection and potential classification using the European Space Agency ERS-1 satellite. In November 1995, the Canadian Space Agency (CSA) launched RADARSAT-1 and with this new source of radar information various projects were developed under the CSA’s Application Development Research Opportunity (ADRO) programs. In 1997, both the US Coast Guard International Ice Patrol (IIP) and C-CORE of St. John’s initiated separate projects under the ADRO focused on investigating the capabilities of iceberg detection and discrimination using synthetic aperture radar (SAR) data from RADARSAT-1. The ADRO-1 program was followed by ADRO-2. In 1999, the CIS initiated a complementary project under the Panel on Energy R&D (PERD) program funded by Natural Resources Canada. All of this inspired future work and jointly, the CIS, the IIP, and C-CORE have coordinated activities in a multi-year validation program which has received support from a variety of sources, including the above mentioned, as well as a consortium of oil and gas companies operating on the East Coast of Newfoundland, and the European Space Agency (ESA). Much of this work culminated in a 2004 report by C-CORE characterizing the lessons learned and capabilities of RADARSAT-1 to that date in time (C-CORE, 2004).

2. **The CIS iceberg program:**
The iceberg program was created in 1983 as part of the Expanded Ice Information Service Program but it only became operational in 1986 when the CIS took delivery of a sophisticated computer system and program designed to produce analysis and prognostic drift outputs (see figure 1). The Berg Analysis and Prediction System (BAPS) essentially models the drift of reported icebergs using environmental grids of water currents and surface wind files over the Canadian East Coast. The drift component of the model is coupled with a deterioration model which uses environmental data from the sea surface temperature as well as the wave height and period over the same domain. Twice daily, a new set of environmental wind and wave fields are ingested into the BAPS to run the drift and deterioration of all icebergs populating the database. The water current and the sea surface temperature fields are updated once daily as these fields have slightly more conservative data compared to the dynamic wind and wave fields. Every two to three weeks, an iceberg survey over a portion of the model area is scheduled (see figure 2). The result of the survey is ingested into the system and used to update an existing iceberg (known as a re-sight), to add an iceberg, or to delete an iceberg which was modeled but has not be seen during the survey. The database of iceberg sightings is synchronized between the IIP and the CIS offices every time a new sighting is added to the database by either agency. Sightings come from different sources ranging from dedicated surveys to reports of opportunity from fishing, commercial, Coast Guard and even military ships, and sometimes satellite data. From the modeled information, a daily iceberg bulletin is issued for mariners and a daily iceberg population chart (see figure 3) is created and distributed to ships at sea and our own Web page at: [http://www.ice.ec.gc.ca/](http://www.ice.ec.gc.ca/).
3. Using SAR Data for Iceberg Detection:
The CIS uses primarily RADARSAT ScanSAR Wide image mode for its sea and lake ice program maximizing the overall swath width to 500 kilometers (see figure 4). Moreover, the data are 2x2 block-averaged in order to reduce the file size to be manipulated on the individual workstations. Although one can definitely see icebergs and point targets in these images, the averaging and the 100m effective pixel size only allows for the largest targets to be seen when using an image captured specifically for the sea ice program. For iceberg detection, it was recognized that it was necessary to use full resolution data and to use image modes (e.g. ScanSAR Narrow, Wide) which have higher resolution, albeit with some sacrifice in swath width, and that we would require a system adapted to the various beam modes available on RADARSAT-1.

The original effort concentrated on the detection capability using ScanSAR Narrow “B” (SCNB) mode (Figure 4). This mode was a good compromise with a 50 meter resolution (25-m pixel) offering a swath width of 300 kilometers. The SCNB mode was preferred due to the shallower incidence angles which reduces the background ocean clutter (or backscatter), thus increasing the contrast with potential bright iceberg and other point targets. All ScanSAR modes are composed of higher resolution single beam modes (e.g. Wide-1, -2 and -3; see figure 4) “stitched” together to provide a wider swath coverage. Each of these beam modes corresponds to a range in incidence angles. Wide-1 corresponds to the steeper range of incidence angles which was found to be more susceptible to ocean backscatter in windy conditions. The Wide-3 beam mode corresponds to the shallow incidence angle range (39-45°) which provided a better probability of detection during the investigation work done by C-CORE.

Use of ScanSAR Narrow B mode still only allowed the detection of medium-size icebergs at best (iceberg size dimensions are listed in the Manual of Ice – MANICE; MSC, 2005). In order to be able to reliably and accurately classify a target as a ship or an iceberg, one needs the highest possible resolution images, such as Fine beam, but these provide only a very narrow swath width (50 km) and are not viable to hunt for icebergs over a large body of water such as the Canadian 200 nautical mile exclusive economic zone and beyond. Through investigation and experimentation, C-CORE determined that the best possible compromise was achieved using the Wide-3 beam mode (see figure 5) with the shallowest look angle of the Wide modes and offering a nominal 30m resolution (12.5m pixel size) with a swath width of 150 km. At this resolution a bright pixel surrounded by darker tone pixels would be able to detect a small iceberg/target/ship depending on the signature obtained.

4. The Iceberg Detection Software (IDS):
Based on hundreds of SAR images and field observations of icebergs and vessels collected over the last 10 years, C-CORE has developed (and continues to enhance and improve) their Iceberg Detection Software (IDS) which automatically detects potential iceberg targets in SAR imagery. Over the last decade, C-CORE has carried out several validation programs with both aerial and surface-based observations as part of this development. Additionally, The U.S. Coast Guard International Ice Patrol (IIP) provided many hours of ground truth flight information when possible in order to advance the proper classification of the satellite detections. Provincial Aerospace Limited (PAL) – a local Newfoundland airline company managing the ice and iceberg information for the offshore industries – has also provided numerous flight hours of aerial
observations. With all of those data C-CORE has developed a significant database of iceberg signatures and ship signatures and continues to populate it with all information derived from subsequent missions.

Target detection outputs from the IDS are quality assessed by expert analysts at C-CORE and standard MANICE-coded messages are generated and transmitted to the CIS and IIP. The SAR-derived target information is then reconciled against and integrated manually with the current operational iceberg database in the Berg Analysis and Prediction System (BAPS) used at both the CIS and the IIP. While the IDS processing is intended to reduce the need for detailed manual analysis of every image plus providing a ‘first-guess’ of potential iceberg targets on which to focus, there are several issues to be aware of when automatically processing radar imagery.

For example, the optimized beam mode often includes an imaging artifact called a nadir ambiguity (a bright line running vertically on the image parallel to the track of the satellite) which can confuse the algorithm into ‘detecting’ multiple erroneous targets. A methodology is needed to mask this anomaly prior to running the algorithm on the image but currently this step requires manual intervention. Another issue pertains to the land mask and small islands being potentially detected as icebergs because of their brighter reflectivity. An accurate map on the east coast of Canada (scale 1:50,000) is being used to mask the coast and most of the small islands along the Newfoundland and Labrador coast while a 1:250,000 map is used outside of the iceberg operational area. Care must also be taken when using the algorithm on images which contain sea ice which can also confuse the algorithm. Manual pre-processing of a frame which allows the user to select a polygon to be defined where the IDS will be run can alleviate this to some extent. Additionally, careful ordering of imagery can minimize this problem. Another factor is the variability of the ocean backscatter response in the imagery as a function of the wind roughening due to storms passing over the regions of interest which affects the ability to detect icebergs, and other targets such as ships. These factors must be considered when using SAR imagery and the IDS for iceberg detection. The current operational practice of having to order imagery two to three weeks ahead of time along with all other factors mentioned above prevents us from obtaining a 100% trusted output throughout any given image.

The IDS algorithm has been improved with every new version to the point that not only detection is possible but also classification. At first, the IDS operator is tasked to scrutinize the image and double check the classification of the target into three different categories – iceberg, ship or target. The operator can overwrite the IDS output if necessary to provide a more accurate output to the end users. Some targets can exhibit certain characteristics that aid in classification. For example if a side lobe signature indicating a very strong backscatter likely from a steel structure is present, the operator and the algorithm can quite reliably classify the target as a ship. The algorithm is fairly robust in determining several parameters with respect to each target detected. Parameters used for classification include shape parameters, morphological features and intensity features. Specific examples include mean, variance, maximum pixel intensity, area, major axis and minor axis length. Several of these features are also used to automatically populate the various mandatory fields required for processing a MANICE iceberg message into the BAPS.
5. Operational Issues and Use of SAR-Derived Target Information

MANICE reporting includes a section for iceberg reports originally designed on a 5-digit code created primarily for airborne and ship borne observations. When it was decided to consider space borne detection, the 5-digit code was amended to allow for a radar swath signature of 500 km (typical of a RADARSAT-1 ScanSAR wide A swath) to exist. This amendment to the code was used to address other issues and deficiencies previously noted but never addressed such items as the valid date of the message. It was difficult to identify the decade of an archived message when the 5-digit code describing the day, month and year was used (ddmmy). As a result, the amended code was no longer restricted to 5-digits but was expanded to reflect up to 7-digits in specific sections of the code. Several tables were updated including the platform type table which now allows for space borne observation - a category which did not exist when the code was originally developed. Also, the confidence level table now reflects three new confidence levels (satellite high, medium and low) added to the existing Radar, Visual or Radar & Visual categories.

As mentioned earlier, the resolution of the imagery can be used to support the classification of some targets into an iceberg or ship class, but often the size and shape of the radar signature cannot be reconciled in classifying the target with 100% confidence. Essentially, space borne detections are classified as icebergs, ships and targets (or unknown) when there is uncertainty. How to treat these unknown targets quickly becomes a nightmare from an operational point of view as well as from a statistical point of view. If an unknown target is treated as an iceberg, the overall population has a tendency to grow and the limit of all known icebergs expands at times too rapidly. If it is discarded or treated as a ship, then there is a risk to end up with a potential iceberg outside the published limit. Statistically speaking, treating a target as a berg when in reality (after ground truth) it is a ship means that the false alarm rate is high. Currently, CIS ignores the unknown target classification unless there is a recent iceberg report within a short distance of the newly detected target which could easily be reconciled with.

The iceberg program along the eastern Canadian coast is only one of many operational programs undertaken by the CIS in coordination with C-CORE who acquire data for the CIS/IIP programs and also collect high resolution Fine beam mode data around coastal locations for tourism. The CIS sea ice monitoring program is very much active between the months of January and July and by far is the program requiring the most acquisitions of satellite imagery to provide factual ice information on a daily basis. The Integrated Satellite Tracking of Polluters (ISTOP) program is also very active year around but tends to concentrate its acquisition along the major shipping corridors. Nevertheless, this program utilizes a different mode of RADARSAT-1 (ScanSAR Narrow-A) which can conflict with other acquisition mode more preferable for iceberg detection or the sea ice program. In order to allow for every program to benefit from satellite imagery, careful acquisition planning is required. It must be pointed out that when the satellite is being re-configured to acquire a different beam mode for a different client, there is a physical portion of the image in the data acquisition which is lost during the beam switch. Thus, there can be potential gaps in coverage for the various applications.

Despite the above challenges and limitations, the IDS output is currently being utilized in a semi-operational aspect at the CIS. IDS target information derived from SAR imagery is being used along the Labrador coast during the fall twice monthly to provide a potential re-sight capability
between reconnaissance flights. CIS understands the limitation of this work as the icebergs, at that time of the year, might not be of large enough size to be readily detected. Moreover, the images are acquired three weeks ahead of time and there is no way to be certain the imagery will be free of atmospheric (i.e., wind roughening) effects. The CIS is evaluating the cost benefit of using IDS away from the main shipping corridor i.e., the international trade route transiting the Canadian Grand Banks in order to gain a better understanding of the operational repercussion of using this output year around keeping in mind that IDS must be operated outside any sea ice region for now. The IDS is also used for data collected for iceberg population surveys in Davis Strait in the fall to get a sense of the anticipated icebergs in the southern regions in the upcoming season (see figure 6).

Starting in 2008 upon the retreat of sea ice, IIP will be working closely with C-CORE to order surveillance frames near the Limit of All Known Ice (LAKI) to fill the gaps between scheduled bi-weekly aerial reconnaissance detachments. IIP will be adding all IDS high confidence targets to the database, and closely examining all low confidence targets for possible re-sight opportunities of small or medium targets already present in the database. If and when any low confidence targets are re-sighted as icebergs, IIP will provide that information to C-CORE to aid in the enhancement and improvement of IDS.

6. Envisat ASAR and Other Advanced SAR Missions

On March 1st 2002, the European Space Agency (ESA) launched Envisat and with this new satellite came new technology. The Advanced Synthetic Aperture Radar (ASAR) on board can be used in various modes, including the Wide Swath (WS) mode which provides a 400 km swath with 75 m pixels (150m resolution) and an Alternating Polarization (AP) mode which offers swath widths from 56-105 km at 12.5 m pixel spacing (~30m resolution) and 3 combinations of dual-polarization: HH/HV, VV/VH, and HH/VV. It was expected that AP mode would help better discriminate between ship and icebergs and indeed this was found to be the case (Howell et al., 2004). Combining the dual-polarization capability with smaller pixel size and better resolution facilitates improved detection capability for smaller targets and smaller targets also means being able to differentiate between a smaller vessel and a smaller iceberg. However, the biggest caveat of this mode is the limited swath width as it takes several frames to map a large area of water through which bergs migrate. Regardless, the classification capability is better with the Envisat ASAR data.

RADARSAT-2, the successor to RADARSAT-1, was successfully launched on December 14th, 2007 and with its wide-swath dual-polarization and narrow swath quad-polarization capabilities hold some interesting possibilities for iceberg detection. Recent work (C-CORE, 2008; Howell et al., 2007) with airborne quad-polarization data shows excellent possibilities for improving the discrimination of and classification of icebergs versus ships. However, the quad-polarization modes on RADARSAT-2 are limited to 25 km, and thus are not particularly useful for broad area iceberg mapping. However, Wide 3 and ScanSAR Narrow currently used will now be available with dual-polarization. Wide 3 will offer similar capabilities as the Envisat dual-polarized mode with an increased swath width and ScanSAR Narrow with dual-polarization will be worth exploring as well.
7. Summary:
The iceberg programs which monitor the iceberg infested water in the western portion of the North Atlantic to advise mariners of the boundary separating ice free water from bergy water will continue to rely on visual reconnaissance flights for several years before an automated process can be implemented. Nonetheless, the early work done with the automatic detection of targets from SAR satellites will undoubtedly pay off in the longer term.

One of the significant challenges for these programs is discriminating between ships and icebergs. If at any given time one was aware of all the ship’s position over a portion of the ocean, then subtracting the known ship’s position from the total targets detected by the IDS would provide a much better understanding of the iceberg population – above a certain size (determined by the image resolution) – over the water area. Of course, this assumes that all bergs over a certain size would be 100% reliably detected, which is not presently the case. That being said, the Automatic Identification System (AIS) imposed to all ships over 300 tons will help in the foreseeable future to remove some ambiguities with respect to radar target classification. Moreover, if the regulation continues to lower the tonnage of ships which must be equipped with an AIS transponder, then the iceberg program could become primarily automated with minimal human supervision. A SAR satellite equipped with an AIS receiver would be useful in order to capture a more global picture and allow identification of ships (versus iceberg) over a huge portion of the ocean. However, this will not likely be available until the CSA RADARSAT Constellation satellite mission or the ESA Sentinel mission. Until then, satellite SAR data will be used to augment the traditional iceberg reconnaissance methods to assist in detecting various targets, but it cannot yet provide a 100% reliable solution.

References


Figure 6. Use of IDS in Davis Strait fall survey. The purple triangles represent the IDS detected icebergs within the dataset acquired. Unfortunately, at this scale it is impossible to see the “bright” dots in the center of each triangle; what can be seen are the atmospheric disturbance signatures described earlier in the paper.
Figure 1. Graphical representation of all iceberg sightings modeled to the current date (analysis drift). Longer model drift tracks represent older sightings while shorter one shows recent addition to the database.
Figure 2. Display of airborne MANICE-coded iceberg message along with its graphical representation of surveyed area on BAPS.
Figure 3. Daily iceberg density chart from CIS

Figure 4. RADARSAT-1 Beam Modes
Figure 5. Radarsat-1 Wide-3 image with ship and iceberg targets
Adaptation of Radar Based River Ice Mapping to the Nunavik Context

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Over-ice transport is an essential element of life in the north. Native people are commonly using the inland rivers and lakes network to access their hunting and fishing grounds. However, these ice routes are being strongly impacted by recent warming and associated reductions in the seasonal duration and thickness of ice, which may lead to increasingly risky areas or inaccessible trails. In this context, the Kativik Regional Government (Nunavik) has initiated an ice monitoring program along the trail networks surrounding certain communities, relying on traditional knowledge and scientific measurements. Through the Canadian Government International Polar Year Program, a complementary monitoring approach is being tested. This paper presents the process developed to adapt and implement radar based river ice maps to the context of the Koksoak River area (Kuujjuaq, Qc). The first step was to meet with local hunters and experts to establish the trail network, to acquire the Inuit’s perspective about river ice in the area (ice features, changing conditions, risky areas) and to discuss a product that would suit their needs (boundaries, frequency, medium, language, etc.). The second step of the process was to plan for a test season, with a weekly near real-time prototype product. Aspects of this planning would involve the adaptation of image processing methods and map design, as well as developing means to deliver the maps to the users. The third step of the process was to develop a validation protocol, which relies on field observations. This part involves the installation on site of instruments such as an ice profiler (SWIP) and a high resolution webcam. But it also involves the map users themselves, with feedbacks on the maps, acquisition of ground photos and measurements of ice thickness. Preliminary results of the 2007-2008 winter season are also presented.
Augmenting a River Ice Flood Forecasting Service Using Satellite RADAR Imagery

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The development of ice covers on large rivers is a dynamic process that can result in ice jamming and flooding of large areas. One place subjected to this type of flooding is the Town of Badger on the island of Newfoundland in Newfoundland and Labrador. Badger, located at the confluence of three rivers, has a history of flooding dating back to 1916. The Government of Newfoundland and Labrador through the Water Resources Management Division provides an annual flood forecasting service for Badger using a computer simulation model that simulates ice conditions on the Exploits River. Following the severe flood of February 15, 2003, the flood forecasting service was augmented using satellite RADAR imagery from the ENVISAT and RADARSAT polar orbiting satellites. This was the first integration of RADAR imagery into operational flood forecasting of river ice in Canada. This paper describes the River Ice flood forecasting service at Badger, the integration of RADAR imagery into the service, and the improvements that were accomplished over the past four years. It describes the derivation of ice parameters from RADAR data, the use of this information in model calibration and validation, and the path forward for this service.
1. Introduction

The development of ice covers on large rivers is a major concern for water resources management, hydropower generation and flood damage prevention. Rivers normally freeze up in an upstream direction in a highly complex and dynamic process. Of particular concern is the formation of consolidated ice covers from thinner layers of frazil ice, which in turn can cause ice damming and flooding over large areas. The severity and economic impact of floods related to ice dams is exacerbated by the danger of post-flooding freeze-up (Puestow et. al. 2003).

One place subjected to this type of flooding is the Town of Badger on the island of Newfoundland in Newfoundland and Labrador. Badger, is located at the confluence of two small brooks, Red Indian Brook and Badger Brook, in the Exploits River (Figure 1).

2. Flooding at Badger and Flood Forecasting

Badger has a history of flooding dating back to 1916. There have been no floods in Badger in ice-free conditions and all of the major floods in Badger have occurred in either January or February. It has been determined that these floods were caused by the rapid generation of frazil ice which jams the Exploits River just downstream of Badger (Fenco Newfoundland Limited, 1985). While the flooding in the lower areas of the town adjacent to the Exploits River has fortunately not resulted in any deaths, the economic impact has been severe.

The Government of Newfoundland and Labrador through the Water Resources Management Division (WRMD) monitors flow on the Exploits River through a water level station at Badger and through two flow measuring stations situated approximately 28 km upstream of Badger, below the confluence of Noel Paul's Brook with Exploits River and at the dam on Red Indian Lake. The data for all three water monitoring stations is available to the public via a dedicated flood forecasting web page (http://www.env.gov.nl.ca/wrmd/Badger/default.asp) maintained by WRMD. The information is automatically updated several times each day.

WRMD also provides an annual flood forecasting service for Badger using the custom-developed Ice Progression Model (IPM) to simulate ice conditions on the Exploits River. The ice forecasting season for Badger starts with freeze-up in December and continues to the break-up of the ice cover in April.

Initially the WRMD started carrying out river flow monitoring for Badger following a flood in 1977. In 1985, the IPM was developed as part of a hydrotechnical study of the Badger and Rushy Pond areas (Fenco Newfoundland, 1985). The model was further refined in 1995 (Fenco MacLaren, 1995).

The IPM uses a mathematical model to calculate the volume of ice generated in the open sections of the Exploits River. For this purpose, the IPM divides the Exploits into 32 sections between Grand Falls and the Exploits Dam at the outlet of Red Indian Lake (Figure 2). The water temperature in each segment is initially simulated on the basis of meteorological data and discharge information. When air and water temperatures in a segment fall to below freezing, frazil ice slush is generated in that segment, and combined with that of other segments and
carried downstream. The slush passes over Goodyear’s dam near Grand Falls until it is blocked by border ice growth at the dam or by the ice boom just upstream. Once the downstream progression is stopped, the slush from upstream segments begins to accumulate in segments upstream of the dam. The ice cover grows upstream from the dam until it passes Badger and moves on up to Three Mile Island and beyond. If the IPM indicates that over 2 MCM (million cubic meters) of ice is being generated and coming down the Exploits River between Red Indian Lake and the Town of Badger an alert is issued to the town and the province’s Emergency Response Organization.

On February 14, 2003, two IPM runs were made. The morning model run forecasted 2.6 MCM of ice being generated and the afternoon forecast run estimated 3.1 MCM to be generated. As per protocol the Town of Badger and EMO were advised that there was a considerable amount of ice coming down the Exploits. However since the ice cover had progressed pass an in river island called Three Mile Island (about 5 km upstream of Badger), the town and the WRMD agreed that there a flood alert was not needed since historically flooding had not occurred at Badger once the ice cover had passed Three Mile Island. The ice cover was assumed to be stable and no change was expected in the water level.

On the morning of February 15, the ice cover collapsed and in less than an hour between 8.00 AM and 9.00 AM, the water level rose 2.3 metres. It rose an additional 0.3 in the next two hours and peaked at an elevation of approximately 100.5 metres (geodetic). The flood led to the evacuation of the Town and the declaration of a State of Emergency.

In the days that followed, extremely cold conditions froze the flood waters and encased a large portion of the town in ice for weeks (Figure 3). The flood of February 2003 was the most severe in terms of the depth of inundation, the speed at which the flooding occurred and the damages to the town. The damages from the February 2003 flood have been estimated to be approximately $10 million (Picco et al., 2003). Following the flood, 25 families were moved out of the worst impacted areas of the flood zone. Remaining homes in the 1:20 year and 1:100 year flood zone were flood proofed and the town office and fire hall were moved to safer locations.

The subsequent flood response and a preliminary analysis of the flood event highlighted a number of shortcomings in the flood forecasting system. The first deficiency noted was that the IPM required the location of the ice front and this was not always available from field observers as many river reaches of Exploits are inaccessible. Moreover, field observations were only possible when weather conditions allowed for observers to be outside. The second deficiency noted was that a method was needed to gauge the stability of the ice cover.

3. Augmenting the Flood Forecasting Service

Recent studies have confirmed the utility of Earth Observation (EO) using Synthetic Aperture RADAR (SAR) imagery for the categorization of river ice types (Weber et al., 2003; Jasek et al., 2003). In particular, RADARSAT-1 data showed prospect as a tool to identify the location of the ice front and gauge the stability of the ice cover. A preliminary test using RADARSAT-1 imagery acquired before and after the February 2003 flood showed that consolidated ice that had been observed in flood response field program was successfully identified in the SAR data.
To further demonstrate and investigate the extraction of ice parameters from satellite imagery in near real-time, and to use this information within an operational flood monitoring context, an image acquisition program for the 2003/2004 flood forecasting season was initiated.

In the 2003/2004 season, the primary concern was the location of the ice front, which delimits the upstream extent of the ice cover, and to use this information to calibrate and validate ice progression model predictions.

The Area of Interest (AOI) is comprised of the section of the Exploits River between Red Indian Lake and Grand Falls, and centered on the town of Badger. The river section is approximately 70 km in length. Due to the location of the AOI within the monitoring mask of the Canadian Ice Service (CIS), RADARSAT-1 Standard Beam Mode imagery was not available, and ScanSAR Wide (SCW) data was used instead. While not an ideal data source in terms of spatial resolution (100 m), SCW data covers the entire AOI several times per week in a single scene, and data loss to RADARSAT data user conflict is unlikely. A total of 16 SCW images were acquired between January 1 and March 9 2004. Ancillary geospatial data included digital map sheets 12A/15, 12A/16 and 2D/13 of the Canadian National Topographic System (1:50,000; UTM Zone 21 NAD83). The digital maps were used as reference data in the geometric correction of the satellite imagery.

The location of the ice front was clearly recognizable on all SCW images. Figure 4 shows the image acquired on February 15, 2004. The ice front location is clearly visible at Three Mile Island. The image products were available for comparison with the ice progression model within four to six hours from image acquisition.

In the first year, the imagery was subsequently subjected to a visual interpretation into the interpretive classes presented in Table 1. An example of interpreted ice classes is provided in Figure 5.

The observation and modelling of ice development on the Exploits River was enhanced significantly by the use of RADARSAT imagery. The accurate location of the ice front reduced the uncertainty associated with the timing of the flood.

4. Improvements to the River Ice Service

Over the next four ice seasons 2004/2005, 2005/2006, 2006/2007 and 2007/2008, the river ice service developed in the 2003/2004 season was improved in several ways:

a). Use of ENVISAT Imagery

SAR imagery from the ENVISAT satellite was added to reduce RADARSAT data user conflict and to gain access to Alternating Precision Polarization (APP) modes IS1 through IS7. The APP modes were collected to investigate the information that could be gleaned from polarized imagery. The polarized channels could not be analyzed under the current river ice service but the information was collected for further analysis under a
parallel research project currently in process to investigate the use of dual- and fully-polarized RADAR imagery for the characterization of river ice. Updated ice classification algorithms resulting from this activity will subsequently be integrated into the existing monitoring process.

b). Change Detection Product

A change detection product was added in the 2004/2005 season to analyze ice stability. An example of a change detection product is provided in Figure 6. Figure 6 shows a Change Detection between February 15, 2008 10:17am (NST) and February 14, 2008 10:09pm (NST). Both are ENVISAT APP IS7 (great for comparison) but have opposing orbits. The Feb 14 orbit is ascending. Different change detection methods were investigated. Ratio between images was used in the 2004/2005 season but better results were obtained using differencing between images from 2005/2006 onwards. To generate the change detection product the SAR image is standardized, to account for differing beam modes, by calculating the mean and standard deviation of the calibrated image. The previous image (also standardized) is then subtracted from the working standardized image. A mask is used to ensure only the river is processed. The results of the change detection product were further improved in the 2007/2008 season by implementing protocols under which only images with similar beam modes being compared. The change detection product needs further improvement. The change detection product will be researched further under the parallel research project.

c). Ice Classification Product

After the preliminary ice classification product that was developed in the 2003/2004 season and shown in Figure 5. An ice classification product was not used generated for the Badger River Ice Service until the product was resurrected in the 2007/2008 season using a three level classification schema shown in Table 2. An example of interpreted ice classes is provided in Figure 7. This is for February 15, 2008 10:17am Newfoundland Time. ENVISAT APP HH/HV Descending orbit, IS7. The ice front can be identified in this product as being in segment 15. The ice classification is based on classifying backscatter. The SAR image is queried looking for the image value thresholds separating open water from non-consolidated ice and from consolidated ice. A script is used to create a classified image to be overlaid on the SAR image. A mask is used to ensure only the river is processed. A supporting field program was initiated to provide field photographs to be used in the ice classification. Based on the initial results from the 2007/2008 ice season, a more comprehensive field program is planned for the 2008/2009 season to improve the ice classification. The further use of ancillary data such as water temperature from real time water temperature sensors in the Exploits River will also be investigated in the 2008/2009 season. In the parallel research project the use of texture based classification will also be investigated to improve the ice classification product.
d) RADARSAT Beam modes

After using SCW data in 2003/2004 season, RADARSAT-1 Standard beam modes S1 through S7 were used in subsequent seasons. On rare occasions, the SCW mode had to be used due to swath orientations and conflict with the CIS from time to time.

5. Conclusions

The augmentation of the Badger Flood forecasting service through the use of Satellite RADAR imagery from the ENVISAT and RADARSAT polar orbiting satellites has significantly enhanced the observation and modelling of ice progression on the Exploits River. The accurate location of the ice front reduces the uncertainty associated with the timing of the flood. The use of RADAR Images in an operational sense to determine the location of the ice front provides several important advantages over volunteer observations and / or optical imagery. RADAR offers: a "big picture" view, all weather operation (i.e. largely unaffected by clouds or precipitation), and twice daily observations if needed. The importance of RADAR imagery for accurate location of the ice front cannot be overstated. It represents a major improvement in the flood forecast for the residents of Badger.

6. Path Forward

RADAR imagery has been used as the primary source of information on the location of the ice front on Exploits River for the last 5 years and RADAR images will continue to be needed in the future for accurate location of the ice front. While the River Ice Service has improved the location of the ice front, to provide an insight into ice stability the change detection product and the ice classification products require further research and development. The River Ice Service has been pioneered by C-CORE in collaboration with WRMD and is being adopted by a variety of provinces in Canada and internationally in Russia. This development of an extended user community is expected to improve the River Ice Service and provide a forum for exchange of ideas and experiences and standardization of practices. The parallel research project investigating the use of polarized RADAR imagery will address some of the deficiencies of the River Ice Service. One unknown in the continued use of the River Ice Service is the pricing of RADARSAT 2 imagery. If NRT imagery from RADARSAT 2 is priced above current RADARSAT 1 prices then the River Ice Service will not be economically viable for many users despite its demonstrated utility.

Acknowledgments

This RADAR imagery service has been provided by C-CORE under the Polar View initiative and has been funded by the European Space Agency’s Global Monitoring for Environment and Security (GMES) program. All RADARSAT imagery was provided by the Canadian Space Agency in support of GMES.
References


Figure 1. Badger.

Figure 2. IPM River sections 17-26
Figure 3. Flood damage at Badger - February 2003

Figure 4. RADARSAT SCW image on February 15, 2004
**Figure 5.** Interpreted February 15, 2004

**Figure 6.** Change detection product comparing Feb 15 10:17am (NST) and Feb 14 10:09pm (NST).
Figure 7. Ice Classification product for February 15, 2008 10:17am Newfoundland Time. (ENVISAT APP HH/HV Descending orbit, IS7)

Table 1. 2004 Description of Interpretive Classes

<table>
<thead>
<tr>
<th>SAR Image Interpretation (Backscatter)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dark Ice/Open Water</td>
<td>Smooth ice cover or open water</td>
</tr>
<tr>
<td>Medium-Dark Ice</td>
<td>Ice cover with some roughness</td>
</tr>
<tr>
<td>Medium-Bright Ice</td>
<td>Rough ice/consolidated ice cover</td>
</tr>
<tr>
<td>Bright Ice</td>
<td>Heavily consolidated ice cover</td>
</tr>
</tbody>
</table>

Table 2. Description of Interpretive Classes for Ice Classification Product

<table>
<thead>
<tr>
<th>SAR Image Interpretation (Backscatter)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low return</td>
<td>Open water, water on ice, or smooth ice</td>
</tr>
<tr>
<td>Medium return</td>
<td>Rough ice, snow covered, non-consolidated</td>
</tr>
<tr>
<td>High/bright return</td>
<td>Heavily consolidated ice</td>
</tr>
</tbody>
</table>
Monitoring lake ice in Norway using remote sensing (MODIS, RADARSAT), system development

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Abstract

The principal aim of the study is to establish an operational processing chain in order to monitor Open Water Surface (OWS) for lakes larger than about 10 km$^2$ and hopefully some of the largest rivers (i.e. more than 100 m wide), during winter time. We will apply three key operational satellite sensors: NOAA AVHRR, Terra MODIS and Radarsat (1/2) ScanSAR. Most lakes in Norway have an area of less than 100 km$^2$. Thus passive microwave images cannot be used due to the relatively low satellite resolution. Multi-temporal information will be used to merge radar data into time gaps in optical acquisition. For validation we will use ground observations and higher resolution satellites (Landsat ETM+, Terra ASTER/MODIS, Radarsat-1 S7) if data is available. OWS presently needs pixel to sub-pixel information, due to the relatively small features that should be mapped.

Due to significant differences in surface roughness of various type of land (land, snow on frozen lakes, glaciers, forest etc) the spectral signature or backscatter coefficient will respond differently in time and space. The images will therefore be masked for various land types. For OWS study the mask should remove all the pixels that do not belong to the water classes.

Results from spring 2007 and 2008 have shown that OWS maps can be very useful for discriminating ice-free and ice-covered lakes during break-up situations. OWS maps can be used in an overlay analysis with maps of lakes to create maps of ice covered lakes. Lake ice has a strong influence on local energy budget. Lake-ice freeze-up and break-up dates are also good indicators of regional climatic variability and change and important knowledge in real time to calculate ice strength for safe traffic on the ice. The images will improve our Internet based ice information system “ice warning” by increasing spatial cover in regions with sparse data.
1. Introduction

Very large expenses of land at northern latitudes are covered by lakes. Lake and river is also an important component of the Norwegian terrestrial cryosphere. Therefore, as a major component of the terrestrial landscape, unfrozen and frozen lakes play a significant role in the energy and water balance of cold regions. Lake ice break-up dates are also good indicators of regional climate variability (Livingstone, 1997). A change of only a few degrees in air temperature is sufficient to shift freeze-up and break-up dates by several weeks (Liston and Hall, 1995, Walsh, 1995).

Remote sensing has shown to be a powerful tool to map river and lake ice (e.g. Jeffries et al., 2005). Recently, investigation have demonstrated the potential of Moderate Resolution Imaging Spectrometer (MODIS) and Advanced Very High Resolution Radiometer (AVHRR) data for monitoring ice break-up in Arctic river ice break-up (Pavelsky and Smith, 2004) and Latifovic and Pouliot (Latifovic and Pouliot, 2007) have used AVHRR data to determine lake ice phenology. Duguay (2002) demonstrate that Radarsat-1 and ERS-1 Synthetic Aperture Radar (SAR) can be used to monitor the dates of freeze-up and break-up on lakes. Similarly have other studies shown that SAR imagery can be used to detecting small-scale pattern in ice process (Vincent et al., 2004, Weber et al., 2003) and river-ice break up.

This paper provides an overview of the operational system that is under construction for Open Water surface (OWS) mapping during the, winter and spring season from Earth Observation (EO) - data. The open water classification work is conducted as a part of Norwegian project CryoRisk. The main objective of CryoRisk is to modernise and expand the public services at Norwegian Water Resources and Energy Directorate (NVE) and the Norwegian Meteorological Institute (METNO) for monitoring the cryosphere using EO data. To achieve this objective NVE and METNO will establish an operational national production environment for snow cover extraction from satellite remote sensing data. In addition will we also try to use the same production environment to create Open Water surface (OWS) maps which can be used to discriminate ice-free and ice-covered lakes and rivers during freeze-up and break-up situations. OWS maps can be used in an overlay analysis with maps of lakes to create maps of ice covered lakes. It is also very useful for to assessing safety of traffic and transport on lakes and will be used to improve our internet based ice information system “ismelding” by increasing spatial cover in regions with sparse data.

The principal aim is to establish an operational processing chain in order to monitor OWS for relative small lakes (10-300 km\(^2\)) and hopefully some of the largest rivers (i.e. more than 100 m wide), during winter time. The use of optical remote sensing alone is however, unreliable in the rough and clouded climate in Norway. Periods of 1-2 or more weeks without coverage due to clouds are frequent in the important ice break-up situations. Use of the weather and light-independent Synthetic Aperture Radar (SAR) is very attractive in this respect. We will therefore, applying three key operational satellite sensors: NOAA AVHRR, Terra MODIS and Radarsat (1/2) ScanSAR. Multi-temporal information will be used, such that radar data fills time gaps in optical acquisition. For validation we will to use ground observations and higher resolution satellites (Landsat ETM+, Terra ASTER/MODIS, Radarsat-1 S7). OWS presently needs pixel to sub-pixel information, due to the relatively small features that should be mapped. Automatic
procedures in all parts of the processing chain are desirable, but NVE will permit some manual work if this results in better products from a hydrological/glaciological perspective.

2. System requirement

Spatial coverage
Most lakes in Norway have an area of less than 10 km$^2$ and could be difficult to map by remote sensing. In addition, lakes often have an irregular shape (many bays, peninsulas and small islands) amplifying the problem of mapping by remote sensing. We therefore concentrate or work on lakes greater than 10 km$^2$. All these lakes were extracted for southern Norway from NVE's National Lake Cover database of Norway. In total, 73 lakes have an area larger than 10 km$^2$, of which only four lakes have an area > 100 km$^2$ (Mjøsa 369 km$^2$, Femunden 203 km$^2$, Randsfjorden 140 km$^2$ and Tyrifjorden 137 km$^2$). Figure 1 shows the geographical location of all the lakes in Norway with an area greater than 10 km$^2$.

Satellite data
NVE’s primary interest in using EO data is mapping snow in the terrain and snow/ice over lakes and rivers. All sensors capable of doing this are of interest. However, we focus on sensors being operational or expected to become operational within reasonable time. With open water classification, the spatial resolution must be sufficiently to resolve the presence and absence of ice within the lakes and rivers can be resolved. The use of active/passive microwave sensors such as SAR will be limited to Radarsat (1/2). It is also possible to discriminate between open water and ice on large lakes with passive microwave (Walker and Davey, 1993). However, such images have little use in Norway due to the relatively low satellite resolution. The Radarsat 1 ScanSAR scenes, which are provided by Kongsberg Satellite Services (KSAT) have a spatial pixel resolution of 25x25 metres. This spatial resolution should be adequate for open water classification on lakes and rivers. The temporal resolution should allow coverage of a given area every 4-6 day.

For optical sensors, the focus will be NOAA AVHRR (only on the large lakes), MODIS from Terra/Aqua. The spatial resolution of the instrument becomes more important when the lake size is small and the variation in topography is large. Because of this, the MODIS instruments from Terra and Aqua are preferred due to its optical sensors with its spatial resolution of 250 - 1000 metres. MERIS is also an interesting sensor with its high spatial resolution (260x290 meters pixel size), but the ability to discriminate snow/cloud is a problem, because in the visible near infrared (VNIR) snow and most clouds have very similar spectral characteristics.

Data access and operational environment
NVE has previously manually downloaded NOAA AVHRR images for operational use. Today NVE accesses NOAA AVHRR with FTP from METNO who has a direct readout station for AVHRR in Oslo and also receives data from the North Atlantic. This gives us access to all NOAA AVHRR data in near real time. A similar approach is being set up between NVE and KSAT which delivers geocoded Radarsat 1 data to the project. Access to MODIS data is done through an automated FTP interface to NASA. METNO is currently examining various sources for a real time access to MODIS data (e.g. through EUMETSAT EUMETCAST), until this is in place FTP access through NASA will be used. The MODIS data available from NASA lags by approximately one day. Data access to the sensors listed above is rather straightforward.
The operational environment is usually designed to minimise human intervention and if this is required, the application should explain in detail how to handle the exception. The operational environment is divided into: - Pre-processing (data download, data conversion, geocoding): This will be done using in-house software using IDL as the programming language and ENVI as a library. - Algorithm implementations: Same as pre-processing; and - Distribution: this will be done using Microsoft .Net platform which is NVE’s preferred development environment.

**Temporal coverage**
Lake ice coverage varies in time due to changing temperature, radiation and other meteorological variables. The bulk water temperature (the average temperature between the lake near surface and lake bottom) at which lakes freeze over is related to fetch, with small lakes freezing at a bulk temperature of 2-3°C and large lakes freezing at a bulk temperature of 1°C. Lake depth and volume are also important parameters of freeze-up (Stewart and Haugen, 1990). The combined effects of lake fetch, depth and volume result in different freeze-up times from lake to lake. In order to map change in OWS and thereby freeze-up times, a relatively large temporal coverage is needed because lakes of different size, fetch and depth freeze up at different times. The temporal resolution must be high enough to identify quickly changing patterns in freeze-up and ice break-up in the area of interest, with daily to weekly time series of images as a practical goal. In the first stage of the program NVE’s main focus is in the melting period, typically from March to early July, because this will cover the ice break-up times from the costal areas to the highest mountain areas.

The minimum time resolution accepted for open water surface is a weekly product that is updated on a daily basis using all available satellite data to reduce the effect of clouds. Only the most promising satellite data (e.g. for optical sensors only images with reliable discrimination between clouds and snow/sea ice) will be analysed, thus some human inspection is accepted in the process of OWS classification.

Geolocation accuracy is important to NVE when algorithms are used to classify open water. It is then important to classify only the pixel which lies within the boundaries of the specified lake or river. Geolocation accuracy has been manually checked from geocoded Radarsat ScanSAR scenes delivered from KSAT. The geolocation accuracy observed in the scenes varies between 3 – 9 pixels, where each pixel is 25x25 metres. This is considered good enough when observing snow covered area (SCA), but could be a problem for open water classification. Figure 5 gives an example where GIS data is infused into a geocoded scene produced by KSAT.

3. **Auxiliary data available**

**Gridded precipitation and temperature observations**
All available observations from the public meteorological network observing air temperature and precipitation are used to interpolate fields of daily values at 1 km by 1 km resolution for Norway (Engeset et al., 2004a, Tveito et al., 2002). Gridded data are available from 1961 and updated every morning at about 10 a.m. Temperature is observed at about 150 stations and precipitation at about 630 stations at present.

**Gridded snow maps**
National coverage snow maps are produced on a daily basis using a gridded snow model operating on the 1 km² / one-day grids of precipitation and temperature described above (Engeset
et al., 2004a, Engeset et al., 2004b, Tveito et al., 2002). All relevant snow simulations are presented on the web site www.seNorge.no together with the meteorological data (further description of web site, the web services and the data is available at www.seNorge.no). Figure 2 shows examples of gridded snow maps of 4. June 2008 in southern Jotunheimen (mountain area) situated in southern Norway.

4. Algorithms

Open water classification by optical sensors

Lakes

Due to significant differences in surface roughness of various type of land (e.g snow on land, snow on frozen lakes, glaciers, forest etc) the spectral signature will responds differently in time and space. The images should therefore be masked for various land types. For OWS study the mask should remove all the pixels that do not belong to the water classes.

A snow-free newly frozen lake often appears dark in the visible band and can be difficult to distinguish from open water. Consequently, there can be problems in identifying the timing of initial ice formation with visible band data. On large lakes, a considerable thickness of ice is required to resist wind forces without breaking, and initial ice cover formation is much more dynamic than on small lakes. It will therefore take time for the whole lake to freeze and in some years only part of the lakes will be frozen. Fracturing, movement and refreezing of the ice cover during the freeze-up event may cause difference in reflectance and is thus detectable from optical satellites. When snow starts to accumulate on the lake ice the snow covered areas will be possible to distinguish from open water with optical satellites. There could however be several weeks offset between the formation of lake ice and the time of first snow on the ice. The exact time of freeze-up could therefore be difficult to be determined. Similarly, it may be difficult to determine the exact time of break-up. The lake ice break-up in Norway normally starts after the snow have melted completely. If snow ice has been formed during the winter it will be exposed. Because of its high albedo it could be difficult to discriminate from snow. Further melting will gradually expos congelation ice and the colour turns in to dark grey and the albedo decrease. Again, as for the freeze-up situation, it could be difficult to discriminate between open water and snow free congelation ice at the end of the lake ice period.

During the winter and spring snow covered areas on Norwegian lakes will be follow by optical satellite images. We will use the snow product (snow classification) developed in the CryoRisk project together with a lake ice mask to determine OWS vs snow covered and frozen lakes.

Rivers

We do not expect much information on rivers since Norwegian rivers are relatively small compared with the satellite resolution, where AVHRR has 1 km resolution and MODIS has 500 or 250 m resolution for the channel planned to be used. However, some initial studies have shown promising results using the satellite derived snow maps for simply mapping the presence or absence of ice. The break-up of river ice is controlled by both thermodynamic/hydrothermal and dynamic/hydrodynamic processes (Prowse, 1995). Therefore the river ice will often break-up before all the snow has melted from the ice surface. Because the contrast in spectral signature between snow covered river ice and open water is large it should be possible to monitor break-up in larger rivers (e.g. Pavelsky and Smith, 2004). We will therefore use the snow product (snow
classification) described earlier together with a river/lake ice mask to determine break-up patterns/timing.

**Classification by SAR**

**Snow classification**
SAR satellites presently are limited to the C-band (5.6 GHz). The radar signal in this band isn’t affected by clouds as are optical sensors. This is a great advantage when it comes to monitoring snow covered areas, but there are some drawbacks. The C-band is capable at detecting wet snow only. The sensitivity to dry snow is very low. This has led to an alternative way of detecting snow covered area where the wet snow results are combined with a height model (DEM) to infer dry snow (Malnes, 2007, Nagler and Rott, 2000). This method has its drawbacks as the area of dry snow will contribute 100% to the total snow cover area. This will usually overestimate the snow cover for dry snow, as it seems likely that parts of the terrain with steep slopes and those parts which are facing south may be snow-free.

The most common way to classify wet snow from SAR is to take the difference in dB between an acquired image and a reference image taken under snow free or dry snow conditions (Baghadi et al., 1997, Nagler and Rott, 2000). This method uses the idea that wet snow has low backscattering compared with dry snow and bare ground which have strong backscattering. This is valid only when the area is not forested, as these will contribute to the backscattering even if the snow is wet and reduce the difference between bare ground/dry snow and wet snow. Therefore the algorithm is applicable only for areas without forest. The difference in dB approach eliminates the topography which affects the backscattering. The Nagler wet snow algorithm (Baghadi et al., 1997, Nagler and Rott, 2000) describes a scheme as given above where a pixel is classified as wet snow if the difference is less than -3dB.

A challenge with this approach is that it is difficult to acquire good reference images. The reference images must have same satellite geometry (viewing angle) as the image which it is compared used with. Malnes (2007) describes in detail Norut IT implementation of the Nagler algorithm.

**Open water classification**

**Lakes**
Due to significant differences in surface roughness of various type of land (land, snow on frozen lakes, glaciers, forest etc) the backscatter coefficient responds differently. In addition will the backscatter from different categories responds different to wet/dry snow cover. The images should therefore be masked for various land types. For OWS study the mask should remove all the pixels that do not belong to the water classes.

In open areas (non-forested), open water bodies (lakes) will result in a low backscatter compared to the surrounding terrain, since the water will act as a specular reflector, which reflects most of the energy away from the radar antenna (Henderson, 1995). Traditionally, SAR based detection of lakes/floods in non-forested areas has been performed by thresholding the image intensity (Malnes et al., 2002). However, this approach has problems with separating land and water at low incidence angles, since the contrast between water and land decrease with decreasing incident angle and high wind speed.
When lakes freeze and become completely ice covered, the backscatter is low due to specular reflection away from the radar of the relatively smooth upper and lower surfaces of the ice. However deformation features, such as cracks or ridging/rafting features that developed as the wind fractured and displaced the thin ice cover during the ice formation period, will intensify the backscatter. Backscatter intensity from deformation features is strong compared with that of the adjacent ice cover at the beginning of the ice growth period (Morris et al., 1995). As the ice cover thickens there is an increase in backscatter due to the combined effects of reflection off the ice-water interface and forward scattering from bubble inclusions and from slush (wet snow). Further, a frozen lake will be covered by snow on top of the ice during the winter, and the backscatter increases compared with that of open water.

When the snow becomes wet due to either surface melting or flooding, the backscattering reduces to about the same level as for open water (Malnes and Guneriussen, 2002). Flooding and snow ice formation occur when the mass of snow is sufficient to overcome the buoyancy of the ice and depress the ice surface below water level; fractures present allow water to flow up to the ice surface and wet the snow cover, and the resulting slush freezes to create snow ice. Where melting snow cover is present, the absorption of the radar signal will reduce the backscatter. Melting will expose the snow ice, with relatively high backscattering. However the backscatter could be low due to an increase in specular reflection away from the radar receiver, resulting from internal melting of the ice and pounding of water. The detection of ice break-up can therefore sometimes be difficult with radar when the surface becomes wet (pounding and/or wet snow) (Duguay et al., 2002, Hall et al., 1994).

However, when all the ice vanishes and the backscattering from open water is low, the differences in surface roughness may cause significant temporal changes of backscattering coefficient. This change in surface roughness is due mainly to wind action on the lake surface. When the lakes are frozen a more stable backscattering coefficient will be expected. Large temporal change in backscatter may thus be used as an open water indicator. For SCA monitoring which should cover larger part of Norway, mostly RADARSAT ScanSar (SCN16) Fare modes are preferable. The track width is about 500 km and the incident angles vary from 31° to 40°. The backscatter of water (especially under wave action) and snow is highly variable with the incidence angle. The incidence angle dependency of the SAR-images is a problem within one images (particularly for ScanSAR), but also due to the fact that it drives the need to have a different reference image for each imaging geometry. In order to have an operational tool which used all standard products and imaging geometries of Radarsat we need more than 50 reference images for each area that should be mapped.

**Rivers**

In an operational context satellite synthetic aperture radar (SAR) imagery in particular has been shown to yield cost-effective information on ice type in small and medium rivers (i.e. more than 100 m wide). Autumn ice jams can be identified in SAR images because deformation creates rough surface that cause strong backscatter and bright signatures for months after their initial formation. Similarly, winter and spring cracks/ice jams could be detected by SAR (Weydahl, pers. com., 2007).
5. Preliminary test of the system for OWS mapping

**RADARSAT**

The data used is collected from the 10 000 km² area in the southern part of the mountain area Jotunheimen situated in southern of Norway (see Fig. 1 for location). A time series of 17 Radarsat ScanSAR images, (ascending and descending passes) were collected from the spring season (Mars 18 - June 8) of 2007. The geocoded RADARSAT images came from KSAT and were median filtered with a 3 by 3 kernel to reduce speckle. The images were then subsequently masked to remove the terrain that gives ambiguous results from the lake ice cover algorithms. The masking removed the pixels that belonged to the forest or rock classes. In addition we also removed pixels that were distorted due to folding in the close vicinity of the lake (the latter mask varies as a function of the imaging geometry).

Figure 3 shows two unmasked geocoded Radarsat scenes of the test area from May 25 and June 1 2008 projected to UTM WGS-84 zone N33 on a 25 m x 25 m grid. Visual inspection of the two scenes shows evidence of ice on Lake Bygdin and Tyin and partly on Gjende on the May 25 whereas on the June 2 lake Gjende looks ice free. We observed that the difference between backscattering from lake Gjende and the two others lakes Tyin and Bygdin were less than -1 dB one the images from May 25. However in to small areas where we expect the lake to be free of ice on Gjende backscatter was significant lower by – 3 dB. The difference between backscatter from lake Gjende and Bygdin/Tyin where more than – 3 dB one images from June 1. The backscatters on Gjende were similar to the ice free lake Vangsmjøsi some 23 km to the south. Similar backscatter differences were also found on image taken on the June 8. The different in backscatter found on the lakes from the same imagery support the idea of refining the Nagler-algorithem by using different approaches to different land/water categories as proposed by Malnes and Guneriussen (2002). A possible detection scheme for detecting ice on lakes could thus be to only investigate pixels that belong to the water mask. Pixels that belong to the water mask are preserved and processed. The difference between backscatter of known open water lake and possible ice covered lakes in the same imagery could be used to classify open water vs ice covered lake. More investigation is needed to define the expected threshold value.

**MODIS**

MODIS data with a spatial resolution of 250 m were chosen since this fine resolution provided some extra information when mapping the presence or absence of ice cover (snow cover on top of ice) on the relative small Norwegian lakes. A RGB color composite was created by a combination of two MODIS channel 1 (620-670 nm) was assigned to red and blue and channel 2 (841-876 nm) was assigned to green. Clouds are shown as white and land areas without snow are green, areas with thick snow cover are white, thinner snow cover are grey/pink/purple in the image. A daily time series of RGB images was generated for the southern part of Norway starting in April 2008 until early June 2008. The presence or absence of snow on ice was mapped visually. On the larger lakes the differences in the location of the ice breakup front(s) were mapped visually and digitized in GIS environment. The imagery was often affected by clouds which largely obscured observation. In order to reduce the misinterpret of clouds as snow on top of ice we used the MODIS standard snow-cover product to investigate where or not pixels were contaminated bye clouds The MODIS snow product is based on the Normalized Differenced Snow Index (NDSI), and other criteria tests that are described in e.g. Hall and Riggs (2002)and the Snow Product User Guide (Riggs et al., 2006).
Figure 4 shows two geocoded MODIS color composites from the May 28 and June 4, 2008 projected to UTM WGS-84 zone N33 on a 250 m x 250 m grid. Visual inspection of the scene from May 28 shows evidence of snow at from Lake Bygdin and Tyin, while Lake Gjende looks only partly snow covered. On the June 4 Lake Gjende looks ice free whereas Lake Bygdin and Tyin still have some remaining snow. Two classified (SCA) MODIS imagery was also inspected for the same dates indicating the same pattern as found in the RGB imagery’s.

**Comparison with SAR**
In figure 5 we show a comparison between the Radarsat (SAR), MODIS color composites (250m x 250m) and classified (SCA) MODIS imagery (500m x 500m) and MODIS reflectance imagery (500m x 500m). The correspondence is good between the MODIS color composites and the Radarsat images, but somewhat poorer with respect to classified (SCA) and reflectance MODIS imagery. Some overall feature can, however be recognized. The reason for the poor correspondence is probably mainly due to the large difference in resolution. Another source of error in the classified MODIS data might be caused by identifying cloud as snow. If the cloud is not identified as certain cloud in the cloud mask or, more commonly, by missing snow, possibly because thin, sparse snow cover was identified as cloud in the cloud mask (e.g. Hall and Riggs, 2007). The uncertainties caused by poor resolution have significant effect when mapping visually the present or absent of ice cover (snow cover on top of ice) on the relative small Norwegian lakes.

**6. Conclusion**
We have established a production line for near real time automated download of geocoded Radarsat images from KSAT and automated download and geocoding of Terra MODIS images (both reflectance and snow products).

For the relative small and narrow lakes in Norway MODIS data with a spatial resolution of 250m x 250m greatly improved the visually mapping of the presence or absence of ice cover (snow cover on top of ice) compared to the 500m x 500m resolution. We found further a 3 dB change in backscatter from a probably ice covered but snow free lake compared to an open lake in the same Radarsat imagery. The change in backscatter could presumably be used to map ice covered lakes. However more work is needed to develop algorithm and to verify the findings.

**Acknowledgement**
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**References**


Figure 1. Left: Map of Norway gray areas indicates lakes with an area greater than 100 km². Square indicate position of study area. Right: Maps of the study southern Jotunheimen. Dark blue denotes lakes > 100 km², whit denotes glacier, green denotes forest/vegetation, light green denotes bogs.

Figure 2. Map of snow depth in the study area (shown in fig. 1) on the June 4 2008. Data and graphics from www.seNorge.no.
Figure 3. Two Radarsat ScanSar (SCN16) FAR scenes acquired over study area southern Jotunheimen (for location see fig.1) the May 25 (left) and June 1 (right) 2008. Names of lakes are given in Fig. 1 and 4.
Figure 4. Upper left and right: Two MODIS color composites imagery acquired over southern Jotunheimen on the May 28 (upper left) and June 4 2008 (upper right). Clouds are shown as white and land areas with out snow are green, areas with thick snow covered are white, thinner snow cover are grey/pink/purple in the image. The lakes ice appears pink and open water is black. Lower left and right: The corresponding snow cover map based on MOD10_L2 (lower right and left). The outlines of the lake are marked with blue.
Figure 5. Same area shown by different sensors and imagery modes. Left upper: MODIS color composite imagery, June 4. Right upper: Radarsat imagery, June 1. Left lower: Classified snow MODIS imagery, June 4. Right lower: MODIS reflectance image, June 4. Classified snow covers in MODIS and reflectance imagery have 500 x 500 m resolution, results in far less accurate description. The outlines of the lake are marked with blue.
Analysis of a River Ice Cover using High-Resolution Satellite Data

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A variety of remote sensing techniques have been used to analyze river ice conditions and river ice jams including satellite imagery, aerial photography, and web cameras. Satellite imagery provides large scale images displaying the entire length of ice covered river reaches but often at poor spatial resolution. Recently, multiple image dates were acquired of an in-place ice cover on the Winooski River at Montpelier, Vermont, USA, by the QuickBird satellite. QuickBird is a high resolution commercial satellite which collects panchromatic (black & white) imagery at 60 cm resolution and multispectral imagery at 2.4 meter resolution. The images were collected on 09 March 2007 and 22 March 2007 during a period when the City of Montpelier was threatened by flooding resulting from the breakup of the ice cover and the formation of ice jams. This research seeks to examine the capability of using high resolution QuickBird imagery to classify river ice types and to determine ice cover characteristics. Multiple spatial and spectral analyses were performed to quantify the variation in ice conditions. Processing techniques included pansharpening each image (fusing the panchromatic band with the multispectral bands to create a high resolution multispectral image) and subsetting to the extent of the river channel banks and bridges. Texture and edge detection algorithms were used to characterize the spectral variation in rough ice conditions and classify the structural characteristics of river ice cover. Results show these techniques can be successfully used to identify variations in river ice cover characteristics and could provide an additional tool for monitoring river ice conditions in high hazard areas.
River ice, hydropower, and dam decommissioning
Interaction between ice conditions and hydro power regulations over 100 years. The Norwegian case.

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Water regulation for power production in Norway started in the late 1800’s. The 1900’s was the time of power development of the “large water falls”. It was soon recognized that the regulation of water could cause changes in the ice conditions and thus create ice problems both in the watercourse and at the power plant. Ice investigations were recognized as an important part in the planning of power projects, aiming to avoid potential conflicts connected to changes in ice conditions. Courts of evaluation were established to determine the extent of such problems, and thus provide a base for establishing compensation to people affected.

Today the major part of the hydro power potential has been developed. The period of constructing new large water power regulations has come to an end. The focus is now on developing in smaller rivers, and on increasing production in existing power plants.

In parallel with the ice investigations there has been a focus on fish. In recent years the mutual interaction between ice and biological life has been more and more focused, especially as regards fish, aiming to find the most advantageous scheme of regulation.

The climate and thus hydrology and ice condition varies greatly throughout the country and from year to year. Influence of variations in climate on the interaction between ice and power regulation has been evaluated. Prognoses for expected future climate changes are different in the north and the south, and the east and the west. Possible changes relevant to ice conditions are discussed.
1. Geography, topography, latitude and ice conditions

In Norway there are large variations in climate from south to north, and from the coast to inland, this because of the large variations in geography, topography and latitude of the country. There are large gradients in elevation from low coastland areas to high mountainous areas, narrow valleys with steep slopes, and wider valleys with more moderate slopes. The large gradients and seasonal variations in climate impact both runoff and ice conditions. The long coast is facing the North Sea and the Arctic Ocean. The Gulf stream sweeps the western coast from south to north, and contributes to a gradient from maritime to continental climate going inland from the coast. The Gulf stream also reduces the north–south gradient, and contributes to the existence of ice-free fjords along the coast. The lowest winter temperatures are found in the interior fairly close to the Swedish border in the southern part and in the interior of the Finmark plateau in the northern part.

Normally there are weather conditions promoting ice formation in rivers and lakes all over the country, except in a narrow rim along the south-western coast. There are, however, large regional variations.

The rivers vary in size and slope, many being of moderate size and fairly steep. In the valleys long, narrow and deep lakes, referred to as fjord lakes, are common. This differences in size and slope are important for the type of ice situation that will appear, and what kind of ice problems that will dominate. The duration of ice cover varies with the location and the size of rivers and lakes. Generally ice formation starts by November, first in the smaller rivers and most continental areas, and in the north. Ice release may be as late as May or June in the highest elevations in the south and in the north.

Ice covered rivers and lakes have a history of being safe and easy travelling routes. Regular winter traffic on ice is today mostly for leisure purposes. Knowledge of ice, its occurrence and quality, is of great importance. Ice observations have been collected for more than 100 years, but unfortunately these series are far from complete. Ice problems, such as ice runs and winter flooding are still well known phenomenon in many areas. The occurrence of these varies, however, from year to year, and depends on the winter climate.

This paper describes major results and experiences of ice investigations connected to water power regulations, and the history of adjusting the scheme of regulation to reduce ice problems.

2. The development of power production and legal framework

The development of hydro power in Norway started in the 1870’s, then with power production mainly for light purposes. The first municipal power supply system was established in the town of Hammerfest, on Norway’s arctic coast, and the town had electrical streetlights in 1890, said to be the first place in the world.

Gradually more power stations with intake reservoirs were built thereafter, with the real boom in hydro power development and hydro based industry to come after the turn of the century.
An extensive legal framework for utilizing hydro power was soon recognized and established. The impact on the affected local population, and their rightful share of the benefits of the hydro power development, had also to be taken care of, including compensation for negative influence on the environment.

Throughout the 20th century hydro power plants were built all over the country. A number of these, built in the period up to 1940, were for the time among the largest hydro power stations in the world. Another period of constructing large power plant systems was from the middle towards the end of the 20th century.

At present the major part of the larger power developments are over. The main focus is now on developing smaller plants located in smaller unregulated streams.

In 1991 a new market based energy law was introduced in Norway. The aim was to achieve more economic efficiency in electricity production by means of market forces. The new law introduced an open power market for the whole country. All consumers could now choose where to buy electricity independent of their homestead. This has, however, had some effect on the running of the individual plants. As there are short and long-term variations in electricity, prices and demand, this causes more frequent short-time variations in hydro power production, and thus water discharge, than before. This, of course, is unfavourable for the ice conditions, especially when the discharge is located on a river. The open market now includes a large part of Europe so that trade of electricity with other European countries have increased.

In Norway almost all, nearly 99%, of the electricity production is based on hydro power. Mean annual hydro power potential is calculated to ca 192 TWh.

3. Early ice investigations and experiences

It was early foreseen (around year 1900) that water regulation influenced the ice conditions in lakes and rivers, first that the length of the season and also the bearing capacity of the ice was reduced with increasing winter discharge. Observations of ice formation in turbulent water and formation of bottom ice were also performed.

Extensive ice-runs in the Glomma river in the southeast of the country in the late 1920’s initiated serious concerns about possible influence of increased winter discharge from the newly regulated lake Aursunden at the head of the river basin. To investigate this a governmental commission was appointed. A basic study of the heat exchange and ice formation was performed under the leadership of Olav Devik, then associate professor at The Norwegian Technical University. This initiated further studies of supercooled waterfilms, the existence of frazil ice and the formation of bottom ice.

Ice runs were also frequent in many other rivers these winters. The established commission concluded that the formation and breakdown of ice dams these winters were not so different in the regulated and not regulated comparable rivers. The consequences, however, were much more serious for the regulated rivers with increased winter flow. Problems connected to flooding and icing were more serious in the regulated rivers. Based on these findings stricter regulations were enforced on the discharge from the reservoir.
The studies of ice formation in turbulent water was continued, and fundamental work was performed and presented by Olav Devik in his doctorate in 1930: (Thermische und Dynamische bedingungen der Eisbildung in Wasserlaufen – auf Norwegische Verhaltnisse angewandt). Here he made a thorough analysis of the heat exchange between water, ice and air, well known today, and still a fundamental and very important work on this topic. He calculated mean parameter values, applicable to local conditions, and thus very useful in solving practical problems.

The existence of frazil and bottom ice has been known all places with ice in the rivers “as long as man can remember”. However, the scientific explanation for its formation had long been discussed. Again Devik made his mark. Devik was one of the first to realize the importance of super cooling of water and to measure it. He detected supercooled water films that had been forced underneath obstacles in the water, fundamental for the explanation of forming bottom ice and bottom ice dams. He based his measurements of super cooling on radiation from the surface and thus was able do measure temperature in very thin layers.

4. On the present situation concerning handling of ice investigations versus water power regulations, some actual problems

The majority of ice problems connected to water power development is caused by increased winter discharge. This was early realized, and observations and investigations of ice conditions were soon mandatory parts of the planning of power development. Ice investigations are also often performed as a part of the work of the water courts to decide on compensation for changes connected to the power development.

4.1 Stabilizing ice conditions to gradually increase winter discharge (Glomma)

As mentioned above the lake Aursunden in the very upper part of the Glomma watercourse was regulated to provide winter water to power stations in the lower part of the river, and taken into use in 1924. From the very beginning this increase in winter discharge caused ice problems. On the slow flowing areas the ice was no longer safe for travelling, and in the more swift stretches there were frequently ice-runs and jamming. To reduce ice runs and jamming, special restrictions have been laid on the winter discharge.

The vulnerable river stretch is being inspected regularly in the ice forming season. To avoid ice runs the discharge is kept low in the fall and early winter, and should not be increased before the river is stabilized and bottom ice dams are emptied.

4.2 Extreme frazil formation and flooding (Einunna)

In situations with extreme frazil production the storing volume for frazil may be too small. When the water velocity decreases the frazil will be stored along the river. When there is an icecover downstream the frazil producing area, the river often is being blocked by frazil so that adjacent area is flooded. A threatened area is Einunna in the higher mountain plateau in the central part of southern Norway. “Warm”water is released from a reservoir, along a fairly swift stream, onto a more slow flowing area with adjacent farmhouses. In extreme weather with low temperatures and snowfall.frazil is produced and the increase in water stage is very fast. In addition drifting snow into the open river contributes to the problems. An observation station with remote
observations of weather and hydrology is established at the site, to enable quick action if the water stage increases and the farmhouses are being threatened.

When this happens the discharge must be reduced quickly. Attempt has been done to make local adjustments to the river bed and the discharge to promote formation of bottom ice dams to increase ice cover, so far without satisfactory results.

4.3 Sudden unexpected increase in discharge (Orkla, Bardu, Glomma).

Downstream reservoirs, between intakes and outlet of power stations, the discharge in the river is normally small and limited to a restricted low water flow. Often there is a considerable volume of ice and snow accumulated in these river sections. A sudden outburst of water here might cause ice run and jamming. This may happen when the power station suddenly stops for some reason, and the water supply coming from upstream can not be stored in the intake reservoir.

This was the case last winter in the Orkla river. The intake reservoir is quite small and the water travelling time from the next reservoir upstream is ca 6 hours. The water overflowing the dam created an ice run that created a 3 km long jam. Fortunately there was no serious damage by the ice in this case. In this case the intake reservoir was full. The fallout of the power station was due to delivery problems for the power. As there is no by-pass arrangement in this station the overflow could not be avoided.

With extreme weather conditions the racks in the intakes in the reservoirs may be clogged by frazil. This has happened in Bardu river. The intake reservoir is most often ice covered, but was by this occasion broken up due to extreme wind conditions. The temperature was low, frazil ice clogged the rack, and the power station stopped. In this case it was possible to hold back the water from overflowing. In the river downstream the outlet of the power station the water diminished and the surface ice here stranded. A controlled and gentle increase in discharge, when the racks were cleared off, and the power station could run again, avoided further ice problems downstream.

From Høyegga dam in the Glomma river the water is diverted to the neighbouring valley. Downstream the dam there is a restricted fairly low winter discharge, and the river is ice covered in winter. The power station stopped and the water overflowed the dam. It was possible only for a very short time to reduce the overflow. Part of the overflowing water froze on top of the existing ice cover and the ice thickness increased considerably. This happened in January and the power station was out of function throughout the spring flood. This was a vulnerable situation as ice runs in this area was quite common before the diversion. Measures to reduce the ice thickness were discussed. Fortunately the weather conditions were very favourable for the situation and the ice melted off gently.

In all the cases mentioned there has been only one turbine in the power station. In the last example a second turbine would have solved the problem. That is not the case when the stop is caused by intake or delivery problems, as in the other examples mentioned. There is, however, many good reasons to install at least 2 turbines in the power stations.
4.4 Ice conditions in reservoirs
In Norway there are ca 900 reservoirs and lakes were the ice is influenced by hydro power and water regulations. Increased flow through lakes will create larger open areas at inlets and outlets. The increased throughflow will increase the areas of weaker ice in sounds, and new sounds may occur as the water level drops, and thus also be subjected to weakening of the ice. New open areas and areas with weakened ice will also occur by inlets and outlets of tunnels and power stations. The deeper parts of the reservoirs will only have minor changes in ice thickness.

When there is a large change in water level throughout the winter season, a rim of crevassed ice will ground along the shore, especially on steep and rough areas. Short time variations in water level may give overall unsafe ice

When the ice has sufficient strength ice covered lakes are relatively safe and easy winter roads. Leisure use of ice has been more and more popular, both by local people and tourists. Knowledge of the ice conditions are of great importance for safety reasons. Information of weakened ice caused by water power development is therefore given on the internet.

4.5 Selective withdrawal to reduce effects on water temperature and ice coverage, to reduce effects on salmon (Alta)
The Alta water power project is one of the most controversial water power projects in Norway. The planning started in the 1970’s and the power plant was set into production in 1987.

Alta river is one of the richest salmon rivers in Norway, and there were serious concerns how the power plant would influence ice conditions and fish habitat in the river. Extensive investigations of possible environmental impacts were therefore performed during the planning stage. Before regulation the river was ice covered except for limited leads. As expected the ice cover was nearly absent several km downstream the outlet, and the water temperature was somewhat lower in summer and slightly higher in fall and winter. The investigations on water temperature, ice conditions and fish population have continued after commissioning to document changes caused by the power development. The amount of fish near the power station outlet was reduced. The biologists claimed that this partly was caused by less shelter due to the reduced ice cover. One was therefore looking for compensating measures.

There is an upper (near surface) and a lower (near bottom) intake to the power plant. The upper intake was built to reduce the water temperature changes during summer. It was investigated to use the upper intake in winter to reduce temperature in the discharged water. By doing this the discharge water has been somewhat colder, and part of the upper reach of the river has been ice covered in cold periods. This has proved to be sufficient to be advantageous for the fish habitat.

The upper intake can, however, only be used a limited time. When the water level in the reservoir is reduced 15 m the lower intake must be used to avoid air induction. Then the temperature increases 1 - 2 °C in a short time, due to the configuration of the reservoir, and the ice cover will melt in this upper reach. To extend the period of potential for ice formation the release from the reservoir is kept fairly low. This lower discharge is favourable for ice formation in the whole river, also further downstream. A reasonable time for change of intake is now found...
to be early April. At the corresponding water stage, limited by the location of the upper intake, only 25% of the reservoir is utilized.

To utilize the reservoir storage below the upper intake before spring flood the power plant discharge must be increased. This means that the spring break-up of the river ice must be started artificially. A challenge here has been to find a way to do this without causing ice problems in the 40 km ice covered river downstream. A method has been tested, and can be executed if a close evaluation of the hydrological conditions in late winter and spring is undertaken.

When opening the lower intake the warmer water (1-2 °C) accumulated in the lower part of the reservoir will be drawn off first and thereafter the water temperature will gradually decrease to the previous level after 10-15 days.

By keeping the discharge constant on the winter level (20-25 m³/s) the first 4-6 days after the change the discharge may be increased to ca 45 m³/s, which is half of maximum discharge, in another week or so. This may cause some water on the ice in certain areas, and leads will open up and gradually extend so that the discharge may be increased further when that is needed. There are also restrictions on the timing of discharge increase, related to fish habitat considerations. The requirement being that the discharge should not be decreased again, once increased.

5. Increased winter freshwater flow to fjords

The increased winter flow created by hydro power production may accumulate as a freshwater layer on top of the more salty fjord. This gives a high potential of increased ice formation in the fjords, and has been studied closely in connection with the planning and construction of larger hydro power developments.

Increased ice formation has through the years been giving problems especially for boat transport and fishing. One measures to reduce or avoid the increased ice formation has been to built a submerged outlet from the power station. This is only possible to do in connection with the planning of the development and when the power plant is discharging directly to the fjord. Another somewhat more flexible measure is to arrange for mixing of the salt and fresh water in the fjord by an air bubbling arrangement.

6. Influence of climatic changes

Future climate in Norway, is expected to give a warmer winter climate all over the country. The major impacts on the ice cover in rivers and lakes are:

- Only a few days shorter ice period in the most continental part of the country.
- The ice period decreases more towards the coast.
- Larger year to year differences.
- More iceruns which may jam at new places.
• Increased area along the coast with seldom ice.
• Longer stretch free of ice downstream large lakes.
• The lake ice will be thinner in the maritime regime, but less change in the continental regime.

The length of the ice season is particularly sensitive to length of time with freezing temperatures and amount of snow fall. Mild spells and heavy rainfall in the winter can trigger ice jamming and ice runs when ice cover has developed. The downscaled scenarios of global warming indicate that the projected changes in temperature will differ regionally in Norway. More unstable ice conditions are expected along the coast and maritime areas from the south and as far north as to the arctic circle. The effect on ice will be somewhat different on rivers than on lakes.

The areas where the ice runs and jamming starts will shift to higher altitudes, moving the maritime ice cover further inland. This can cause a possibility of damages at other settlements than those suffering from ice runs in the past. The season with the risk of ice runs will be shorter, but the year to year variability will be high. Extreme winter rainfall events after the ice has formed can cause severe ice runs.
Simulation of hanging dams downstream of Ossauskoski power plant

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Frazil ice and hanging dams formation in rivers may cause harm for hydropower production and the environment. This is the case especially in regulated rivers in which winter discharges are higher than the natural discharges. Kemijoki River is the longest river in Finland and its basin covers a vast area of Northern Finland. The river is harnessed for hydropower production with 16 separate power plants, mostly owned by Kemijoki Oy company.

This article focuses on Ossauskoski power plant. Its capacity is 93 MW and annual energy output on 2006 was 415 GWh. The present design flow is 750 m\textsuperscript{3}/s and an increase of 330 m\textsuperscript{3}/s is planned resulting 31 MW increase of capacity. The reach below the power plant has however encountered frazil ice problems because of open water areas and hanging dams. A greater discharge will probably change ice conditions downstream of the power plant.

To study the river ice phenomena, an extensive data survey program downstream of the Ossauskoski power plant was conducted 2005-2007 by Kemijoki Oy. In addition to standard weather observations and hydrological statistics from the power plant, several survey flights were flown during the winter 2006-2007. This produced a series of aerial photographs pointing out some critical open water areas. A ground penetrating radar and manual survey were used to gather data on the thickness of solid ice cover, snow and frazil slush. The river geometry was obtained with AquaticSonar Swathe Surveyor equipment.

The JJT-model was used by Finnish Environment Institute to study the effect of hanging dams on water levels. Also an effect of modified river geometry was studied in the calculations. A description of the ice observations and the model simulation results downstream of Ossauskoski power plant is given in this paper.
1. Introduction
Kemijoki River is the longest river in Finland and its basin covers a vast area of Northern Finland. The river is harnessed for hydropower production with 16 separate power plants, mostly owned by Kemijoki Oy company. Ossauskoski hydro power plant (Fig. 1), located 70 km south from the city of Rovaniemi, has got a capacity of 93 MW and annual energy output of 415 GWh (year 2006). After a major renovation and upgrade of all three machinery in 2008, the new capacity will be 124 MW and annual energy output 501 GWh. This will make Ossauskoski the sixth largest hydro power plant in Finland.

Figure 1. Location of the study area.

The increase in power output and discharge may, however, cause changes in the ice conditions downstream of the power plant. The reach has already suffered from frazil ice problems due to thaw regions in the ice cover. Frazil ice and hanging dam formation in rivers is known to cause harm for both hydropower production and environment.

Hence, an R & D project was put together between Finnish Environment Institute (SYKE) and Kemijoki Oy company. The objective of the project was to study a dredging plan suggested by Kemijoki Oy with a modern numerical river ice model. The modeling work was done in SYKE. The project included also a vast observation work by Kemijoki Oy to map the river bed topography and to identify hanging dam locations and extents. Sounding device and ground penetrating radar were used. Several flights were also done above the study area to find the thaw regions in the ice cover.

The model used in this study was a numerical river ice model, JJT (Huokuna 1990), found to be useful in Finnish river ice studies. The original aim was to use the CRISSP2D model (CEATI 2005), but unfortunately the simulation results were not yet available. CRISSP2D testing will be continued at SYKE.
2. Observations

The research consisted four kind of observations: 1) River bed survey, 2) Frazil ice survey, 3) Aerial ice cover study and 4) Hydrological and weather observations.

River bed survey was performed throughout the 19.3 km study reach with AquaticSonar Swathe Surveyor equipment, especially designed for shallow water survey applications. The system consists of interferometric wide angle survey system operating at a frequency of 156 kHz. For accurate positioning and navigation the survey system was connected to RTK-GPS, motion sensor and electronic compass. Sounding data was post-processed and delivered in a 1x1 meter grid. The survey system is a product of Kemijoki Aquatic Technology Oy (KAT) company. An example of the river bed survey results is presented in Fig. 2.

![River bed survey results presented in a 1x1 meter grid.](image)

Frazil ice survey was performed three times in total during the winter 2005-2006 and 2006-2007. On each time five to eight survey lines were observed (Fig. 1). The survey was done with a ground Penetrating Radar (GPR) SIR-2 and SIR 3000 connected with 100 MHz antenna. A GPS equipment was used to locate the desired survey lines. Depending on ice cover condition, the GPR was mounted beneath a hovercraft (Fig. 3) or dragged by a snow mobile.

Frazil ice under the solid ice cover was interpreted by post-processing software and the results showed the cross-section of the river bed with the ice profile. The distance \(d\) from the radar sensor to the reflective medium can be calculated with the electromagnetic wave velocity, \(v\), and the running time, \(t\), of the signal (Eq. 1). The wave velocity, \(v\), can be calculated with the speed of light, \(c\), and relative dielectricity of the medium, \(K\), as showed in Eq. 2 (Healy et al. 2007).

\[
d = vt \tag{1}
\]

\[
v = \frac{c}{\sqrt{K}} \tag{2}
\]
Dielectricity coefficient $K$ used in this study was 75, which is found adequate for frazil ice evaluation done from the GPR profile pictures (Sirniö 2006). Separate dielectricity coefficients were used on different media. An example of frazil ice survey analysis is presented in Fig. 4.

![Ground penetrating radar survey performed with a hovercraft.](image1)

**Figure 3.** Ground penetrating radar survey performed with a hovercraft. Photograph: Kemijoki Oy.

![Frazil ice profile based on ground penetrating radar interpretation.](image2)

**Figure 4.** Frazil ice profile based on ground penetrating radar interpretation (Sirniö 2006).

Aerial ice cover study was performed during six separate flights between December 2006 and April 2007. The aim of the flights was to identify thaw regions in the study reach contributing in frazil ice production. These regions were also supporting the validation process of the numerical river ice model together with the meteorological and hydrological data.

Meteorological and hydrologic observation were performed specifically for this study as well as gathered from permanent observations. Meteorological variables consisted of air temperature (Fig 5), precipitation, cloudiness, wind magnitude and direction, dew point and relative
humidity. Hydrological observations, as discharge (Fig. 6) and water elevation just downstream Ossauskoski power plant were from databases of Kemijoki Oy. The company does also have a permanent water elevation sensor next to Tervola bridge. Three temporary pressure sensors for water elevation observations were installed along the study reach. Water elevation observation points relevant for the study are presented in Figure 1.

Figure 5 Air temperature observations for the winter 2005-2006

Figure 6 Discharge at Ossauskoski power plant during the winter 2005-2006.

3. Numerical model (JJT)
The numerical river ice model JJT (Huokuna 1990) is a combination of one-dimensional hydraulic model for unsteady flow and thermal and ice model. The thermal and ice model include heat exchange calculation, distribution of water temperature along the reach, border ice formation, formation and erosion of hanging dams, ice cover friction factor simulation and thermal growth and decay of ice cover. The model was verified with comprehensive observations such as weather data (air temperature, wind velocity and direction, relative humidity and
cloudiness) and discharge, water elevation and temperature, ice coverage and ice cover growth and decrease. The observations were gathered during three winter on four separate river reaches. Further model description and model verification results are reported by Huokuna (1988, 1990). A schematic diagram of model components is presented in Fig. 7.

**Figure 7. JJT-model components (Huokuna 1990).**

The Ossauskoski-Tervola numerical model consists of the main river reach and three reaches around the islands. The length of the modeled area along the main river is 19.1 km and it is described by 364 cross-sections derived from the river bed survey results. The total number of cross-sections is 421. That means that the distance between the cross-sections is normally very short (50m to 100m). A short distance between the cross sections was used because of the available detailed bottom topography data and because very small scale excavations had to be modeled in the study.

There is a strong short term regulation of the discharge in the Kemijoki River. The discharge at the Ossauskoski power plant may vary during a day from 100 m$^3$/s to 800 m$^3$/s. The fluctuation of water levels and discharge is possible to take very accurately in to the account in the hydraulic model. Simulation of the ice cover formation under fluctuating flow conditions is however not possible with the JJT-model since the constant formation and breakup of thin ice cover can not be modeled correctly. On the grounds of past studies it is observed that satisfactory results for ice cover formation may be achieved by using a 24 hours moving average for upstream boundary conditions. Hence, actually two hydraulic simulations are performed during each time step of the computer run. One hydraulic calculation is done by using 24 hours moving average for the upstream discharge. Hydraulic parameters like water level and flow velocity achieved from that simulation are then used in the thermal simulation and the simulation of ice cover formation. Another flow simulation is based on hourly upstream discharge values and these results are used to produce the "real" water level values.
4. Simulation results

The open water calibration of the hydraulic model was done using observed values for the periods from November 1\textsuperscript{st} to November 21\textsuperscript{st} 2005 and October 10\textsuperscript{th} to October 30\textsuperscript{th} 2006. The observed and the calculated downstream water level at the Ossauskoski power plant for the first calibration period are presented in the fig 8. The downstream boundary condition at Tervola was set as a constant water level of 26.75 m. According to the calibration Manning-n values varied between n=0.028 and n=0.035.

![Water level graph]

**Figure 8.** Open water calibration of the model. Observed and calculated downstream water level at Ossauskoski power plant between November 1\textsuperscript{st} and November 21\textsuperscript{st} 2005.

The flow and ice simulation for the winter 2005-2006 was made from November 22\textsuperscript{th} 2005 to March 31\textsuperscript{st} 2006. Because of the rainy autumn and the late beginning of the winter the discharges were high. In the study reach the ice cover formation started in the beginning of December. The Kemijoki river is relatively deep and the flow velocity is low downstream of the model Tervola, the downstream boundary of the model. The ice cover formation begins first in that section. In the model a fictive ice boom was put on the downstream end of the model to initiate the ice cover formation. A fictive boom was also set 13.5 km downstream from Ossauskoski power plant. An ice bridge is know to be formed at that location but it can not be modeled with the JJT-model. The locations of the fictive ice booms are presented in the longitudinal profile in Fig 9. Calculated ice cover and hanging dam formation on December 16\textsuperscript{th} 2005 is also presented in the same Figure.
The downstream water level of the Ossauskoski Power plant started to rise because of the formation of the ice cover and hanging dams in the river. In the December and January the downstream water level at Ossauskoski Power plant is about 1.2 m higher than open water level. Later on the winter the water level begins to lower in spite of the fact that ice cover is becoming thicker. This is due to slowly diminishing hanging dams. The observed and the calculated downstream water level at Ossauskoski power plant for the winter 2005-2006 is presented in the Fig 10. The calculated open water level is also presented in the same figure. That level is calculated with JJT-model without taking the effect of ice cover and hanging dams on water level into account.

The calculated ice cover and hanging dams in February 9th are presented in the Fig 11. On that day a survey of the frazil accumulations was made. The locations of the observed hanging dams are also presented in the Fig 11. The observed values present the maximum values of the frazil thickness at those locations. The location of the hanging dam at the kilometer 9 is relatively well simulated with the model.
Figure 10. The observed and the calculated downstream water level at Ossauskoski power plant for the winter 2005-2006. For the comparison the calculated open water level is also presented in the same figure.

Figure 11. The calculated ice cover and hanging dams for February 9\textsuperscript{th} 2006. The results of the frazil survey are presented in the same figure. The observed thickness of the hanging dam is the maximum thickness in the cross section.
5. Discussion and conclusions
To study the river ice phenomena, an extensive data survey program downstream of the Ossauskoski power plant was conducted 2005-2007 by Kemijoki Oy. The JJT-model was used by Finnish Environment Institute to study the effect of hanging dams on water levels. The model results shows that a numerical model can be a feasible tool to study the effect of hanging dams on water levels.

The ice bridging phenomena can not be modeled in a reliable way by the JJT-model. For that reason the CRISSP2D model is meant to be used for this river reach. The 2D-results were not available as this paper was prepared.

In addition to the results presented in this paper same calculations have been made with the modified geometry. These modeling results will be a basis for further actions in the Kemijoki river downstream Ossauskoski power plant. The company will analyze the gains in power production and if worthwhile, proceed to more specific ground survey of the river topography for the dredging plan. In the final stage, a feasibility analysis will be carried out to make the final decision whether to fulfill the dredging or not.

It is assumed that discharges and adverse effects of frazil ice phenomena are going to increase due to climate change. Though climate change impacts were not studied in this project, they must be counted to some extent in the feasibility analysis.

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References


Evaluating the Impact of Dam Removal on Break-up Jams

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Many dams across the United States are being decommissioned for stream restoration or as a result of structural deficiencies. On northern rivers, there are a number of examples of dam removal increasing the frequency and severity of damaging floods due to jams. Dam removal can change the location where breakup ice jams occur or initiate the formation of freeze-up jams. This paper presents an overview, based on recent studies, of the procedures used to evaluate the impact of dam removal on breakup ice jams. The first step is to understand breakup ice jam formation under existing conditions by reviewing the historical record of known jams. The historical ice events may provide data on the jam locations, the reach length of contributing ice, and the resultant stages and damages. In many cases, no historical record of jams prior to construction exists and the selection of post dam removal ice jam scenarios are based on an analysis of the river geomorphology, ice jam mechanics, and current data. The ice conditions then are characterized in terms of the meteorological and hydrologic data which provides an estimate of the range of parent ice thicknesses and discharges at which jams occur. Finally, the study reach is analyzed under these conditions with and without the dam in place to determine whether the dam plays a role in ice processes. In many cases, a hydraulic model is used to quantitatively estimate the stages that result from jams following dam removal. When insufficient data or project constraints do not allow for a model study, an analysis of the pre- and post-dam removal conditions is performed. The result is an estimation of the potential locations and severity of ice jams following dam removal.
1. Introduction

Over the past three centuries more than 2.5 million dams were built in the United States to meet the power, water supply, flood control, and recreational needs of a variety of users (National Research Council 1992). These dams range in size from small farm pond dams less than two meters (6 ft) in height to the 770-ft-tall Oroville Dam completed in 1968. Despite the large number of recently built dams, Doyle et al. (2003) noted that the period 1950–1970 could be termed the “golden age of dam building,” with tens of thousands of dams built each decade. The Federal Emergency Management Agency (FEMA) (2001) estimated that by 2020, 85% of large dams would be at or nearing their design life spans. The American Society of Civil Engineers (ASCE) report card gave dams a grade of D in 2005, primarily due to increasing numbers of unsafe dams (ASCE 2005). The grade is based on the following observations:

- Between 1998 and 2005, the number of unsafe dams rose by 33%.
- Because of constrained budgets, the number of unsafe dams is increasing faster than those being repaired.
- The combination of rapid downstream development and inadequate past design practices, coupled with a predicted increase in extreme events is leading to life safety risks.

Increased awareness of the ecological, recreational, and economic issues, in addition to safety issues associated with dams, has led to reevaluation of their continuing usefulness (American Rivers et al. 1999, American Institute of Biological Sciences 2002, Heinz Center 2002). Interest in dam decommissioning, including dam removal, has grown substantially over the past 20 years. Decommissioning alternatives include dam removal, which is often assumed to be synonymous with decommissioning; the use of nature-like fishways to bypass a dam (e.g., the USACE’s New Savannah Bluff Lock and Dam By-Pass); the use of rock arch ramps or boulder vanes (e.g., USACE projects on the Red River of the North at Fargo and Grand Forks); partial breaching (e.g., USACE projects on the Chattahoochee River); and modifying dam operations in an effort to reproduce pre-dam flow conditions, which the USACE is pursuing at several locations with The Nature Conservancy.

Dam decommissioning is a non-trivial issue that requires scientific, sociological, and economic analyses. Typical dam removal studies address open-water impacts of dam removal. However, as White and Moore (2002) point out, dam removals on northern rivers can also significantly affect the ice formation, growth, and breakup processes. There are several examples of dam removals resulting in changed ice conditions that increased the frequency and severity of damaging floods (Tuthill and White 1997, White and Moore 2002, Vuyovich and White 2006, Tuthill et al. 2007). Dams alter the natural ice regime by blocking or hindering the downstream transport of ice. Once a dam is removed the ice can be transported farther downstream until it reaches some other obstruction; such as a sharp bend, man-made infrastructure or a thick ice cover formed over a slower moving reach. In cases where a dam has been in place for many years, development may have occurred along the riverbanks in downstream reaches. This development may be susceptible to increased ice damage once the dam is removed. Ice jam mitigation techniques are available; however, the time to prepare and design these techniques to address these unintended consequences is early in the dam decommissioning evaluation. This paper discusses the method used to evaluate ice impacts of dam decommissioning and provides case studies at three locations.
2. Analysis Method

Several studies have addressed the need to carefully examine the environmental, social and economic effects of dam removal (ASCE 1997, Heinz Center 2002, Conyngham et al 2006). Ice issues are often over-simplified or not addressed. White (2001) highlighted the importance of sediment and ice transport issues in a dam decommission study and suggested steps to assess the impacts of ice. A similar approach has been used in standard investigations of ice jam occurrence (Tuthill et al. 2003, Vuyovich et al. 2005). The suggested steps to evaluate the impact of dam removal on ice processes include:

- Compile and review historical ice events;
- Compile meteorological and hydrologic data from stations along the reach;
- Characterize the ice conditions of the study reach by evaluating the meteorological and hydrologic data at the time of each ice jam event; and
- Analyze the ice jam conditions with and without the dam in place to determine what role the dam plays in similar events.

These steps are described in more detail in the following sections. The first three steps are fairly straightforward. The fourth step is highly dependant on the data available and the scope of the study. Studies can range from preliminary investigations to determine if ice should be considered in later phases of the dam removal study, to detailed hydraulic models of the study reach with and without the dam in place. The method used to determine the role of the dam in ice processes will depend largely on what is required. The case studies in this paper use three different methods of analyzing the ice impacts of dam removal.

2.1 Historical Ice Event Data

Historic ice jam events were reviewed for information on where jams occur, resulting stage increases, estimates of ice thickness, damages, and relevant conditions at the time of the event. Ice jams are highly localized and can be triggered by such factors as changes in river slope or configuration, bridges and other structures obstructing flow, or upstream jam releases. Temperature and discharge also contribute to ice jam formation, all of which makes jams difficult to predict without prior observations. Often historical ice event data are not readily available or reported. One reason for the underreporting of ice events involves perception stage (Gerard and Karpuk 1979), which is defined as the minimum stage at which a source will perceive an event. If an ice jam occurs but does not exceed the perception stage, most observers do not report the event. Fortunately, in a dam removal study, the most significant and damaging ice jam events are the most important in assessing the impact that removing the dam will have.

The Cold Regions Research and Engineering Laboratory (CRREL) Ice Jam Database (IJDB 2007) holds over 16,000 records of ice events throughout the U.S. and is a good source of information for ice analyses. Other sources include project reports, trip reports, local newspaper accounts and historical records.

2.2 Meteorological and Hydrologic Data

Temperature data are used to estimate ice thickness based on accumulated freezing degree days (AFDD) (White 2004). In this method, thermally induced (but not frazil) ice thickness can be estimated on a given date during the winter based on average daily temperature using the
modified Stefan equation (USACE 2002). Although this method provides a reasonable estimate of ice growth caused by thermal processes, it is important to note that the ice thickness may be underestimated because of other factors, such as water velocity and the presence of a snow cover on top of the ice. Also, frazil ice deposition can contribute to ice thickness. Air temperature data is also used to identify the temperature patterns leading up to an ice event and over the course of the winter. Precipitation data are helpful in detecting the types of rainfall events that lead to increased discharge and ice break-ups. In the United States, daily maximum and minimum air temperature data and daily precipitation data for National Weather Service (NWS) meteorological stations is available from National Climatic Data Center (NCDC 2007).

Discharge data are used to identify the effects of various flow rates, and particularly the change in flow rate, on ice processes. Sufficient flow is required to break up a solid ice cover and transport it downstream. However, a steady rise in flow over many days or weeks will often result in a weaker ice cover as the water scours the underside, and it usually will not cause a major event. Significant events are generally a result of a rapid rise in discharge, which breaks up a strong ice cover and jams downstream. The average daily discharge is typically used, and is available for most major streams in the U.S. from the United States Geological Survey (USGS, 2007).

2.3 Characterizing Local Ice Regime

The formation of breakup ice jams is strongly influenced by the winter weather conditions, the river discharge, and the river’s geomorphology. The influence of the winter weather can be understood by examining the winter air temperatures. A strong relationship exists between the thickness of thermally grown ice and the number of accumulated freezing degree days that occur during the winter. Mechanical breakup occurs when there is sufficient flow to break up the solid ice cover and transport it downstream. Breakup jams often occur during warming periods that cause the ice cover to deteriorate to some degree, but warm temperatures without an increase in discharge generally lead to a thermal melt-out of the ice (USACE 2002). Ice jams occur at locations with limited ice conveyance, such as at sharp bends or channel constrictions, at bridges and other structures obstructing flow, or where the river slope decreases. All of these factors combined make predicting jams difficult without prior observations.

With sufficient data, though, much can be learned about the ice regime of a particular reach. By analyzing the temperature and discharge data and calculated ice thickness on the date of known ice events, the conditions leading to ice events can be understood, such as the average discharge, the rise in discharge in the days preceding the event, the average ice thickness and the average temperature and change in temperature in the days preceding the event. Figure 1 shows a typical plot of the discharge, AFDD and meteorological data used to evaluate the data. This information can be used to “predict” damaging ice events by analyzing the data for other time periods.
For a dam removal study, information on ice events prior to construction of the dam is especially important. If available, data during earlier events will be used to analyze data taken once the dam was in place to determine if significant events that would have occurred in the past are being mitigated by the presence of the dam, or whether no change has occurred. Often dams have been in place longer than local recording stations. For ice events that occurred while the dam was in place, it is important to note whether the ice jammed behind the dam and whether the dam held back the ice run, even temporarily.

2.4 Analyzing the impact of a dam on the ice regime

The method used to analyze the ice processes with and without the dam in place depends on the data available and the scope of the current study. If no data exists prior to construction of the dam, then a hydraulic model may be the only way to compare with and without dam scenarios. If data does exist prior to construction then a “hind-casting” (Tuthill et al. 1996) method could be used in which conditions leading to major ice jams before the dam was built are used to predict major events after the dam was built. In preliminary studies designed to determine if further analysis of ice is necessary, historical records may be sufficient to say with some certainty whether the dam plays a role in altering the ice process. The following three case studies demonstrate different methods used to analyze the impacts of dams on ice processes.

3. Case Studies

3.1 Lancaster, NH

The Israel River in Lancaster, NH has experienced numerous damaging ice jams. The river rises in the White Mountains and flows through upland meadows before reaching the relatively flat
backwater of the Connecticut River just below the town of Lancaster. The average channel slope for the basin is 0.03, though the average slope in the reach through Lancaster is about 0.0083 (Provan and Lorber, Inc. 2003). Where the Israel River flows into the Connecticut River, the average slope is about 0.0001 (Figure 2). During many winters, ice from the upper Israel River breaks up and flows downstream to form a jam in this slow-moving backwater reach.

![Figure 2. Israel River Profile](image.png)

In 1981, the New England Division of the U.S. Army Corps of Engineers completed an ice control structure (ICS) located about 0.5 miles upstream from the center of town. The 9-ft-high concrete-capped gabion weir was designed to retain both frazil ice during freezeup and broken ice after ice cover breakup thereby reducing the amount of frazil which accumulates in the backwater of the Connecticut River, and the volume of the subsequent breakup ice jams (USACE 1964, USACE 1974). These downstream frazil deposits resist the passage of the breakup ice run and are believed to be the major cause of the historic ice jam flooding problem at Lancaster. Relatively few ice jam flood events are reported in Lancaster prior to 1950, when the last of 4 mill dams had finally breached. It is thought that two of the dams, located upstream of the village, intercepted frazil ice, reducing its deposition in the backwater reach below the village. It is also likely that the upstream dams retained the breakup ice run to some degree, reducing the severity of downstream ice jams.

Since construction of the ICS, the frequency of ice jam events in Lancaster has not decreased, while the severity of ice jam flooding has. A review of historical ice jam events and estimated discharge and temperature data show that winter conditions at the time of ice breakup have not moderated since 1981. Fifteen ice jam events were recorded prior to construction of the ICS in 1981. These include two major events that occurred in the late 1800s that destroyed the Main Street Bridge—a covered bridge in 1886 and an iron bridge in 1895. Since construction of the ice control structure, 23 ice jams have been recorded in Lancaster. Sixteen of the 23 jams did not cause any flooding. Four caused flooding in the police station, which was located in the basement of the town hall. Three events—March 1992, February 1996, and January 1997 (the latter two freezeup jams)—caused minor road flooding along Canal Street. There may be temporal bias because ice conditions were monitored more closely after the ICS was built than
they had been before it was built. Field monitoring has shown that the ICS does provide some degree of frazil ice retention as well as ice retention during the breakup period (Figure 3).

![Image](https://via.placeholder.com/150)

**Figure 3.** Ice cover retained upstream of the ICS during 24 Mar 2003 event

For this study, the threshold criteria were developed for the period prior to the construction of the ICS. The use of threshold criteria to predict breakup ice jams is one of a number of breakup ice jam prediction methods (White 2003). The threshold criteria were then applied to the entire data set to “predict” breakup ice jams that would have been potentially significant in the post-ICS time period. Threshold criteria characterizing severe pre-ICS breakup ice jam events were developed based on air temperature and discharge data. The criteria were then used to estimate the likelihood of severe ice jam flooding in the post construction period, assuming the ICS had not been built.

Criteria were developed based on the five jams with the highest recorded stages (1968, 1950, 1970, 1973, and 1977) and applied to post-ICS data. The physical variables used in the threshold criteria were the estimated ice thickness at the time of the event, the estimated discharge, and the change in temperature and discharge in the days leading up to the event. The following severe breakup ice jam initiation criteria were established for the Israel River at Lancaster:

- Ice thickness greater than 17 in. at the time of breakup;
- Discharge of at least 1700 cfs more than the annual base flow at the time of the event;
- Flow increasing at the time of breakup, but not for more than three days prior to breakup (i.e., rapid rise in discharge);
- Temperatures that increased in the few days just prior to the jam but were not above freezing for an entire 10 days leading up to the jam (melt out); and
- No ice breakup 30 days prior to the jam (i.e., no discharge greater than 1000 cfs).

These criteria were applied to data for the entire period of record to hind-cast severe breakup ice jams during the period before the ICS was built and to predict severe breakup ice jams in the period after the ICS was built. In this case, severe ice jam events are those that cause damaging floods. Since the construction of the ICS, no damaging floods have occurred in Lancaster, so any predicted in the analysis were assumed to have been prevented by the structure. The hind-casting analysis predicted six severe jams while the dam was in place. An actual ice event was recorded
for four of the predicted ice events at Lancaster (1989, 1990, 1992, and 1996), though none caused significant flooding. The analysis shows that the same type of conditions that caused significant ice jams before the structure was built have occurred since construction without any serious ice jam flooding.

The results of this analysis indicated that the Israel River ICS does provide some flood damage reduction benefit to the Town of Lancaster (Vuyovich and White 2006). Further analysis of the study reach and the structure was recommended to provide more quantitative information on the flood damage reduction benefits provided by the ICS.

3.2 Merrimack, NH

The Merrimack Village Dam is located at Merrimack, NH on the Souhegan River approximately 2,000 feet upstream of the confluence with the Merrimack River. The potential removal of the dam is currently being investigated. Ice forms on the Souhegan River nearly every winter and an important consideration is determining the impact of the removal of the Merrimack Village Dam on the Souhegan River ice conditions. Of specific concern were the ice impacts on the historic Chamberlain Bridge, located approximately 130 feet downstream of the Merrimack Village dam.

Figure 3. Merrimack Village Dam, looking upstream, taken from Chamberlain Bridge

The Souhegan River is approximately 34 miles long with a drainage area of roughly 220 mi². The profile is shown in Figure 4. It is likely that the dams upstream reduce the amount of ice that travels to lower portions of the river. The McClane Dam, in Milford, New Hampshire, lies approximately at the midpoint of the river, below the Souheган’s steep upper half. Ice jams have been reported at the McClane Dam (IJDB 2007). Below the McClane Dam, at Milford, the Souhegan River flattens out significantly for 12 miles before reaching a series of rapids called Wildcat Falls in Merrimack, New Hampshire. Approximately one mile upstream of the rapids, the river forms a sharp oxbow, which is the location of several reported ice jams. It is likely that a solid ice cover at the impoundment stops ice released from the oxbow without causing any damage, until the discharge is sufficient to carry the ice over the dam or the ice accumulation melts in place.
For this study, the USACE Hydrologic Engineering Center’s River Analysis System (HEC-RAS 2006) was used to model the Souhegan River. The model was geo-referenced and used to estimate the ice jam thickness and resulting water surface profiles with and without an ice jam in place for both the pre- and post-dam-removal conditions. A number of material properties of ice need to be determined to model an ice jam in HEC-RAS (e.g., White 1999), including ice thickness, ice volume, specific gravity of ice, angle of internal friction, porosity of ice accumulation, ratio of lateral to longitudinal ice stresses, Manning’s n value of ice and maximum under-ice velocity. Ice thickness was estimated using the AFDD method. The ice volume was estimated using a USGS topographic map of the area and assuming that all of the ice downstream of the McClane Dam would contribute to an ice jam. If the maximum under-ice velocity parameter is set too low within this reach, the thickness of the jam will be artificially reduced. The maximum under-ice velocity was set to 15 ft/s within the ice jam extent to allow the ice jam to progress up the steep section from the Merrimack backwater past the Chamberlain Bridge (Vuyovich and White 2007).

The two-year and the ten-year open water flood flows, based on the FEMA Flood Insurance Study for the Town of Merrimack, were used to model the range of maximum ice jam discharges (FEMA 1979). At the location of the ice jam, the two-year discharge is 3,200 cfs and the ten-year discharge is 8,370 cfs. Based on the review of hydrologic data and historical ice events, the actual maximum discharge an ice jam could withstand on the Souhegan River is estimated to be between these two discharges.

Based on this analysis, the most likely location for a break-up ice run to jam once the dam is removed is where it meets the backwater of the Merrimack River, approximately 1,000 feet upstream from the mouth of the Souhegan River at the location of a large sediment island. This analysis assumes that ice currently stopped behind the Merrimack Village Dam impoundment will be able to pass further downstream once the dam is removed. An ice jam at this location will extend upstream through the Chamberlain Bridge resulting in a higher water and ice surface...
level through the bridge than during an open water event at the same discharge. The ice and
water surface levels are not expected to contact the top portion of the bridge, or roadway.

The ice jam extending through the bridge may impact the vertical side walls of the bridge.
Forces acting on the vertical side walls are a result of the movement of ice rubble and tend to be
significantly smaller forces than ice crushing forces. Although scour of erodible bed sediment
during ice jams at bridges is often a concern due to the destabilization of bridge piers and
abutments, scour does not appear to be an issue at the Chamberlain Bridge, as the bridge was
constructed on exposed bedrock.

Based on the flood inundation mapping it does not appear that ice jams further downstream will
result in significantly higher water surface elevations in developed areas than during similar open
water events. For that reason, increased flooding due to ice jams once the dam is removed does
not appear to be a significant issue.

3.3 Fremont, OH
The Ballville Dam, located approximately 1.5 miles southwest of Fremont, OH on the Sandusky
River, is the subject of an investigation into the environmental impacts of its removal. There is a
concern that the Ballville Dam has mitigated damaging ice jam events in Fremont and this
benefit will be lost if the dam is removed. This study investigated how ice processes on the
Sandusky River have been impacted by the dam and whether future hydraulic modeling and
design planning should include the impacts of river ice.

The Sandusky River flows from south to north, traveling about 127 miles with a mean gradient
of 0.00074 ft/ft (Vuyovich 2008). The steepest portion of the river is a series of rapids with
exposed bedrock between the Ballville Dam and downtown Fremont, with a mean gradient of
0.003 ft/ft (Figure 5). Fremont, OH is located near the downstream end of the Sandusky River,
approximately 10 miles before the river empties into the Sandusky Bay of Lake Erie. The water
surface between Fremont, OH and the Sandusky Bay is essentially flat, as the backwater from
the bay reaches almost all the way to town. The channel bottom elevation at Fremont is below
the mean lake level at Sandusky Bay (USACE 1963). The location of Fremont, at the upstream
extent of the Sandusky Bay backwater is a likely location for ice jams to occur.

Small ice jams have been reported at the USGS gage upstream of the Ballville Dam on the
Sandusky River on average every three years. Most ice jams on the Sandusky River are minor
and cause little or no flooding. Prior to construction of the dam these small, frequent events
were reported downstream in Fremont (Thomas 1913). Significant ice jam events, those that
cause major flooding and extensive damage are less frequent. Six major ice jams were found in
recorded history; 1833, 1843, 1883, 1904, 1959 and 1963. The earliest ice jams were found in a
history of the Sandusky River written by Lucy E. Keefer (1904). According to the account, the
first State Street Bridge, a timber bridge built in 1828, was destroyed by the extraordinary ice
jam of 1833, which “bore it on a mass of ice and driftwood to the upper point of the island where
it lodged and lay stretched almost from bank to bank.” A second bridge built at the same
location the following summer was destroyed by an ice jam in 1843.
This analysis focused on the ability of the dam to retain ice during the most significant ice jam events. Of the six major ice jam events that caused extensive damage and flooding in Fremont (1833, 1843, 1883, 1904, 1959 and 1963), two occurred after the Ballville Dam was constructed; 1959 and 1963. According to historical accounts of both the 1959 and 1963, ice remained in place upstream of the Ballville Dam while flooding occurred in Fremont due to jams downstream (Figure 4). Water levels just beneath the roadway of the State Street Bridge can be seen in photos of the flooding during the 1959 event. Additional ice from upstream could have caused significant damage to the bridge had it impacted the roadway. In 1963, the ice jam was estimated to be 5 feet thick, compared to 10-25 feet estimated for the 1904 ice jam event.

Based on this analysis, the Ballville Dam has had an impact on reducing damaging ice jams in Fremont, Ohio. Further investigation of ice processes in the Sandusky River, including an ice hydraulic model, was recommended to determine if the flood walls will protect Fremont from ice jam flooding, and if the bridges in town will be at risk from additional ice resulting from the removal of the Ballville Dam.
4. Conclusion

In northern regions, it is important to investigate the potential ice issues that could arise from the removal of a dam. Failure to do so on rivers with an active ice regime could result in chronic ice problems, or worse, catastrophic ice events. In the United States, hydrologic and meteorological data is readily available in most locations. The CRREL ice jam database is a good starting point for gathering historical ice event information. More ice information can be found in local archives and project reports.

The available data and requirements of the analysis will determine what method is used to evaluate the impacts of the dam on ice. Those methods can range from a historical analysis which determines whether the dam has or has not affected ice processes, to a detailed hydraulic model which compares water surface levels for the with and without dam scenarios.

If ice is determined to be an issue in a dam decommissioning study, several alternatives exist for mitigating damaging ice jams without the dam in place. However, the time to design these for ice control is during the preliminary dam removal analysis. The first step is to determine the risk of flooding and ice damage to bridges and floodwalls caused by the additional ice carried from upstream past the current dam location. A hydraulic model of the ice conditions using HEC-RAS (USACE 2006) should be conducted to determine the water levels and flooding. Once the levels of risk are determined, a river ice control plan could be developed to reduce the probability of flooding to acceptable levels. The two major alternatives for controlling ice are permanent ice control structure and river ice management. A permanent ice control structure such as ice piers at the location of the dam (Tuthill and Lever 2006) would protect downstream areas from damaging ice jams. The expected level of protection of permanent ice control structures, such as ice piers, can be quantitatively estimated with a high level of confidence based on laboratory tests, field performance of existing structures, and ice modeling. Generally, permanent structures can be expected to provide a high level of protection. River ice management includes active measures to reduce the ice strength and/or melt the ice in place, including hole drilling, ice cutting and darkening the ice surface (Haehnel 1998). Active measures must be applied at the appropriate time, before an ice jam occurs but with enough lead time to be effective. It is often difficult to judge the appropriate time and degree of application. The expected level of protection of active measures is often hard to quantitatively estimate with confidence because of the difficulties in judging the level of performance in application. Active measures generally provide less protection than permanent structures.

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Study on Prevention of Ice Damage to Water Transfer Projects in Xinjiang, China

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Abstract: This article reviews different forms of ice damage and engineering measures taken to prevent ice damage to water transfer projects in Xinjiang, China. The geographical, climatic and hydrological factors influencing the ice damage to water projects are briefly described and causes of ice damage are analyzed. The technical measures for prevention and control of ice damage applied in engineering practice are summarized, including construction of ice storage reservoir; operation of power stations with ice cover; methods of ice thawing such as raising water temperature, electric heating, and pressure water jet; ice transport in canals; and hydraulic and mechanical measures for ice releasing. With the economy growth in Xinjiang, more and more challenges come out facing to water engineers and administrators, and so new research topics are brought about in aspects of preventing ice damage to water projects. Key words: water transfer projects; ice damage; ice transport; Xinjiang
1. Introduction
The Xinjiang Uyghur Autonomous Region located in the Northwest part of China, the East Longitude of the region is 73°30′-96°30′ and the North Latitude of it is 34°10′-49°30′. With a total area of 1,660,000 km², it is the largest province in China accounting for one sixth of the total area of the country.

1.1 Geographical features
The geographical features of Xinjiang are mountains alternating with basins and basins encircled by high mountains. It has the Altai Mountains in its north, the Kunlun Mountains in its south. As the Tianshan Mountains lies across the middle, it is divided into the north and south parts: the north Xinjiang and the south Xinjiang. The Junggar Basin saddles between Altai Mountains and the Tianshan Mountains in the North, and the Tarim Basin spans between the Tianshan Mountains and the Kunlun Mountains in the South. The elevation of the three mountains ranges from 4,000m to 8,000m a.s.l., and they have towering and broad mountain bodies. The elevation of the plain areas at the edges of the basin is from 300m to 1,200m a.s.l. The elevation difference between the mountain ridge and the plain areas at the edges of the basin can be as big as 3,000m to 6,000m.

1.2 Climatic Patterns
Xinjiang covers a vast area spanning from 34°10′ North Latitude to 49°30′ North Latitude. Its vertical elevation ranges from 155m a.s.l.(Aydin Lake) to 8,611m a.s.l.(Mount Godwin-Austen). Such a great elevation difference creates great temperature fluctuations with a range of variation from -51.5°C to 48.9°C. Because of high latitude and elevation, Xinjiang is very cold during winter and has a short frost-free period of 150 days.

1.3 Hydrological patterns
As Xinjiang is in the mid of Euro-Asia Continent and far away from seas, it characterized with dry climate. But mid-latitude west-wind circulation creates favorable conditions for water vapor transport, and the water vapor blocked and lifted by high mountains ranges forms sufficient precipitation in the mountainous areas, which means that different regions in Xinjiang vary greatly from each other in terms of annual precipitation. The overall pattern is that the north Xinjiang has more precipitation than the south Xinjiang, the northwest part has more precipitation than the southeast part, and the mountainous areas have more of it than the plain areas and basins. From the northwest of the region to its southeast the annual precipitation gradually declines. The annual precipitation in mountainous areas is around 300mm -600mm, and that of plain oasis is around 20mm -250mm. During the flood period in summer the runoff is large and during the draught period in winter the runoff is small. The ratio between the flood and draught period is usually one to several hundred or even to one thousand. Winter is very cold, and the ice period in river channels is usually 100-200 days with small water runoff but large ice runoff. The ice to water ratio in rivers is 1:4 to 1:2.

2. Overview of ice damage to water transfer projects in Xinjiang
The ice damage to water transfer projects in Xinjiang mainly occurs in diversion-type hydropower stations and water diversion projects. During the winter time diversion-type hydropower stations either operate with ice cover or rely on ice discharge, the latter will create more ice damage. Water transportation projects mainly have four types of ice damage:
Damage to canal head works: river channels in front of the canal head works froze up, resulting in phenomenon of water above ice and ice above water and making river changes its course. Sometimes ice-flood produces ice dam and inlet sluice is blocked, making water diversion impossible.

![Figure 1. Ice slush in the forebay of Jingou No. 1 Hydropower station, Feb.15, 2008](image)

Damage to water conveyance channel: high density of ice slush in water conveyance channels produces ice jam, which in turn makes the channel froze up and ice-water overflow canal dykes; sever land ice creates ice bridge and blockage, in turn making the channel intakes froze up and ice-water overflow the dykes; and hydraulic structures on canals induces ice blockage.

![Figure 2. Land-ice(left) and slush accumulation(right) in the conveyance canal of Jingou No.1 Hydropower station, Feb. 15, 2008](image)

Damage to deicing sluices: improperly designed structures of deicing sluice makes smooth ice discharge impossible, due to irrational horizontal direction and location of the structures or insufficient designed size for discharging ice.

Damage to sluice gate: sluice gate was frozen with gate slot, making the gate stuck.
3. Causes of ice damage to water transfer projects

3.1 Low temperature and long freezing period
As Xinjiang locates in a rigid cold region, winter temperature is usually between -10°C to -40°C, and the extreme low temperature could be as low as -50.8°C. The accumulated daily-mean minus temperature in one year is about -1,000°C to -1,500°C. Throughout a year the region experiences continues negative temperature for around 130 days.

3.2 Small water runoff, large ice runoff, and long ice period
Most rivers in Xinjiang are of mountain-river type and have thawed ice and snow as their source. During winter time rivers have small water runoff but large ice runoff, and over a hundred days of ice period. All this produces ice blockage, ice jam in water conveyance channel and ice-water overflow.

3.3 Improper engineering design
In early times due to limited knowledge about ice damage to water transfer projects, improper design of water projects brought about lots of problems in ice damage prevention and caused economic losses to those projects.

4. Research on preventative measures against ice damage to water transfer projects
In most of the water transfer projects in Xinjiang, including water diversion canals in diversion-type hydropower stations, ice damage heavily affects the normal operation of the stations. In some sever cases, hydropower stations are even forced to stop. For a long period the researchers, designers and engineers have been working hard to work out preventative measures against ice damage. Thanks to their efforts, the measures can be summarized as in ice storing, ice cover forming, ice thawing, and ice discharge.

4.1 Ice storing
Ice storing refers to construction of ice storage reservoirs. Ice slush running in river channel will be kept in ice storage reservoirs, and clear water without ice slush will be diverted out so as to have ice-free water flow or flow with little ice slush in canals and ensure safe operation of the projects during winter time. Ice storage reservoir falls into two types: blocking type ice storage reservoir is usually constructed at the head of water diversion canal, while injection-type ice storage pool is usually constructed at natural billabongs or valleys along the approach channel.

In designing ice storage reservoirs, the volume for storing ice is calculate according to the maximum volume of incoming ice slush. A few points must be taken into consideration: since ice would be piled up in upstream direction, the ice storage reservoir may be as large as twice of that for storing flood in summer; a skimmer wall type of inlet gate should be adopted and enough water depth over the sill crest should be ensured in order to avoid subsurface frazil ice entering into diversion canals; blocking type ice storage reservoir should eject sediment at low water level in summer time to keep enough volume for storing ice in winter. A successful example of this kind ice storage reservoir is Shihuiyao reservoir in Xinjiang, which has been properly operated for more than 30 years.
4.2 Formation of ice cover
The basic approach is practiced during the ice cover forming period. When there will be a drastic temperature drop according weather forecast, it is to raise the water level of diversion canal to the maximum water level and to reduce the flow velocity to around 0.5m/s. The whole diversion canal will form ice cover in a short period. After that, at every a certain distance, an ice hole will be drilled, and water overflowing from this hole will freeze, and the ice cover will gets thicker and thicker, and finally forms a fully enclosed ice cover in the whole canal. Then it is to draw down the water level in the canal in order to form an isolating layer (air layer) between the ice cover and water surface to isolate water flow from open air, which avoids further formation of ice. In the early of 1950s this method was successfully used in Aletai region and gradually introduced to other places in Xinjiang.

4.3 Ice melting
There are three methods to melt ice, namely, raising water temperature, pressure water jet and heating.

4.3.1 Raising water temperature to warm up the water in conveyance channels. Natural spring water, cooling water from thermal power plants and underground water from deep wells are of relatively high temperature and can be introduced to the water conveyance channel to melt ice.

In the No.4 hydropower station on the Manasi River, natural spring water is diverted, the temperature of the spring is 7°C, and the runoff is 3m³/s, accounting for 25% of the flow rate of the diverted water in winter time. So a normal operation of the diversion canal is ensured. The Jingouhe hydropower station in Shawan County raises water temperature in diversion canal by pumping underground water from deep wells. The temperature of well water is 6.5°C, and runoff 0.24m³/s, accounting for 15% runoff of the canal. And it makes the diversion canal operate smoothly in winter time.

The length $L_T$ (in kilometer) of ice-free canal in winter can be calculated by applying of a semi-empirical formula derived in the Xinjiang Survey and Design Institute for Water Conservancy and Hydropower:

$$L_T = \frac{K_T Q_s t_s}{Q_m} \quad [1]$$

where, $K_T$ is a integrated coefficient(km/°C) which can be taken from table 1, $Q_s$ is the flow rate of spring or well water(m³/s), $t_s$ stands for the temperature of spring or well water(°C), $Q_m$ is the flow rate in the canal after mixing(m³/s).

Figure. 1 shows the operation of approach canal of the Hongshanzui No. 2 hydropower station at the temperature of -15°C, Feb. 15, 2008. Three pumps were used to pump ground water to feed the approach canal and the canal water was free of ice slush.
Table 1. Value of the integrated coefficient, $K_r$ (km/°C)

<table>
<thead>
<tr>
<th>canal direction &amp; bank slope location</th>
<th>average minimum daily temperature, $t_a$ (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-10 to -15</td>
</tr>
<tr>
<td>from east to west, at north slope</td>
<td>13.2</td>
</tr>
<tr>
<td>from south to north, at both slopes;</td>
<td>12.6</td>
</tr>
<tr>
<td>from northeast to southwest, at north</td>
<td>11.5</td>
</tr>
<tr>
<td>from southeast to northwest, at south</td>
<td>10.0</td>
</tr>
</tbody>
</table>

**Figure 3.** Raising water temperature by pumping ground water into the approach canal of Hongshanzui No. 2 hydropower station in winter time.

### 4.3.2 Pressure water jet

Pressure water jet is applied to produce an ice-free water belt between the hydraulic structures and frozen water. It is to prevent hydraulic structures from freezing together with the ice, and protect the structures from static ice pressure.

The application condition for this method is the heat supplied by the pumped water from the bottom layer should be bigger than the heat loss of a 1.0m wide non-freezing surface water, an empirical formula derived and used in the Xinjiang Survey and Design Institute for Water Conservancy and Hydropower to calculate flow rate is as follows,

$$0.582Q_p t_w > 0.003K (1.553 E - t_a)WL$$

where $Q_p$ is the flow rate of the submerged pump (m$^3$/h); $t_w$ is the temperature of the deep water where the submerged pump takes water (°C); $E$ is saturated water pressure (hpa) relating with air temperatures; $t_a$ stands for the minimum air temperature (°C); $W$ stands for the maximum wind...
speed in the coldest mon th(m/s); \( L \) is the length of the non-freezing water area(m); \( K \) is a safety factor, \( K=1.5 \).

Another empirical formula (derived and used in the Xinjiang Survey and Design Institute for Water Conservancy and Hydropower) for melting speed is:

\[
\frac{\Delta \delta}{\Delta T} = 0.138 (\phi V_0)^{0.62} \frac{t_w}{h_t}
\]

where \( \Delta \delta \) is the length of ice melting; \( \Delta T \) is the time of ice melting; \( \phi \) is diameter of jet hole; \( V_0 \) is the exist velocity(m/s); \( h_t \) is water depth of the jet(m). The pump lift should be two times of the water depth of the submerged pump. The water temperature at the submerged pump location should be higher than 0.2°C. The jet pipe should be deposited as deep as 1.5m -3.0m. The method of pressure jet is proved as a good approach to control ice damage due to its simple equipment, low invest, small site-area, easy installation, convenient maintenance and good performance.

**4.3.3 Heating.** Three ways can be applied for heating: electric air heater, electric water heater, and electric oil heater. Electricity heating tube will warm up heat preservation medium such as air, water and oil. This method is mainly used to prevent ice damage to metal structures, such as gates and trash racks.

**4.4. Ice transport**

Ice transport refers to transportation of ice slush by means of water conveyance channel. Many technical factors would affect the ice transport through conveyance channel, among which the flow velocity of ice slush is the major one. Both practice and research works have proven that M.B.Podapov formula(1959) is a good one to calculate the flow velocity \( V_k \) of ice transport (m/s).

\[
V_k = 0.057 \frac{C}{\sqrt{C + 8}} \sqrt[3]{\frac{h_l}{D_i}}
\]

where \( C \) is Chezy coefficient, \( D_i \) is the particle diameter of ice slush(m), \( h_l \) is the thickness of ice slush(m).

The technical conditions for ice transport in water conveyance channel:

- The flow velocity of ice transport in conveyance channels should be between 1.2m/s -1.5m/s;
- The channel should be as straight as possible. The bend, if necessary, should have its radius 10 times bigger than the designed width of the water surface in the channel.
- For water conveyance channels that would experience a large flow rate variation from summer to winter, a compound cross section should be used to make sure that low flow in winter can get sufficient velocity and depth in the lower part of the section and ice transport can be possible.
- As to the intake basin of a hydropower station, due to the sudden expansion of waterway section, the sudden decline of flow velocity would result in difficulties for ice transport. To
keep the flow at certain velocity, two options can be selected. One is to reduce the operation water level at the intake basin, and the other is to reduce its waterway section either by using vertical partition wall in plane or movable horizontal dividers in the vertical. For instance, the intake basin of the No.2 Wupu Hydropower Station in Xinjiang uses two-layer horizontal dividers in winter, which accelerates the flow velocity in the intake basin and generates good results.

4.5 Ice release
Ice release refers to discharging of ice slush in head works or in conveyance channels by applying either hydraulic or mechanical approaches.

4.5.1 Hydraulic ice release. The ice slush will be discharged through deicing sluice. The key of this method is the layout of the deicing sluice. There are four types of arrangement of the deicing sluice: forward ice release with forward water diversion; forward ice discharge with side water diversion; side ice release with forward water diversion; and ice discharge through bends which is a special case of forward ice release. It has been proven by experience that the forward ice release would get the best result and the side ice release would get a poor result. The best structure of the deicing sluice should be of oscillating flashboard.

4.5.2 Mechanical ice release. It is to remove ice slush in mechanical means. In Xinjiang a very popular approach is to use swinging turnstile screen machine which was developed in Xinjiang 1970s. This is a very effective approach as in summer it can be used as decontaminators and in winter it can be used as deicing tools. After several decades of practice and improvement, it has been proven as a cost effective machine in ice deicing and is wildly used in Xinjiang.

5. Conclusion
Over the past several decades, a lot of studies have been done on the preventative measures against ice damage to water transfer projects. After exploration, practice and systematic sum up, researchers have obtained some positive experience as well as negative lessons, and many of the research results have been applied in engineering practice and obtained good results.

However, what we have achieved so far in developing ice damage control technology in Xinjiang can not satisfy the real needs in engineering practice. Xinjiang covers a vast area and has very non-uniform distribution of water resources, in order to address the increasing demand for water resources propelled by rapid economic growth, more water works especially water transfer projects will be in planning, designing and implementing. All the projects are and will be constructed in rigid cold area, with long distance and large flow rate. Water transfer in open channels suffers from severe ice damage. Researchers and engineers are and will be facing to greater challenges in ice damage control, more work should be conducted by means of prototype observation, model experiment and mathematical simulation in order to make breakthroughs in solving practical problems.

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Mechanical behaviour of river ice, ice covered flow, and thermal modeling
Experimental Research on Mechanical Behavior of River Ice

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The mechanical behavior of river ice is explored through analysis on current state of research on river ice in China and on basis of mechanical behavior experiments of the Huma River ice in Heilongjiang Province and in combination with current river ice mechanical behavior experimental research achievements in China. The mechanical behavior is studied through river ice mechanical behavior experiments with statistic analysis method. The law of influence of temperature and strain rate on compression strength, bending strength, and modulus of elasticity of river ice is studied systematically. A mathematical model for relation between compression strength or modulus of elasticity of river ice and ice temperature is established. The statistic law between compression strength and bending strength of river ice is studied.
1. Introduction

Ice regime usually occurs in all rivers, lakes, and reservoirs north to 30° N in China. The problem of ice flood is the most serious in the regions north to 35° N and in rivers on the Qinghai-Tibet Plateau in spring, such as the Inner Mongolian section of the Yellow River, the lower reaches section of the Yellow River, the upper reaches of the Nenjiang River, the section of the Songhua River below Yilan, and the upper reaches section of the Heilongjiang River where frequently occurring ice-jam flood during the ice flood period and the damage caused by drift ice raft to structures have caused large loss to people’s lives and properties (e.g., Gao Peisheng et al. 2003, Lin Lejiang et al. 1998). Ice load is one of the main loads that must be considered for design and construction of bridges and hydraulic structures in cold regions. Drift ice, as an occasional function, has serious influence on damage to bridge piers, stability of piers, and bridge vibration, imposing severe threat on safe operation of bridge structures. At present, there is not any perfect method both at home and abroad for determining ice load.

There are only few institutes that are engaged in study on river ice mechanical behavior and drift ice percussive force in China, and the regions under their study are only limited to few rivers, such as the Yellow River (e.g., Zhang Mingyuan et al. 1993) and rivers in Heilongjiang Province (e.g., Li Shizan 1985, Cai Zhirui et al. 1997, Lu Qinnian et al. 2002, Yu Tianlai et al. 2007). Since 80’s of the last century, the engineering and technical workers of our country used the research achievements of the former USSR and Canadian scholars as reference and gained some achievements in combination of the practical circumstances of our country. Huang Maohuan got the compression strength of river ice in a section of the Yellow River in Inner Mongolia through test and study. Zhang Mingyuan et al. (1993) got the compression strength, bending strength, and shearing strength of the river ice in the Yellow River mouth and the relation between strength, temperature, and strain rate was analyzed through test and study. The former Qiqihar Railway Bureau (1985) completed on-site experiment on drift ice dynamic pressure against bridge piers. Cai Zhirui (1997) carried out test and study on percussive force of drift ice against bridge piers in Harbin and Jiamusi sections of the Songhua River respectively. Lu Qinnian (2002) carried out test and study on the mechanical behavior and percussive force of river ice in Harbin section of the Songhua River and proposed an ice pressure calculating formula that suited the ice regime of Heilongjiang Province. Yu Tianlai (2007) completed their study on the mechanical behavior of the Huma River ice and its percussive force against bridge piers.

For study on river ice load in our country, the main method is to find out the mechanical behavior and percussive force of the ice of some rivers through lab tests and on-site tests. No systematic and deep study on the river ice has been done. The existing problems are: Only few objects for river ice have been studied. The temperature range is small in the test. Test data of river ice are only limited to some particular rivers. Since the properties of river ice are scattered in a wide range in different regions and for different rivers, now we don’t have sufficient representative test data on river ice. The values of the standard strength of river ice in the current applicable codes are questionable. There are fewer test data about percussive force of drift ice against bridge piers. The method of calculation about the percussive force of drift ice against bridge piers in the currently applicable codes is not well backed by test data.

In view of the insufficiencies mentioned above, relations between temperature, loading rate, and strength or modulus of elasticity are studied and mathematical models between ice temperature

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and strength or modulus of elasticity are established by this paper. Through the study of this paper, it can provide the theoretical basis for calculation method about percussive force of drift ice on bridge structures.

2. Mechanical Behavior Experiment on River Ice

On basis of the achievements of study on the mechanical behavior experiment of the Huma River ice in Jan. to Feb. 2006 and on basis of the analysis on the achievements by other institutes, the purpose of this paper is to explore the law of mechanical behavior of river ice from three aspects: compression test, bending test, and modulus of elasticity.

2.1 Uniaxial Unconfined Compression Test

Uniaxial unconfined compression test on river ice is of clear mechanical meaning. It is the most fundamental and most important test in study of mechanics. The Huma River ice at five different temperatures was tested with five different loading rates in Jan. and Feb. 2006. The results of the test are listed in Table 1.

<table>
<thead>
<tr>
<th>Loading Rate / mm•min(^{-1})</th>
<th>-5°C</th>
<th>-15°C</th>
<th>-20°C</th>
<th>-30°C</th>
<th>-35°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>2.0186</td>
<td>3.7775</td>
<td>4.4550</td>
<td>7.2575</td>
<td>6.4338</td>
</tr>
<tr>
<td>10</td>
<td>3.5114</td>
<td>5.6588</td>
<td>6.4886</td>
<td>6.9300</td>
<td>5.1750</td>
</tr>
<tr>
<td>20</td>
<td>3.4788</td>
<td>4.5775</td>
<td>5.1350</td>
<td>5.4675</td>
<td>6.8850</td>
</tr>
<tr>
<td>30</td>
<td>2.7587</td>
<td>2.7900</td>
<td>4.0157</td>
<td>6.6863</td>
<td>6.0675</td>
</tr>
<tr>
<td>60</td>
<td>2.8288</td>
<td>3.5263</td>
<td>4.1213</td>
<td>5.6100</td>
<td>3.7471</td>
</tr>
<tr>
<td>Average Value</td>
<td>2.9192</td>
<td>4.0660</td>
<td>4.8431</td>
<td>6.3903</td>
<td>5.6617</td>
</tr>
</tbody>
</table>

2.1.1 Effect of Temperature on Compression Strength

A linear model was used to set up a mathematical model for relation of compression strength of river ice and ice temperature by Zhang Mingyuan et al. (1995), but the error is too high if such model is used to calculate the compression strength of the Huma River ice, so the model is not applicable to our case. In consideration of the regional features and the difference of water flow during drift ice period, and on basis of the results in Table 1, a mathematical model for relation of the compression strength of the Huma River ice and ice temperature is established in this paper. A relational expression for the compression strength and ice temperature of the Huma River ice in the range of -5°C~ -30°C is obtained through data fitting.

\[ f_i = At + B \]  \[1\]

Where: \( A \) and \( B \) are fitting coefficients as shown in Table 2; \( t \) is the absolute value of ice temperature, from -5°C to -30°C, and the same below.

Table 2 shows that under loading rates of 0.5, 20, and 60 mm•min\(^{-1}\), correlation coefficient \( r \) is greater than \( r_{\text{min}}=0.95 \). The linear fitting is satisfactory. While under loading rates of 10 or 30 mm•min\(^{-1}\), linear fitting is not satisfactory and polynomial fitting is better for the fitting.

<table>
<thead>
<tr>
<th>Formula</th>
<th>Strain Rate / 10(^{-3})ε•s(^{-1})</th>
<th>( A )</th>
<th>( B )</th>
<th>Correlation Coefficient ( r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_i = At + B )</td>
<td>0.048</td>
<td>0.21</td>
<td>0.61</td>
<td>0.98</td>
</tr>
<tr>
<td>0.95</td>
<td>0.14</td>
<td>3.23</td>
<td>0.94</td>
<td></td>
</tr>
<tr>
<td>1.90</td>
<td>0.08</td>
<td>3.25</td>
<td>0.96</td>
<td></td>
</tr>
<tr>
<td>2.86</td>
<td>0.16</td>
<td>1.25</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>5.71</td>
<td>0.12</td>
<td>1.95</td>
<td>0.97</td>
<td></td>
</tr>
</tbody>
</table>
For data at strain rate of 0.00095 $\varepsilon\cdot s^{-1}$, the fitting result is:

$$f_i = -0.0057t^2 + 0.3369t + 1.9552$$ \[2\]

For data at strain rate of 0.00286 $\varepsilon\cdot s^{-1}$, the fitting result is:

$$f_i = 0.0088t^2 - 0.1474t + 3.2336$$ \[3\]

The correlation coefficient $r$ in expressions [2] and [3] is greater than 0.99, showing an ideal fitting.

Taking strain rate of $3\times10^{-3} \varepsilon\cdot s^{-1}$ as an example, the test values of compression strength of the river ice in the Inner Mongolian section of the Yellow River, in the Yellow River Mouth (e.g., Zhang Mingyuan et al. 1993), in the Songhua River (e.g., Lu Qinnian et al. 2002), and in the Huma River are listed in Table 3. They can be compared with the calculated results from expression [3].

<table>
<thead>
<tr>
<th>Sample Place</th>
<th>Test Value</th>
<th>Error</th>
<th>Test Value</th>
<th>Error</th>
<th>Test Value</th>
<th>Error</th>
<th>Test Value</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inner Mongolian Ice Fringe</td>
<td>3.309</td>
<td>18%</td>
<td>3.659</td>
<td>28%</td>
<td>3.988</td>
<td>25%</td>
<td>——</td>
<td>——</td>
</tr>
<tr>
<td>Yellow River Clear Ice</td>
<td>2.485</td>
<td>-9.3%</td>
<td>3.032</td>
<td>13%</td>
<td>3.469</td>
<td>13%</td>
<td>——</td>
<td>——</td>
</tr>
<tr>
<td>River ice in Yellow River mouth</td>
<td>2.620</td>
<td>-3.7%</td>
<td>3.210</td>
<td>18%</td>
<td>3.450</td>
<td>13%</td>
<td>5.340</td>
<td>29%</td>
</tr>
<tr>
<td>Harbin section of Songhua River</td>
<td>2.540</td>
<td>-7%</td>
<td>2.920</td>
<td>9.6%</td>
<td>3.300</td>
<td>9%</td>
<td>3.680</td>
<td>-3.4%</td>
</tr>
<tr>
<td>Huma River ice</td>
<td>2.759</td>
<td>1.5%</td>
<td>2.774</td>
<td>4.9%</td>
<td>2.790</td>
<td>-7.6%</td>
<td>4.016</td>
<td>5.2%</td>
</tr>
<tr>
<td>Calculated value of expression [3]</td>
<td>2.717</td>
<td></td>
<td>2.640</td>
<td></td>
<td>3.003</td>
<td></td>
<td>3.806</td>
<td></td>
</tr>
</tbody>
</table>

Notes: 1. The compression strength of the river ice in the Inner Mongolian section of the Yellow River is obtained through calculation according to reference [3].

2. In certain temperature range, the compression strength of river ice has linear relation with temperature. So when related test data are not available, calculation can be made by method of interpolation.

It is indicated by Table 3 that the compression strength of river ice in the Inner Mongolian section of the Yellow River and in the Yellow River Mouth has higher errors compared with the calculated values from expression [3], where the compression strength of river ice in Harbin section of the Songhua River and the Huma River has lower errors compared with the calculated values from expression [3]. Therefore we should divide the whole country into a few zones and carry out systematic study respectively and establish mathematical models for relation of ice temperature and compression strength.

According to the data in Table 1 and Table 3 and with respect to the current places of sample origin, when strain rate is about $3\times10^{-3} \varepsilon\cdot s^{-1}$, the relation of compression strength and ice temperature.
compression strength and temperature is shown in Figure 1.

It is shown in Figure 1 that under given strain rate, compression strength increases with decrease of ice temperature in the range of test temperatures except for the Huma River ice. The compression strength of the Huma River ice increases with temperature in the range of -5°C--30°C, and reaches the maximum at -30°C. Below -30°C, its compression strength will reduce with decrease of temperature due to the change of ice phase. The conclusion from some researchers in the past that compression strength increased with decrease of ice temperature was only a one-sided view, since their tests were carried out in a temperature range that was too narrow for an overall understanding of the relation between compression strength and temperature of ice.

2.1.2 Effect of Strain Rate on Compression Strength
Curves of relation between compression strength and strain rate at -5°C and -15°C for the Huma River ice and the Songhua River ice are shown in Figure 2.

Figure 2 shows that under a fixed same temperature and when strain rate is less than $(0.952\sim1.426)\times10^{-3}\, \varepsilon\cdot s^{-1}$, the compression strength of the Huma River ice increases with increase of strain rate. After the strain rate exceeds this range, the compression strength tends to decrease with increase of strain rate. The compression strength reached the maximum within this range of strain. The inflexion point occurs in the strain range of $(0.5\sim1.75)\times10^{-3}\, \varepsilon\cdot s^{-1}$ for river ice in Harbin section of the Songhua River, similar to the inflexion range of the Huma River ice. But the inflexion point of the river ice in the Yellow River mouth has lower value, in the range of $(0.04\sim0.2)\times10^{-3}\, \varepsilon\cdot s^{-1}$. It is known from the result of test that the compression strength of river ice has its maximum with changing strain rate, but the inflexion points of strain rates for rivers in different regions differ too much.

2.1.3 Analysis on Ductile-Brittle Destruction
The stress-strain relation under various loading rates for the Huma River ice at -5°C is shown in Figure 3.

It is shown in Figure 3 that the river ice appears in its brittle property at high strain rate, while appears in its ductile property at low strain rate under a fixed temperature. Ductile and brittle properties coexist in a certain range of strain rate, and the compression strength of ice reaches its maximum in the range. It is known from Figure 3 that the strain rate at the separation point for ductile-brittle destruction falls between $0.0476\times10^{-3}\, \varepsilon\cdot s^{-1}$--$0.952\times10^{-3}\, \varepsilon\cdot s^{-1}$ for the Huma River ice at -5°C.
2.2 Bending Test

In ice load calculation expression, we need to use bending strength of river ice for the calculation for bridge piers with inclined surface. Bending tests are carried out to determine the bending strength of river ice accurately. Now the methods used in ice bending test include cantilever beam bending test, three-point bending test, and four-point bending test. Three-point bending test is carried out for river ice in the Yellow River mouth (e.g., Zhang Mingyuan et al. 1993) and for the Songhua River ice (e.g., Lu Qinnian et al. 2002), with expression [4] as the bending strength calculation expression. Four-point bending test is carried out for the Huma River ice (e.g., Yu Tianlai et al. 2007), with expression [5] as the bending strength calculation expression. The results of bending tests are shown in Table 4.

<table>
<thead>
<tr>
<th>Loading Rate / mm•min⁻¹</th>
<th>Bending Strength / MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.7768</td>
</tr>
<tr>
<td>20</td>
<td>1.3949</td>
</tr>
<tr>
<td>40</td>
<td>1.7582</td>
</tr>
<tr>
<td>60</td>
<td>1.8949</td>
</tr>
<tr>
<td>Average Value</td>
<td>1.7062</td>
</tr>
</tbody>
</table>

\[
\sigma = \frac{3}{2} \frac{pl}{bh^2} \quad [4]
\]

\[
\sigma = \frac{pl}{bh^2} \quad [5]
\]

Where \( p \) is the destructive load of the beam; \( l \) is the span of the beam; and \( b \) and \( h \) are the width and height of the cross section.

It is indicated in Table 4 that under a fixed loading rate, in the temperature range of -5°C–30°C, bending strength generally increases gradually with decrease of temperature except for at -20°C, and will reach maximum at -30°C. After -30°C, bending strength tends to decrease with decrease of temperature.

Russia, China, and other countries suggest that the bending strength of river ice be 0.7 times of its compression strength (e.g., Cai Zhirui 1988). From comparison and analysis of the test data in compression strength test and bending strength test of the Huma River ice as shown in Table 1 and Table 4, it is indicated that at the same temperature and under the same loading rate, the bending strength is always less than compression strength. Through statistic analysis on the test data under the same loading rate, the bending strength of the Huma River ice is 0.40–0.67 times of its compression strength, averagely being 0.54 times, different from the suggested value. According to Lu Qinnian et al. (2002), the ratio of bending strength to compression strength for the Songhua River ice is 0.5–0.82 times, averagely being 0.7 times. This value is the same as the suggested value. It can be seen that the ratio of bending strength to compression strength of river ice for different rivers are in different values. Their specific values shall be determined through statistic analysis on test data.
3 Modulus of Elasticity of River Ice

Modulus of elasticity of river ice is one of the basic parameters for computer simulation analysis on percussive force of drift ice. Modulus of elasticity of river ice is seldom studied in China at present. We have not had a full-rang understanding to its law. For this reason, modulus of elasticity of river ice is discussed by this paper based on the test results of uniaxial unconfined compression test and bending test on the Huma River ice.

3.1 Compressive Modulus of Elasticity

It is known from Figure 3 that in the initial stage of compression test, elasticity of river ice is not significant. Analysis on its main reasons indicates that the surfaces of the test pieces are not made smooth enough. The pressing head of the tester does not have close contact with the test piece, and the test pieces have errors in its dimension. So we cut off an elastic section of the rising part of stress-strain curve and calculate the values of modulus of elasticity according to expression [6]. The results are shown in Table 5.

\[ E = \frac{l}{bh} \frac{\Delta p}{\Delta l} \]  

Table 5. Compressive modulus of elasticity.

<table>
<thead>
<tr>
<th>Loading Rate / mm•min(^{-1})</th>
<th>Modulus of Elasticity E/GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>-5°C</td>
<td>0.443 0.548 0.572 0.641 0.589</td>
</tr>
<tr>
<td>-10°C</td>
<td>0.546 0.651 0.662 0.738 0.604</td>
</tr>
<tr>
<td>-20°C</td>
<td>0.616 0.628 0.670 0.794 0.682</td>
</tr>
<tr>
<td>-30°C</td>
<td>0.527 0.559 0.618 0.835 0.513</td>
</tr>
<tr>
<td>-35°C</td>
<td>0.507 0.529 0.572 0.688 0.487</td>
</tr>
<tr>
<td>Average Value</td>
<td>0.528 0.583 0.619 0.739 0.575</td>
</tr>
</tbody>
</table>

It is known from Table 5 that the compressive modulus of elasticity of the Huma River ice is between 0.528 to 0.739GPa. The curve of relation between compressive modulus of elasticity and temperature is shown in Figure 4. It is indicated by Table 5 and Figure 4 that the compressive modulus of elasticity of river ice changes with temperature and has an obvious brittle point. Generally under the same loading rate, modulus of elasticity of river ice increases with decrease of ice temperature, and will reach its maximum at -30°C. Modulus of elasticity of river ice gradually decreases with decrease of river ice temperature below -30°C.

3.1.1 Effect of Temperature on Compressive Modulus of Elasticity

Single element variance analysis method is used to test the significance of effect of temperature on compressive modulus of elasticity under various loading rates. As shown in Table 6, the results of test indicate that temperature has significant effect on compressive modulus of elasticity.

Regression analysis is made to Table 5 to establish correlation of compressive modulus of elasticity and temperature of river ice and test the confidence of the regression equation. A
relation expression between compressive modulus of elasticity and temperature of river ice is obtained through fitting in the range of -5 ~ -30°C as follows:

Table 6. Single element variance analysis on compressive modulus of elasticity.

<table>
<thead>
<tr>
<th>Loading Rate/ mm/min</th>
<th>$F$</th>
<th>$F_{0.05}(df_A,df_e)$</th>
<th>$F_{0.01}(df_A,df_e)$</th>
<th>Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>2.87</td>
<td>2.68</td>
<td>3.99</td>
<td>Significant</td>
</tr>
<tr>
<td>10</td>
<td>8.33</td>
<td>2.66</td>
<td>3.95</td>
<td>Very Significant</td>
</tr>
<tr>
<td>20</td>
<td>1.20</td>
<td>2.67</td>
<td>4.02</td>
<td>Not Significant</td>
</tr>
<tr>
<td>30</td>
<td>9.07</td>
<td>2.67</td>
<td>4.02</td>
<td>Very Significant</td>
</tr>
<tr>
<td>60</td>
<td>2.07</td>
<td>2.71</td>
<td>4.07</td>
<td>Not Significant</td>
</tr>
</tbody>
</table>

Note: $F \geq F_{0.01}(df_A,df_e)$ means very significant; $F_{0.05}(df_A,df_e) \leq F \leq F_{0.01}(df_A,df_e)$ means significant; $F \leq F_{0.05}(df_A,df_e)$ means not significant.

$E_c = Ct + D$ \[7\]

Where $E_c$ is compressive modulus of elasticity of river ice; $C$ and $D$ are the fitting coefficients, as shown in Table 7.

Table 7. Values of $C$ and $D$ in fitting formula.

<table>
<thead>
<tr>
<th>Loading Rate /mm•min$^{-1}$</th>
<th>$C$</th>
<th>$D$</th>
<th>Correlation Coefficient $r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>0.00779</td>
<td>0.415</td>
<td>0.99</td>
</tr>
<tr>
<td>10</td>
<td>0.00717</td>
<td>0.533</td>
<td>0.92</td>
</tr>
<tr>
<td>20</td>
<td>0.00483</td>
<td>0.575</td>
<td>0.93</td>
</tr>
<tr>
<td>30</td>
<td>0.0157</td>
<td>0.343</td>
<td>0.98</td>
</tr>
<tr>
<td>60</td>
<td>0.00727</td>
<td>0.447</td>
<td>0.94</td>
</tr>
</tbody>
</table>

It is indicated by Table 7 that under loading rates of 0.5 and 40 mm•min$^{-1}$, the correlation coefficient $r$ is greater than $r_{min}=0.95$, showing good fitting. While under loading rates of 10, 20, and 30 mm•min$^{-1}$, it is not proper to make linear fitting, so polynomial fitting is used.

For data at loading rate of 10 mm/min, the result of fitting is:

$$E_c = -0.0002t^2 + 0.015t + 0.4826$$ \[8\]

For data at loading rate of 20 mm/min, the result of fitting is:

$$E_c = 0.0002t^2 - 0.0032t + 0.6263$$ \[9\]

For data at loading rate of 60 mm/min, the result of fitting is:

$$E_c = 0.0003t^2 - 0.0036t + 0.5167$$ \[10\]

The fitting correlation coefficient $r$ of expressions [8], [9], and [10] is greater than $r_{min} = 0.95$, indicating the fitting is satisfactory. For convenience in application, polynomial is used to establish a unified relation between compressive modulus of elasticity and temperature of river ice under various loading rate conditions as shown in expression [11].

$$E_c = 0.0002t^2 + 0.0008t + 0.519$$ \[11\]
Values of compressive modulus of elasticity are calculated with expression [11] and comparison is made with the test results in Table 5. The maximum error between test values and calculated values is within 19%. To calculate compressive modulus of elasticity with expression [11] can meet our basic requirements. It is suggested that when conditions allow, fitting formulae under different loading rates be used to calculate compressive modulus of elasticity of river ice.

### 3.1.2 Effect of Loading Rate on Compressive Modulus of Elasticity

Curves of relation between compressive modulus of elasticity and loading rate are drawn in Figure 5 on basis of the data in Table 6.

It is indicated by Figure 5 that under a fixed temperature, compressive modulus of elasticity changes with loading rate and has an extreme point. Under different temperatures, the loading rates at peak value of compressive modulus of elasticity are in different values. Under the same temperature, when loading rate is lower than the peak value, modulus of elasticity generally increases with increase of loading rate, and when loading rate is higher than the peak value, modulus of elasticity decreases with increase of loading rate.

### 3.2 Bending Modulus of Elasticity

According to the theory of elasticity, in case of four-point bending, the bending modulus of elasticity is:

\[
E = \frac{23}{108} \frac{f^3}{p} \frac{1}{f}
\]

Where \( f \) is the deflection at the middle of span of the beam.

According to the results of bending tests if we do not consider the date in the initial stage of the tests to calculate bending modulus of elasticity by expression [12], the results of calculation are shown in Table 8.

It is indicated through comparison and analysis on the calculated results of compressive modulus of elasticity and bending modulus of elasticity in Table 5 and Table 8 that under a fixed temperature and loading rate, values of compressive modulus of elasticity are always lower than values of bending modulus of elasticity. Through statistical analysis on calculated modulus of elasticity under a fixed loading rate, compressive modulus of elasticity is 0.35 ~ 0.52 times of bending modulus of elasticity, 0.40 times in average. The relation between bending modulus of elasticity and temperature is shown in Figure 6.

It is indicated by Table 8 and Figure 6 that bending modulus of elasticity of river ice changes with temperature, and has no obvious brittle point. Within the temperature range in the tests, the bending modulus of elasticity of river ice generally increases with decrease of ice temperature.
under a fixed loading rate. This relation is different to the relation between compressive modulus of elasticity and temperature.

### Table 8. Bending modulus of elasticity.

<table>
<thead>
<tr>
<th>Loading Rate / mm(\cdot)min(^{-1})</th>
<th>Modulus of Elasticity E/GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>-5°C</td>
<td>0.805</td>
</tr>
<tr>
<td>-15°C</td>
<td>1.265</td>
</tr>
<tr>
<td>-20°C</td>
<td>1.653</td>
</tr>
<tr>
<td>-30°C</td>
<td>1.698</td>
</tr>
<tr>
<td>-35°C</td>
<td>2.679</td>
</tr>
<tr>
<td>5 mm/min</td>
<td>1.265</td>
</tr>
<tr>
<td>20 mm/min</td>
<td>1.783</td>
</tr>
<tr>
<td>40 mm/min</td>
<td>1.538</td>
</tr>
<tr>
<td>60 mm/min</td>
<td>1.648</td>
</tr>
<tr>
<td>Average Value</td>
<td>1.562</td>
</tr>
</tbody>
</table>

![Figure 6. Curves of relation between bending modulus of elasticity and temperature.](image)

### 3.2.1 Effect of Temperature on Bending Modulus of Elasticity

Single element variance analysis method is used to test the significance of effect of temperature on bending modulus of elasticity under various loading rates. As shown in Table 9, the results of test indicate that temperature has significant effect on bending modulus of elasticity.

### Table 9. Single element variance analysis on bending modulus of elasticity.

<table>
<thead>
<tr>
<th>Loading Rate / mm/min</th>
<th>F</th>
<th>(F_{0.05}(df_\alpha,df_e))</th>
<th>(F_{0.01}(df_\alpha,df_e))</th>
<th>Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>20.75</td>
<td>2.74</td>
<td>4.14</td>
<td>Very Significant</td>
</tr>
<tr>
<td>20</td>
<td>4.70</td>
<td>2.73</td>
<td>4.11</td>
<td>Very Significant</td>
</tr>
<tr>
<td>30</td>
<td>3.94</td>
<td>2.70</td>
<td>4.04</td>
<td>Significant</td>
</tr>
<tr>
<td>60</td>
<td>1.86</td>
<td>2.73</td>
<td>4.11</td>
<td>Not Significant</td>
</tr>
</tbody>
</table>

Note: \(F \geq F_{0.01}(df_\alpha,df_e)\) means very significant; \(F_{0.05}(df_\alpha,df_e) \leq F \leq F_{0.01}(df_\alpha,df_e)\) means significant; \(F \leq F_{0.05}(df_\alpha,df_e)\) means not significant.

Regression analysis is made to Table 8 to establish correlation of bending modulus of elasticity and temperature of river ice and test the confidence of the regression equation. A relation expression between bending modulus of elasticity and temperature of river ice is obtained through fitting in the range of -5 ~ -30 as follows:

\[
E_w = at^2 + bt + c
\]  

[13]

Where \(E_w\) is bending modulus of elasticity of river ice; \(a\), \(b\), and \(c\) are the fitting coefficients, as shown in Table 10.

![Table 10. Values of fitting coefficients a, b, and c.](image)

<table>
<thead>
<tr>
<th>Loading Rate / mm/min</th>
<th>(a)</th>
<th>(b)</th>
<th>(c)</th>
<th>Correlation Coefficient / (r)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>-0.0014</td>
<td>0.0857</td>
<td>0.3907</td>
<td>0.98</td>
</tr>
<tr>
<td>20</td>
<td>-0.0016</td>
<td>0.0807</td>
<td>0.8406</td>
<td>0.96</td>
</tr>
<tr>
<td>40</td>
<td>-0.0011</td>
<td>0.0576</td>
<td>0.8842</td>
<td>0.99</td>
</tr>
<tr>
<td>60</td>
<td>0.0001</td>
<td>0.0113</td>
<td>1.0396</td>
<td>1.00</td>
</tr>
</tbody>
</table>

It is indicated by Table 10 that under various loading rates, the correlation coefficient \(r\) is greater than \(r_{\text{min}}=0.95\), showing the fitting is satisfactory with polynomial. For convenience in application, a unified relation as shown in expression [14] between bending modulus of elasticity and temperature of river ice is established.

\[
E_w = -0.001t^2 + 0.0589t + 0.7888
\]  

[14]
Values of bending modulus of elasticity under various temperatures are calculated according to expression [14] and the results of calculation are compared with the test results in Table 8. Large errors are found. The maximum error can reach 31%. Therefore bending modulus of elasticity shall be calculated using the fitting formulae under different loading rates.

3.2.2 Effect of Loading Rate on Bending Modulus of Elasticity

The relation between bending modulus of elasticity and loading rate is shown in Figure 7. It is indicated by Figure 7 that under a fixed temperature, bending modulus of elasticity changes with loading rate and has an extreme point. Under different temperatures, the loading rates at peak value of bending modulus of elasticity are basically the same. Under the same temperature, when loading rate is lower than the peak value, modulus of elasticity generally increases with increase of loading rate, and when loading rate is higher than the peak value, modulus of elasticity decreases with increase of loading rate.

4. Conclusions

On basis of analysis on the current state and inadequacy of river ice study in China and on basis of the mechanical behavior tests on the ice of Huma River in Heilongjiang Province, this paper tries to explore the mechanical behaviors of river ice in connection with the current achievements in research of river ice, and the conclusion as follows are obtained.

(1) Compression strength and bending strength of river ice under various temperatures and loading rates have been obtained and study on the effect of temperature and loading rate on compression strength and bending strength has been carried out. The understood laws are as follows. Generally speaking, compression strength and bending strength reach their maximum at -30°C, and tend to decrease below -30°C. In the range of -5~20°C, effect of strain rate on compression strength appears in the same way. When strain rate is approximately 10^-3 s^-1, compression strength reaches its maximum. When strain rate exceeds this value, compression strength decreases gradually.

(2) A mathematical model for relation between compression strength and ice temperature has been established. The relation between ice temperature, strain rate, and compression strength for current sample places has been analyzed. Mathematical models are developed for more and less different ice temperatures and strength for rivers in different regions.

(3) Compressive modulus of elasticity and bending modulus of elasticity under different temperatures and loading rates have been obtained and study on the effect of temperature and loading rate on modulus of elasticity has been carried out. The understood laws are as follows. Compressive modulus of elasticity of river ice changes with temperature and has an obvious brittle point. Compressive modulus of elasticity reaches its maximum at -30°C, and tends to decrease below -30°C. Bending modulus of elasticity of river ice increases with decrease of...
temperature. Modulus of elasticity changes with loading rate and has its maximum. The loading rates for maximum bending modulus of elasticity at different temperatures are approximately 20 mm/min, but the loading rates for maximum compressive modulus of elasticity are more or less different. The compressive modulus of elasticity of river ice is 0.4 times of its bending modulus of elasticity.

(4) This is the first time to establish a mathematical model for relation between modulus of elasticity and temperature of river ice, providing a theoretical basis for calculating modulus of elasticity.

Acknowledgments
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Vertical diffusion in ice-covered flow

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Turbulent momentum transfer in the nature ice-covered flows is considered. The method of estimate of the turbulent viscosity in the core of an ice-covered flow is developed. Coefficient of turbulent diffusion in the boundary layers formed at the bottom and at the ice cover is calculated by using the Boussinesq hypothesis starting from power velocity profile. Depth distribution of turbulent transfer coefficient is calculated as a function of mean characteristics of a flow.
1. Introduction
The majority of rivers in Russia are covered with ice during several months a year. The intensity of turbulence diffusion plays a great role in impurity transfer in nature streams. Despite of small value of turbulent transfer in comparison with that along stream-wise direction it counts for much significance in rivers and reservoirs. Turbulence diffusion is one of mechanisms of oxygen transfer to the bottom regions of a flow. The estimate of time of dilution of waste waters till non-dangerous concentrations demands knowledge of the magnitude of coefficient of turbulent momentum transfer $K$ and its distribution through the depth. Turbulence diffusion is also very important during spring ice melting at nature water bodies. In practice vertical turbulent transfer is often estimated by depth averaged coefficient as it is many times smaller than that in horizontal transverse direction. In the vicinity of a source of impurity it may appear to be not sufficient.

Ice-covered channel flow is characterized by two boundaries of different roughness: bottom and ice cover. In small rivers the ice cover is usually attached to the banks, and the flow is similar to the flow in a pipe with the rectangular cross section. Similar flows can be formed in large rivers far from a hydroelectric power station. As a rule, the surface of the underneath of ice is smoother than that of the bottom, but its roughness depends on conditions of freezing-over and varies during winter. If there are cracks between the banks and the ice cover, it freely floats at the water surface changing the shape of velocity profile and hydraulic resistance of the flow.

In lakes and reservoirs which freeze in winter there also appear ice-covered flows formed by the ice cover and immovable bulk of water [Anisimova et al. (2001)]. In reservoirs such flow can be initiated by the outlet from the aft bay and in lakes – by effluent. In these cases the boundary layer is generated at the underneath of ice, and at the interface between the current and immovable bulk of water appears the mixing layer.

The system of Reynolds equations describing stream motion includes the stress tensor of Reynolds, which is usually unknown. To close the system one uses the tensor of turbulent transfer coefficients $K$, which are connected with the mean characteristics of a flow. In this work we study the connection of vertical turbulent transfer coefficient with the mean characteristics of nature flow.

2. Difficulties in description of transfer coefficient in a flat ice-covered flow
Let us consider an ice-covered flow of the depth $h$, width $B$ and stream-wise velocity $u$ in the system of coordinates: $x$ is stream-wise axis, $y$ is vertical axis ($y = 0$ is the bottom), $z$ is transverse axis, velocity components $v$ and $w$ correspond to the axes $y$ and $z$. Analysis of theoretical approaches to the nature of streams and the methods of their modeling shows the advantages of the models of turbulence based on description of turbulence kinetic energy transfer [Prediction methods…(1980)]. However the equations connecting kinetic energy with the shear stress must be modified for the flows where shear stress reverses sign. Requirement of zero magnitudes of turbulent viscosity $\nu_t$ and vertical coefficient of momentum transfer $K_y$ simultaneously with zero velocity gradients turns out to be not valid. In particular such situation arises in ice-covered flows in which mass and momentum are transferred through the plane, where $\frac{du}{dy} = 0$. In this case water motion is described with the help of equations for kinetic
energy \(k\) and energy dissipation \(\varepsilon\) of a unit of mass and coefficient of turbulent diffusion can be expressed as

\[
K_y = \frac{\nu_t}{\text{Sc}},
\]

where \(\text{Sc}\) is the turbulent Schmidt number.

In generalizations of \(k - \varepsilon\) models, developed by different authors the expression for \(K_y\) is obtained in the form [Prediction … (1980)]

\[
K_y = C \cdot \frac{k^2}{\varepsilon \text{Sc}},
\]

where \(C = 0.09\) is the empirical constant [Nosov (2004)].

3. Turbulent momentum transfer in nature streams
In a general case molecular and momentum transfer should described by different transfer coefficients, \(K_m\) and \(K_y\) correspondingly [Batchelor (1970)]. The results of the works [Fischer (1973), Jobson & Sayre (1970), Tennekes & Lumley (1972), Roberts & Webster (2002)] show that for impurity of neutral buoyancy one can take the \(K_y = K_m\), and the turbulent Schmidt number \(\text{Sc} = 1\). This enables one to compare magnitudes of transfer coefficients obtained in experiments with dye with those calculated from the velocity measurements. However, for describing of the mixing layers of water of different densities it was suggested to use \(\text{Sc} = 0.5\) [Prediction … (1980)]. In the following we consider vertical component of turbulent transfer coefficient \(K_y\). Momentum transfer in open flow is adequately described by Boussinesq hypothesis, where \(\tau_{xy}\) is the shear stress tensor component

\[
\rho K_y \frac{\partial u}{\partial y} = \tau_{xy}, \quad \tau_{xy} = -\rho \overline{u'v'}
\]

\(\rho\) is density of water, \(u', v'\) are fluctuations of longitudinal and vertical components of stream velocity.

4. Coefficient of turbulent vertical transfer in open channel flow
Turbulent diffusion in open flow can be described by coefficient of eddy viscosity expressed by analogy with molecular viscosity \(\nu\) in the form \(K_y = -\rho u'v'/\rho du/dy\). Except for the boundary layers the magnitude of \(K_y\) in nature streams is not constant and considerably exceeds molecular viscosity. Using the depth distributions of the shear stress \(\tau_{xy}\) and mean longitudinal velocity \(u\) it is possible to calculate the coefficient of turbulent diffusion and to close Reynolds equations. Basing on the linear and power laws for the dependence of \(\tau_{xy}\) and \(u\) on the depth, the expression (3) for \(K_y\) can be rewritten as [Dolgopolova (2007)]

\[
K_y = \frac{\kappa^2 <u> h n (1-y/h)}{(1+n)(y/h)^{n+1}},
\]
where \( n \) is the variable power exponent, depending on the hydraulic resistance of a flow. Normalizing (4) by \(<u>h\) and by \( \kappa n \), one obtains the dimensionless coefficients of turbulent diffusion \( K'_y \) and \( K''_y \)

\[
K'_y = \frac{\kappa^2 n (1 - y/h)}{(1 + n) (y/h)^{n-1}}, \tag{5}
\]

\[
K''_y = \frac{\kappa(1 - y/h)}{(1 + n) (y/h)^{n-1}} \tag{5 a}
\]

Numerous experiments show that in nature streams the power \( n \) in profile of velocity varies in the range \( n = 0.1 – 0.3 \). In this range the coefficient \( K''_y = K'_y / \kappa n \) is practically constant [Dolgopolova (2007)] (Figure 1), and expression (5a) may be considered as a universal one for river flows.

Comparison of (5a) with dimensionless coefficient \( \varepsilon_y/u_s h \) which was obtained by Elder (1959), starting from the logarithmic velocity profile \( \varepsilon_y = (y/h)(1 - y/h)\kappa h u_s \), shows good correspondence (Figure 1). Results of experiments in rivers [Sukhodolov et al. (1998)] confirm adequacy of using of expression (5a) for description of turbulent transfer coefficient in open channel flows.

5. Two-layer description of ice-covered flow

Let us consider an ice-covered flow as a stream composed of two currents, which are formed by bottom and by the ice cover [Uzuner (1975)]. In each of these currents the depth distributions of velocity and shear stress may be described by power and linear laws correspondingly [Dolgopolova (2002)], where subscribes “b” and “i” are used for flows formed by bottom and ice boundary surfaces

\[
u_b = (1 + n_b) <u>_b \left(\frac{y}{h_b}\right)^{n_b}, \quad \nu_i = (1 + n_i) <u>_i \left(\frac{y}{h_i}\right)^{n_i},
\]

\[
\tau_b = u_b^2 (1 - y/h_b), \quad \tau_i = u_i^2 (1 - y/h_i).
\]

The detailed experiments of ice-covered flows in flume [Hanjalic & Launde (1972), Smith & Ettema (1994)] show that the shear stress reverses its sign at the horizon, which location depends on the ratio of roughness of the bottom and the underneath of ice. It was found that the horizons at which \( \tau_y = 0 \) and \( d\tau/dy = 0 \) do not coincide, and the discrepancy increases with the increase of the difference between the roughness of the boundaries [Hanjalic & Launde (1972)]. Therefore the structure of the flows in rivers and reservoirs becomes more complex due to presence of ice.
6. Division of a flow
In this approach the question arises as how to divide the flow. Processing of data of measurements of velocity profiles in ice-covered flow in the River Moskva shows that division of the flow by a plane going through the maximum velocity is hardly possible since there is a considerable layer of practically zero velocity gradient in the middle of the flow. Location of dynamic axes was found from the condition of smooth sewing of velocity profiles in each of the flows by varying $n_b$ and $n_i$. Comparison of calculations of mean parameters of the upper and lower flows with measurements shows adequacy of two-layer description of ice-covered flow [Dolgopolova, 2002]. However, such description gives $\bar{K}_y = 0$ at the boundary between lower and upper flows as a consequence of zero shear stress. At the same time kinetic energy in this area is not equal to zero, and at the interface of interaction of lower and upper layers formed by boundaries of different roughness the layer where $K_y \neq 0$ is developed. Thus it is reasonable to split an ice-covered flow into three layers: two boundary layers at the bottom and at the ice cover and an intermediate layer.

7. Three-layer description of ice covered flow
The idea of description of the ice-covered flow by multi-layer model was proposed by many authors [Debolskii et al. (1994), Kozlov (2000), Dolgopolova (1994)]. Solution of equations of motion for ice-covered flow presents mathematical difficulties [Zyryanov (2005), Prediction … (1980)] and does not give simple expressions for turbulent transfer coefficient. Because of this to calculate dimensionless turbulent transfer coefficient in boundary layers we use formulas (5, 5a),

**Figure 1.** Depth distribution of the vertical transfer coefficient calculated by (5a) and by Elder (1959).

**Figure 2.** Velocity profile measured in ice-covered flow (1) and calculated by three layer model (2).
and in the intermediate layer we estimate the value of $K_y$, with the help of (2) taking it following Shen & Harden (1978) as a constant.

It is known that roughness of the underneath of ice varies during winter, resulting in different roughness of ice and bottom. The velocity profile is asymmetric with respect to dynamic axes of ice-covered flow which was confirmed by experiments in the Rivers Moskva and Desna [Dolgopolova (1998), Debolskaya et al. (1999)]. The measurements show that roughness of ice is less than that of the bottom, and there exists an intermediate layer of interaction between boundary layers with weakly varied velocity of the flow.

8. Location of the intermediate layer
In the two-layer model, from the smooth sewing of power distributions of velocity of both lower and upper layers in 5 cross-sections of the river, were obtained location of dynamic axes and the power exponents $n_b$ and $n_i$ [Dolgopolova (1994), (2002)]. The dependence of the dimensionless thickness of the lower layer $h_b/h$ on the ratio of the Darcy-Weisbach coefficients $f_b$ and $f_i$ calculated for the lower and upper flows, obtained by Uzuner (1975), allows one to compare the magnitudes $h_b/h$, which were calculated from the velocity profiles with the help of the two layer model, with the calculated coefficients $f_b$ and $f_i$. Applying the expression \( f = 0.32 n^2 \) [Dolgopolova (2000)], which was found for a plane open flow, to the lower and upper layers of ice-covered flows we obtain $f_b/f_i \approx 0.6$, that following Uzuner (1975) gives $h_b/h \approx 0.6$. The same magnitude was calculated from the measurement and was accepted as a location of the middle of the intermediate layer. Analyzing mean velocity profiles of ice-covered flow in the River Moskva we found the thickness of this layer $s \sim 0.1 h$ and dimensionless vertical coordinates of the flows formed by bottom and ice are $y_b/h = 0.6 - s/2 = 0.55$ and $y_i/h = 0.6 + s/2 = 0.65$ correspondingly.

9. Velocity profile
As in two-layer model for velocity profiles in the currents formed by bottom and ice cover is used the power law (6). In the intermediate layer velocity distribution is described by the third order polynomial which coefficients are found by sewing velocities and their derivatives in the boundary layers with those at the boundaries of the intermediate stratum. Calculated velocity profiles shows satisfactory fit of measurement in ice-covered river flow (Figure 2), where standard deviation is presented $\sigma = 0.056$ m/s. Existence of the layer at which $du/dy \approx 0$ is confirmed by the results of measurements in ice-covered flow in flumes [Smith & Ettema (1994)] and in nature [Dolgopolova (1998)].

Average magnitudes of power exponents $n_b$ and $n_i$ in (6), calculated by velocity profiles measured in 5 cross sections in the River Moskva are $n_b = 0.22$, $n_i = 0.17$. The difference of power exponents for lower and upper flows indicates the asymmetry of ice-covered flow with respect to the plane passing though the middle of the depth.

10. Turbulent transfer coefficient
The depth distribution of the coefficient $K'_{int}$ in boundary layers formed by bottom and ice cover which was obtained by (5a) under the assumption of validity of linear and power laws for shear stress and mean longitudinal velocity in these layers is shown in Figure 3. The magnitude of $K'_{int}$
in the intermediate layer presented in Figure 3 was estimated by using the approach described below.

To estimate coefficient of turbulent momentum transfer $K_{int}$ in the intermediate layer of ice-covered flow we use expression (2). Kinetic energy of turbulence $k$ can be expressed by three components of velocity fluctuations as following:

$$k = \frac{u^2 + v^2 + w^2}{2}. \quad (7)$$

Because of lack of data on the measurements of three components of velocity fluctuations in rivers we use several approximations to calculate $k$ by (7). In the ice-covered flow in the River Moskva only fluctuations of longitudinal velocity component were measured. In the approximation of isotropic turbulence of intermediate layer of ice-covered flow one has $\sigma_u^2 \approx \sigma_v^2 \approx \sigma_w^2$ and $k \approx 3\sigma_u^2 / 2$. However this assumption is too strong for an ice-covered nature flow in which the depth is much smaller than the width and the roughness of bottom and ice are different, i.e. it is impossible to expect the symmetry of velocity fluctuations. This asymmetry is confirmed by results of measurements of velocity fluctuations in open river flows [Grinvald (1988)] which shows predominance of fluctuations of longitudinal velocity.

Let us estimate the ratio of standard deviation of three components of velocity fluctuations with the help of model expression for depth distribution of $\sigma_i$, developed for an open channel flow by Dolgopolova & Orlov (1989)

$$\sigma_i = a_i + b_i \cdot \sqrt{\frac{y}{h}}, \quad (8)$$

where $i = 1, 2, 3$ – longitudinal $x$, vertical $y$ and transverse $z$ coordinates and numerical coefficients are: $a_1=2.1, b_1= -1.2, a_2 = 1.3, b_2 = -0.6, a_3 = 1.7, b_3 = -1.0$.

Integration of (8) for the intermediate layer of ice-covered flow, which boundaries were determined above, gives the lower assessment as $\sigma_u^2 : \sigma_v^2 : \sigma_w^2 \approx 2.6:1:1$. Then for kinetic energy of turbulence we have

$$k = 0.88\sigma_u^2. \quad (9)$$

Expression (9) is in good agreement with the measurements of velocity fluctuations in the River Columbia from which the kinetic energy of turbulence was estimated by Grinvald (1988) as $k = 0.85\sigma_u^2$. In the following for calculation of coefficient $K_{int}$ by expression (2) we use the assessment (9), which gives for $k$ the magnitude approximately two times smaller than that following from the assumption of isotropic turbulence.

Energy dissipation in turbulent flow $\varepsilon$ can be estimated by the “macro” characteristics as [Landau & Lifshitz (1988), Prediction …(1980)]
\[ \varepsilon \approx \frac{(\Delta u)^3}{l}, \]  

(10)

where \( l \) is the characteristic scale of the distance at which the velocity of the flow considerably changes, \( \Delta u \) is the mean velocity variation.

Analysis of results of measurements of velocity profiles in the River Moskva shows that the velocity variations in the intermediate layer are about \( \Delta u \sim 5 \div 10 \% \) of the depth averaged velocity. Estimating \( \varepsilon \) by standard deviation of longitudinal velocity fluctuations instead of \( \Delta u \), Roberts & Webster (2002) found the same magnitude for numerator of (10).

The thickness of the intermediate layer \( s \) obtained from the measurements of velocity profiles and flow depth in the River Moskva varies in the range \( 8 \text{ cm} < s < 18 \text{ cm} \), and the averaged magnitude equals to \( <s> = 13 \text{ cm} \), that is \( 0.1h \), since the depth averaged across the zone of measurements \(<h> = 1.31 \text{ m} \). Using all the estimates and assuming \( \Delta u = 0.05 <u> \), one obtains from (10) the assessment for \( \varepsilon \)

\[ \varepsilon \approx \frac{(\Delta u)^3}{l} = 1.25 \cdot 10^{-3} \frac{<u>^3}{h}. \]  

(11)

At \( Sc = 1 \) using for \( k \) and \( \varepsilon \) expressions (9) and (11) we obtain for \( K_{\text{int}} \)

\[ K_{\text{int}}' = 7.29 \cdot 10^{-4} <u> h. \]  

(12)

At \( Sc = 0.5 \) we obtain for \( K_{\text{int}} \)

\[ K_{\text{int}}' = 1.46 \cdot 10^{-3} <u> h. \]  

(13)

In the intermediate layer dimensionless numerical coefficients in (12, 13) are the \( K_{\text{int}}' = 7.29 \cdot 10^{-4} \) and \( K_{\text{int}}'' = 1.46 \cdot 10^{-3} \). When constructing the graph in the Figure 3, the expression (12) was used for \( K_{\text{int}} \). There is a smooth sewing of the magnitudes of \( K_{\text{int}}' \) with those of \( K_{y}'' \) at the boundaries of the intermediate layer with the lower and upper flows. The product of dimensionless magnitudes of turbulent transfer coefficients for the boundary layers in ice-covered flow by \( n_b < u >_b h_b \) and \( n_i < u >_i h \) correspondingly gives the depth distribution of \( K_y \) (Figure 4). For the River Moskva the coefficient \( K_{\text{int}} = 4.4 \text{ cm}^2/\text{s} \), was calculated by (12).

Let us estimate depth averaged turbulent transfer coefficient of an ice-covered flow. The magnitude of \( K_{y}' \cdot \kappa = 2.64 \cdot 10^{-2} \) for the boundary layers at the bottom and underneath of ice is independent of the characteristics of a flow and can be considered as universal. For the ice-covered flow in the River Moskva we obtained mean transfer coefficients \( K_{b} = 1.55 \cdot 10^{-3} \text{ m}^2/\text{s} \)
and $K_i = 8.26 \cdot 10^{-4}$ m$^2$/s for the lower and upper layers correspondingly. For the intermediate layer we obtained at $Sc=1 - \overline{K_{in}} = 4.3 \cdot 10^{-3}$ m$^2$/s, at $Sc = 0.5 \overline{K_{in}} = 8.6 \cdot 10^{-3}$ m$^2$/s. Since the location of the dynamic axes in ice-covered flow is determined ambiguously, we used a conversion to the velocity averaged through the whole depth of the ice-covered flow, which is easily determined from the velocity profile.

Assuming $K_{in} = \text{const}$ in the intermediate layer [Shen & Harden (1978)] and mean characteristics of a flow are known, one can calculate turbulent transfer coefficient averaged through the depth of ice-covered flow, which for the flow in the River Moskva yields

$$\frac{1}{h_0} \int_0^h y K_y dy = 1.3 \cdot 10^{-3} \text{ m}^2/\text{s}.$$ 

From the data on velocity measurements in the River Moskva in summer it was obtained $\overline{K_y} = 2.64 \cdot 10^{-3} \text{ m}^2/\text{s}$ ($h = 1.5$ m, $n = 0.148$, $<u> = 0.4$ m/s). The assessments of coefficients $\overline{K_y}$ calculated for open and ice-covered flows in the River Moskva agree with those made by Shen & Harden (1978), who noted that depth averaged turbulent transfer coefficient of ice-covered flow with rough boundaries is about two times smaller than that in open flow.

**Figure 3.** Depth distribution of the dimensionless coefficient of turbulence momentum transfer $K_y'$.  
**Figure 4.** Coefficients $K_y$ calculated for ice-covered flows in the rivers Moskva (1) and Desna (2).
11. Application of calculation of turbulent transfer coefficient for ice covered river
To verify the method of calculation of turbulent transfer coefficient in an ice-covered flow we calculated $K_y$ for the ice-covered flow in the River Desna, where three components of velocity and velocity fluctuations were measured [Debolskaya et al. (1999)]. The relevant characteristics of the flow are $n_b=0.28$, $n_i=0.2$, $h = 0.88$, $<u> = 0.385$. From the ratio of power exponents in the boundary layers formed by bottom and ice the location of the middle of the intermediate layer at $y =0.65h$ was found. As above the thickness of this layer was taken $s=0.1h$. The calculation of $K_y$ was made with the help of mean magnitudes of $<u>_b$, $<u>_i$, $h_b$, $h_i$ for lower and upper layers correspondingly. The shape of depth distribution of $K_y$ presented in the Figure 4 is in good correspondence with that suggested by Debolskaya & Zyryanov (1994). The magnitude of $K_y=1.0 \cdot 10^{-3}$ is in good correspondence with that obtained for the River Moskva.

12. Conclusions
The use of power law for description of velocity profile allows us to take into account incomplete self-similarity of shear flow with respect to global Reynolds number. On the basis of the three-layer model of an ice-covered flow, the method for assessment of vertical coefficient of turbulent diffusion is developed. Calculated values of the coefficient of vertical diffusion are in good correspondence with those obtained by Elder (1959), Fischer (1973), Jobson & Sayre (1970). The distributions of dimensionless universal coefficients $K_y'$ through the depth of ice-covered flow were obtained. Suggested expressions (5), (5a) and (12) enable one to estimate the depth distribution and depth averaged magnitude of vertical transfer coefficient of the ice-covered flow by the minimum of experimental data available. The verification of the model by calculation of coefficient of turbulent diffusion for the ice-covered river flow shows reasonable results.

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Mathematical Modeling of the Ice-Thermal Regime of Water Body

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Two-dimensional model that describes snow-ice cover growth in mineralized water is developed. As a result of salt-free ice increasing before the front of crystallization a layer with higher mineralization and of variable density is formed. The natural convection effect on downward freezing of water is observed. All phenomena listed above affects of the freezing-point and of the speed of freezing water and ice growth process. The mathematical statement based on Stefan's problem. Task comes to the solution of the Stokes problem of Bussinesk’s approximation in water body and the equations of heat conductivity for temperature conditions in ice and snow layers. The front rectification method allowing setting the equations in a regular domain is used.

There are three adjacent areas with desired moving boundaries (figure 1).

\[ \rho_w C_p w \frac{\partial T_w}{\partial t} + u \frac{\partial T_w}{\partial x} + w \frac{\partial T_w}{\partial y} = \frac{\partial}{\partial x} \left( k_x \frac{\partial T_w}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial T_w}{\partial y} \right), \]  

\[ \frac{\partial C}{\partial t} + u \frac{\partial C}{\partial x} + w \frac{\partial C}{\partial y} = \frac{\partial}{\partial x} \left( d_x \frac{\partial C}{\partial x} \right) + \frac{\partial}{\partial y} \left( d_y \frac{\partial C}{\partial y} \right), \]  

\[ \frac{\partial \omega}{\partial t} + u \frac{\partial \omega}{\partial x} + w \frac{\partial \omega}{\partial y} = \frac{\partial}{\partial x} (v_x \frac{\partial \omega}{\partial x}) + \frac{\partial}{\partial y} (v_y \frac{\partial \omega}{\partial y}) + g \frac{\partial}{\partial x} \left( \beta_T T_w + \beta_C C \right), \]  

\[ \Delta \psi + \omega = 0; \quad u = \frac{\partial \psi}{\partial y}, \quad w = -\frac{\partial \psi}{\partial x}; \]  

\[ \beta_f = -\frac{1}{\rho_w} \left( \frac{\partial \rho_w}{\partial T_w} \right); \quad \beta_C = -\frac{1}{\rho_w} \left( \frac{\partial \rho_w}{\partial C} \right), \]

where unknown functions and physical characteristics: \( T(t,x,y) \) – the temperature (water, ice, snow), \( C(t,x,y) \) - the salinity, \( g/dm^3 \); \( u, w \) - the corresponding velocity components, \( \omega(t,x,y) \) - the eddy function, \( \psi(t,x,y) \) - the stream function, \( \rho_{ic}, \rho_{sn}(t,y), \rho_w(T, C) \) – the density of ice, snow and water, respectively, \( kg/m \); \( \lambda \) - the thermal conductivity, \( W/m \); \( k_x, d_x, v_x \) the longitudinal turbulent thermal conductivity, diffusivity and viscosity, respectively, \( W/m, m^2/s \); \( k_y, d_y, v_y \) the vertical turbulent thermal conductivity, diffusivity and viscosity, respectively, by analogy with Blumberg (1977); \( c_p \) the specific heat conductivity, \( J/kg \ {}^\circ C \); \( g \) – the acceleration of gravity.

2. Boundary conditions

2.1) external boundary of water body:

\[ \left. \frac{\partial T_w}{\partial n} \right|_{x=0} = 0; \quad \left. \frac{\partial C}{\partial n} \right|_{x=0} = 0; \quad \left. \frac{\partial T_{ic}}{\partial n} \right|_{x=0} = 0; \quad \left. \frac{\partial T_{sn}}{\partial n} \right|_{x=0} = 0; \]

\[ \left. \frac{\partial T_w}{\partial n} \right|_{y=0} = 0; \quad \left. \frac{\partial T_w}{\partial n} \right|_{y=Depth} = 0; \quad \left. \frac{\partial T_{ic}}{\partial n} \right|_{y=0} = 0; \quad \left. \frac{\partial T_{ic}}{\partial n} \right|_{y=Depth} = 0; \quad \left. \frac{\partial T_{sn}}{\partial n} \right|_{y=0} = 0; \quad \left. \frac{\partial T_{sn}}{\partial n} \right|_{y=Depth} = 0; \]
\[
\frac{\partial T_w}{\partial n} \bigg|_{y=\text{Depth}} = 0 \quad \text{or} \quad T_w \big|_{y=\text{Depth}} = \text{const}, \quad \frac{\partial T_w}{\partial n} \bigg|_{y=\text{Depth}} = F(T), \tag{7}
\]

\[
\psi \big|_\Gamma = \frac{\partial \psi}{\partial n} \bigg|_\Gamma = 0, \quad \Gamma = \{x = 0, x = \text{Width}, y = s(t,x), y = \text{Depth}\}; \tag{8}
\]

where the \textit{const} is a known temperature of ground and \( F \) is assigned heat flow, \( n \) – normal vector to \( \Gamma \).

2.2) at the “water-ice” moving boundary (interface) \( y = s(t,x) \) is defined Stefan’s condition and equation of mass balance:

\[
V_s = \frac{\partial s}{\partial t}; \tag{9}
\]

\[
\lambda_{ic} \frac{\partial T_{ic}}{\partial n} \bigg|_{y=s(t,x)} - k_y \frac{\partial T_w}{\partial n} \bigg|_{y=s(t,x)} = L \rho_w V_s; \tag{10}
\]

\[
V_s C_s = -d_y \frac{\partial C}{\partial n} \bigg|_{y=s(t,x)}, \tag{11}
\]

\[
T_w \big|_{y=s(t,x)} = T_{ic} \big|_{y=s(t,x)} = T_s
\]

\[
T_s = T^* - \gamma C_s \tag{12}
\]

where \( L \) the specific heat of freezing; \( T_s, C_s \) the temperature and solinity on moving boundary (freezing-point), \( T^* \) the ice point of sweet water (for sweet water \( T_s = 0^\circ C, C = 0 \text{ g/dm}^3 \)); \( \gamma \) the equilibrium coefficient.

2.3) at the “ice-snow” interface \( y = \theta \):

\[
\lambda_{ic} \frac{\partial T_{ic}}{\partial n} \bigg|_{y=0} = \lambda_{sn} \frac{\partial T_{sn}}{\partial n} \bigg|_{y=0}; \tag{13}
\]

\[
T_{sn} \big|_{y=0} = T_{ic} \big|_{y=0}; \tag{14}
\]

2.4) at the “snow-air” interface \( y = -l_{sn}(t,x) \):

\[
T_{sn} \big|_{y=-l(t,x)} = T_a(t,x), \tag{15}
\]

where \( T_a \) is the air temperature.

As initial conditions at \( t = 0 \) the positions of all moving boundaries and vertical heat distribution are specified.
3. Method of solution

3.1) The front rectification method, allowing the numerical solution of the mentioned equations set in the regular domain is used.

\[0 \leq \xi \leq \text{Width}; \quad 0 \leq \eta_i \leq 1, \ (i = w, ic, sn);\]

\[\eta_{ic} = \frac{y}{s(t, x)}; \quad \eta_{sn} = -\frac{y}{l_{sn}};\]

\[\eta_w = \frac{l_y - y}{l_w};\]

The positions of moving boundaries is defined by the change of phase equations and mass conservation law

\[l_w (t, x) = \text{Depth} - s(t, x) \quad \text{is the layer of water};\]

\[l_{ic}(t, x) = s(t, x) \quad \text{is the thickness of ice cover}\]

\[l_{sn} (t, x) = \ln(1+bl^*)/b \quad \text{is the depth of snow cover taking into account the shrinkage of snow};\]

\[l^* = 0.001W \frac{p_{\text{water}}}{\rho^*};\]

Figure 2. Transformation of the definitional domain

*Width* is the water body width, *m*; *Depth* is the water body depth, *m*; \(\rho^*\) - the density of new snow, \(\text{kg/m}^3\); \(l^* (t, x)\) is the thickness of freshly fallen snow (*m*) with density \(\rho^*\); \(b=1.25\); \(W\) is the water equivalent of snow, \(\text{mm}\).

3.2) Basic equations in the new variables:

\[
\frac{\partial T_{sn}}{\partial t} + a_{2sn} \frac{\partial T_{sn}}{\partial \eta} = \chi_{sn} \dot{\Delta}T_{sn} ;
\]

\[
\frac{\partial T_{ic}}{\partial t} + a_{2ic} \frac{\partial T_{ic}}{\partial \eta} = \chi_{ic} \dot{\Delta}T_{ic} ;
\]

\[
\frac{\partial T_w}{\partial t} + a_{1w} \frac{\partial T_w}{\partial \xi} + a_{2w} \frac{\partial T_w}{\partial \eta} = \dot{\Delta}T_w ;
\]

\[
\frac{\partial C}{\partial t} + a_{1c} \frac{\partial C}{\partial \xi} + a_{2c} \frac{\partial C}{\partial \eta} = \dot{\Delta}C ;
\]

\[
\frac{\partial \omega}{\partial t} + a_{1\omega} \frac{\partial \omega}{\partial \xi} + a_{2\omega} \frac{\partial \omega}{\partial \eta} = \dot{\Delta} \omega + \dot{f} ;
\]

\[\dot{\Delta} \psi + \omega = 0; \quad u = -\frac{\eta}{l_w} \frac{\partial \psi}{\partial \eta}; \quad w = -\frac{\partial \psi}{l_w \frac{\partial \xi}{\partial \xi}} + u \frac{\partial s}{\partial \xi} ;\]
\( \chi = \frac{\lambda}{c_p \rho} \) - the thermal conductivity, \( m^2/s \);

\[ \tilde{\Delta} = a_{11} \frac{\partial^2}{\partial \xi^2} + a_{12} \frac{\partial^2}{\partial \xi \partial \eta} + a_{22} \frac{\partial^2}{\partial \eta^2}. \]

\[ a_{2_{sn}} = \frac{1}{l_{sn}} \frac{\partial \lambda_{sn}}{\partial \eta} - \frac{\eta}{l_{sn}} \frac{\partial l_{sn}}{\partial t}; \]

\[ a_{2_{ic}} = -\frac{\eta}{s} \frac{\partial s}{\partial t}; \]

\[ a_{2_{w}} = \frac{\eta}{l_w} \frac{\partial s}{\partial t} + \frac{u \eta}{l_w} \frac{\partial s}{\partial \xi} - \frac{w}{l_w} - \frac{1}{Cp \rho l_w^2} \frac{\partial k_y}{\partial \eta}; \]

\[ a_{2_{c}} = \frac{\eta}{l_w} \frac{\partial s}{\partial t} + \frac{u \eta}{l_w} \frac{\partial s}{\partial \xi} - \frac{w}{l_w} - \frac{1}{l_w^2} \frac{\partial d_y}{\partial \eta}; \]

\[ a_{2_{\omega}} = \frac{\eta}{l_w} \frac{\partial s}{\partial t} + \frac{u \eta}{l_w} \frac{\partial s}{\partial \xi} - \frac{w}{l_w} - \frac{1}{l_w^2} \frac{\partial v_y}{\partial \eta}; \]

\[ a_{11_w} = \frac{k_w}{Cp \rho l_w}; \quad a_{11_c} = d_x; \quad a_{11_{\omega}} = v_x; \]

for following functions \( T_w, C, \omega, \psi \):

\[ a_1 = u; \quad a_{12} = \frac{2 \eta}{l_w} \frac{\partial s}{\partial \xi}; \quad a_{22} = \left( \frac{\eta}{l_w} \frac{\partial s}{\partial \xi} \right)^2 + \left( \frac{1}{l_w} \right)^2; \]

for following functions \( T_{ic}, T_{sn} \):

\[ a_{11} = 1; \quad a_{12} = -\frac{2 \eta}{l_{ic,sn}} \frac{\partial s}{\partial \xi}; \quad a_{22} = \left( \frac{\eta}{l_{ic,sn}} \frac{\partial s}{\partial \xi} \right)^2 + \left( \frac{1}{l_{ic,sn}} \right)^2. \]

Boundary conditions also we shall reset in new coordinates ) Coupling conditions in the new variables. For example Eq. [13]:

\[ \left. \lambda_{ic} \frac{\partial T}{\partial \eta} \right|_{\eta=0} = \left. -\frac{\lambda_{sn}}{l_{sn}} \frac{\partial T_{sn}}{\partial \eta} \right|_{\eta=1}; \]

4. Numerical solution

4.1) We integrate system [16]-[20] using standard alternating-direction schemes.
\[ U^n = \{ T_{sn}, T_{ic}, T_w, C, \omega \} \]

\[ B_1 B_2 \frac{U^{n+1} - U^n}{\tau} = K^n \Lambda U^n - a_1^n \frac{\partial U^n}{\partial \xi} - a_2^n \frac{\partial U^n}{\partial \eta}; \]

\[ \begin{aligned}
    B_1 &= E - K_1 \frac{\tau}{2} \Lambda_1 \tilde{\alpha}; \\
    B_2 &= E - K_2 \frac{\tau}{2} \Lambda_2 \tilde{\beta}; \\
\end{aligned} \]

\[ \Lambda_{11} = \frac{\partial^2}{\partial \xi^2}; \quad \Lambda_{12} = \frac{\partial^2}{\partial \xi \partial \eta}; \quad \Lambda_{22} = \frac{\partial^2}{\partial \eta^2}; \]

step 1  \( E - K_1 \frac{\tau}{2} \Lambda_{11} \tilde{\alpha} \) \( \bar{U} = F(U^n) \)

step 2  \( E - K_2 \frac{\tau}{2} \Lambda_{22} \tilde{\beta} \) \( U^{n+1} = \bar{U}; \)

\( \tau \) – is the time step and, \( n \) – layer, \( K, K_1, K_2 \) the respective coefficients \( \tilde{\alpha}, \tilde{\beta} \) characteristic of factorization. The problem is numerically realized by the so called contradirectional sweep method. Value of function \( U \) on external borders is determined from [7] - [15].

4.2 The temperature on border between an ice and a snow is unknown. We define entry conditions of the beginning of the account and we find the unknowns \( \alpha_{ic1}, \beta_{ic1} \) through difference form Eq. [13], [14]

\[ T_{sn,i,N_2} = \alpha_{sn} T_{sn,i,N_2+1} + \beta_{sn} N_2; \quad T_{ic,i,1} = \alpha_{ic1} T_{ic,i,2} + \beta_{ic1} \]

\[ \alpha_{ic1} = \frac{A}{1 - \alpha_{sn} N_2 + A}; \quad \beta_{ic1} = \frac{-\beta_{sn} N_2}{1 - \alpha_{sn} N_2 + A}; \quad A = \frac{k_{ic} \bar{l}_{sn} \bar{h}_{sn}}{k_{sn} \bar{s}_{ic}}. \]

4.3) We calculate temperature of freezing \( T_s \) [9]-[12] by Voyevodin, A.F.and Grankina, T.B. (2006). The problem is numerically realized by the so called contradirectional sweep method.

\[ \gamma A_T C_s^2 + (A_c - B_T) C_s + B_c = 0; \]

\[ C_f = \frac{B_T - A_c + \sqrt{(A_c - B_T)^2 - 4 \gamma A_T B_c}}{2 \gamma A_T}, \]

\[ A_c = \frac{d_T}{\sqrt{1 + \delta s}} \frac{1 - \alpha_{cs1}}{l_w h_w}, \quad B_c = \frac{d_T}{\sqrt{1 + \delta s}} \left[ \frac{\delta s (C_{i+1s} - C_{i-1s})}{2 h} - \frac{\beta_{cs1} (\delta s + 1)}{l_w h_w} \right], \]

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\[
A_T = \sqrt{\delta s + 1} \frac{\lambda_{ic} (1 - \alpha_w^{n+1}) + k_j (1 - \alpha_{w_{N_2}}^{n+1})}{sh_{ic} l_w h_w},
\]
\[
B_T = \frac{1}{L \rho_{ic} \sqrt{1 + \delta s}} \left[ \delta s \left( \frac{T_{i+1,s}^n - T_{i-1,s}^n}{2h} \right) (k_x - \lambda_{ic}) - (\delta s + 1) \left( \frac{\lambda_{ic} \beta_{w_{N_2}}^{n+1} + k_j \alpha_w^{n+1}}{sh_{ic} l_w h_w} \right) \right];
\]
\[
\delta s = \left( \frac{s_{i+1} - s_{j-1}}{2h} \right)^2.
\]

\(\alpha_{w_s, is, w_s}, \beta_{w_s, ic, w_s}\) the sweep coefficients, \(h\) – the space step of the difference grid to \(\xi\), and \(\xi_i = i \bar{h}\); \(h = 1/N, N = \{N_1, N_2\}, h = \{h_w, h_{ic}\}\), \(i = 1, \ldots, N_1+1, j = 1, \ldots, N_2+1\).


In short, the integration of [21] can be approximated by following difference scheme

\[
\frac{\partial \omega}{\partial t} + G \omega = D \omega + \tilde{f}
\]

[24]

Where \(G\) is the convective translation operator, \(D\) is the diffusion translation operator. The operator \(G\) is put in the form of product of two one-dimensional difference operators, that is, \(G = G_1 G_2 = G_2 G_1\), \(G \omega = G_1 \omega + G_2 \omega\), where:

\[
G_1 \omega = \frac{1}{2} \left( a_{1_{\omega}} \frac{\partial \omega}{\partial \xi} + a_{1_{\omega}} \frac{\partial \omega}{\partial \xi} \right), \quad G_2 \omega = \frac{1}{2} \left( a_{2_{\omega}} \frac{\partial \omega}{\partial \eta} + a_{2_{\omega}} \frac{\partial \omega}{\partial \eta} \right);
\]

We solve the problem [24] in two steps.

Step 1

\[
\frac{\partial \omega}{\partial t} + G \omega = 0;
\]

[25]


Step 2

\[
\frac{\partial \omega}{\partial t} = \tilde{\Delta} \omega + \tilde{f};
\]

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we solve involving the equation [22] and using a five-point scheme. Boundary conditions [8] from $\omega$ are approximated by formulae analogous to Tom’s formula (Roache P.J., 1972):

$$\omega_{i,j} = -2a_{22} \frac{\psi_{i,2}}{h_w^2}; \quad \omega_{i,N_{i+1}} = -2a_{22} \frac{\psi_{i,N_i}}{h_w^2};$$

$$\omega_{i,j} = -2a_{11} \frac{\psi_{i,2}}{h^2}; \quad \omega_{N_{i+1}, j} = -2a_{11} \frac{\psi_{N_i,2}}{h^2}.$$

For calculation of values $\psi^{n+1}$ approximation of the Eq. [22] by means of the scheme [23] is used (Voyevodin A.F.(1993)).

### 5. Examples of calculations.

Object of calculation the saline Yarcul by mineralization 5 g/dm$^3$ of complex lake Chany served. On figure 1 and figure 2 results of numerical simulation of dynamics of growth of a snow-ice cover of saline Yarcul and natural data are presented.

![Figure 3](image)

**Figure 3.** The dynamics of ice cover thickness growth in saline Yarcul from period 9 November 1999 – 29 February 2000. Black circle is the observational data.
Figure 4. The dynamics of ice cover thickness growth in saline Yarcul from period 24 November 2002 – 25 March 2003. Black circle is the observational data.

References


Modeling Grounded Ice Jams using the Ice Jam Force Balance Equation

ABSTRACT

It is well known that wide river ice jams can become grounded, especially near the downstream end, or toe, of the jam. The geometry of wide rivers jams has been successfully modeled by solving an ice jam force balance equation assuming that the ice flows comprising the jam act as a granular material. However, to date, the grounded portions of wide river jams have been explicitly excluded from the ice jam models. This exclusion means that stresses acting through the ice in the toe region and flow through the grounded portion are both neglected. This neglect comes with a prior ad hoc assumptions regarding the source of jam stability in the toe, as well as an incorrect view of the jam as a collection of jam fragments moving and deforming at a uniform velocity. The correct way to view the maximum velocity criterion in HEC-RAS is that it is an upper bound on the maximum velocity under the jam. Unfortunately, where ever the maximum velocity criteria is met, the ice jam force balance equation not being solved at any point along the channel! This results in contradictory, arbitrary, confusing, and probably inaccurate results.

Background

Flato and Gerard (1986) pioneered modeling of wide river jams through alternating solutions of the closely coupled granular material ice jam force balanced equation and the steady flow equation. They gave “special consideration” to the toe of the jam, where they assumed that equation of the force balance equation and replaced it with the assumption that the jam thickness would be determined by the erosion velocity of the ice. The solution in the toe coupled the erosion velocity criteria with the steady flow equation rather than coupling the ice jam force balance equation with the steady flow equation. The Flato and Gerard’s model was not valid in the toe. Unfortunately, a certain amount of confusion has been created regarding the proper way to model the erosion velocity under wide river jams.

Figure 1 from Flato and Gerard (1986) Note the flow area maintained under the toe of the jam.

In fairness to Flato and Gerard, no granular models of river ice jams developed to date can simulate grounded ice jams. This is because

Vertical Stress in a grounded jam

Vertical Stress Distribution

Average Vertical Stress

Define: 

\[ \sigma = \frac{1}{2} \left[ \left( \frac{1}{\rho_p} \right) \frac{F_a}{A_s} \right] \]

\[ \frac{J}{1} \]

Solution Procedure

HEC-RAS Ice Cover Editor: Box to Enter maximum velocity criterion

Steady Flow Equation

General Solution Procedure

Summary

References


Analysis of the 2007 Montpelier, VT Ice Cover using High-Resolution Satellite Data

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A variety of remote sensing techniques have been used to analyze river ice conditions and river ice jams using satellite imagery, aerial photography, and web cameras. Satellite imagery provides large-scale images displaying the entire length of ice-covered river reaches but often at poor spatial resolution. Recently, multiple image dates were acquired at an in-place ice cover on the Winooski River at Montpelier, VT by the QuickBird satellite. QuickBird is a high-resolution commercial satellite which collects panchromatic (black & white) imagery at 60 cm resolution and multispectral imagery at 2.4 meter resolution. The images were acquired on March 9 and March 22, 2007. During this time, the City of Montpelier was threatened by flooding resulting from the breakup of the ice cover and the formation of ice jams. The research seeks to examine the capability of using high-resolution QuickBird imagery to classify river ice types and determine ice cover characteristics. Multiple spatial and spectral analyses were performed to quantify the variation in ice conditions. Processing techniques included pre-sharpening each image (using the panchromatic band with the multispectral bands to create a high-resolution multispectral image) and subsetting to the extent of the river channel banks and bridges. Texture and edge detection algorithms were used to characterize the spectral variation in rough ice conditions and classify the structural characteristics of river ice cover. Results show these techniques can be successfully used to identify variations in river ice cover characteristics and could provide an additional tool for monitoring river ice conditions in high hazard areas.

### Change Detection Methodology

- Co-registration of both image dates to UTMs
- Pan-Sharpening: Each image was pan-sharpened by fusing the 2.4-meter Multispectral Image (MSI) with the 0.6-meter panchromatic image using a Boxcar Transform. This produced sub-meter radiometrically calibrated sub-meter multispectral images.
- Image Normalization: The pan-sharpened images were normalized using an empirical line calibration process. This process converts the band-wise Digital Number (DN) values of one image to the calibration units of the other image. Band-wise correlation coefficients were greater than 0.975, ensuring consistency in the radiometric calibration between the two images.
- Image Subtraction: A simple image differencing analysis was used to produce the raw change image. The "before image" 09 March 2007 was subtracted from the "after image" 22 March 2007.
- Final change detection map was produced by setting a change threshold of ± 2 standard deviations of the mean. Pixels outside this change threshold were mapped as significant change and are highlighted in the map.

### Change Detection Discussion

The change detection map developed using these methodologies highlights the utility of high-resolution satellite imagery for assessing river ice conditions. A set of standardized tools was used to properly co-register, calibrate and differentiate the image pairs. A user-defined change threshold was set to highlight only those areas of the river which had undergone significant change in ice conditions. An analysis of the results confirms that the change detection method accurately identified changes in ice conditions between image dates. These results suggest that high resolution satellite imagery can be used to assess changes in ice conditions and possibly help in forecasting ice jam events.

### Summary

The Winooski River in Montpelier, VT has a history of flooding resulting from ice jams (Tuthill et al 1996). These ice jam events have lead to significant damage in the past. During the winter of 2007-07 significant ice formed in the Winooski River, raising concerns of another ice jam event. The river was heavily monitored during the winter season. The monitoring included the acquisition of two high resolution satellite images on and 22 March 2007. After a period of cold air temperatures a low steady flow, the river experienced a period of above freezing air temperatures and high flow. These conditions initiated a period of river ice deterioration and melt.

The images were processed to determine changes in the ice cover between the two dates and to classify the ice cover. The 09 March 2007 image was not classified because of the uniformity of the ice cover at that time. The 22 March 2007 was classified and four classifications were chosen: Open Water, Ponded Water, Solid Ice and Unconsolidated Ice.

The significant changes mapped between the two images show areas that changed from solid ice cover to one of the other classifications. This change is concentrated at several distinct locations, upstream and downstream of the dam and along the channel. The areas of significant change are highlighted in the map. The classification map indicates the heterogeneity of ice types and their distribution. The 22 March image highlights the complexity of the process of breakup in the reach.

### References
