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BEHAVIOR OF ICE COVERS SUBJECT TO LARGE DAILY FLOW AND LEVEL FLUCTUATIONS

ACRES CONSULTING SERVICES LIMITED

RESEARCH

REPORT

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REPORT FOR THE

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ABSTRACT

A research study was conducted to determine the limits to water level variation to maintain stability of solid ice covers in reservoirs, lakes and rivers.

Based on a literature survey of relevant European, Russian and North American publications, it was concluded that there is no established engineering capability in this area. Although the serious consequences of solid ice cover breakup are appreciated, the available documentation of solid ice cover breakup events is qualitative. Thus, the available data are inadequate to develop necessary assumptions for detailed analysis of this phenomenon.

Existing theories for ice cover stability under steady flow conditions have been evaluated for their application to variable water level conditions. A preliminary analysis of the development of stable hinges connecting the solid ice cover to the shoreline is presented. Recommendations are made for laboratory and field studies to gather appropriate data to calibrate these analyses, and to observe the phenomenon in order to make assumptions necessary for further analytical refinements.

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A = cross-sectional flow area (m^2)

Al = ice albedo (average value 0.25)

$A_{o/w}$ = supercooled open-water generation area (m^2)

a = an exponent in the Reynold's analogy

a = an empirical coefficient

B = channel (top) width (m)

B_i = total border ice growth (m)

b = an empirical coefficient

C = Chezy roughness coefficient for water passage, assumed equal for riverbed and underside of ice cover

C_p = specific heat of water (J/kg°C)

C_s = insolation coefficient (Table 3.1) which can be set to zero for overnight heat loss calculation (J/m^2 day)

c = an experimental coefficient

D = accumulated degree-days of freezing (°C-days)

d_b = trashrack bar diameter (m)

d_s = characteristic dimension of ice fragments taken as floe thickness or slush blanket thickness (m)

E = the elastic modulus of ice (Pa)

e = relative humidity of air

Fr_{cr} = critical Froude number for ice cover progression

Fr_{it} = densimetric Froude number at incipient under-ice block transport

Fr' = densimetric Froude number

f = friction coefficient

f_D = Darcy-Weisback friction factor

f_w = wind drag on the upper surface of the cover (Pa) (positive if acting in the downstream direction)

g = acceleration due to gravity (m/s^2)

H = mean upstream open-water flow depth (m)

H_o = summer open-water flow depth (m)

h_e = convective heat transfer coefficient from flow to ice cover (J/m^2 s °C)

i_s = water equivalent of snow precipitation (m/day)

J = thermal equivalent = 1 N-m/J

K_i = roughness height of ice (m)

k = coefficient of heat transfer from flow to atmosphere representative of site conditions (J/m^2 day °C)

k_a = thermal conductivity of air (J/m^2 day °C)

k_c = conduction coefficient (J/m s °C)

k_f = thermal conductivity of the fluid (J/m s °C)

k_w = thermal conductivity of water (J/m day °C)

L_f = latent heat of fusion of ice (J/kg)

ΔL = progression rate of a packed ice cover (m/day)

ℓ = hinge link length (m)

ℓ = a characteristic length (m)

ℓ, ℓ_o = river location along flow axis (m)

ℓ_b = length of ice block (m)

ℓ_i, ℓ_b = upper (ice) and lower (bed) flow layer depths

m = an adhesive parameter

$\frac{m}{N}$ = fraction of hours of clear sky during daylight hours

N_b = number of boundaries

N_i = surface area concentration of slush pans (m^2/m^2)

N_u = $h_e\, \ell/k_f$ = Nusselt Number

n = an experimental coefficient

n_i = Manning's n-value under the ice cover

n_b = Manning's n-value of the channel bed

n_c = composite roughness with ice cover

P = power per unit area (W/m^2)

Pr = $C_p \mu/k_f$ = Prandtl Number (13.6 for water at 0°C)

Q = river discharge (m^3/s)

Q_{ex} = external artificial addition of heat (J/m^2 day)

Q_i = volume rate of ice production (m^3/day)

Q_s = surface ice transport rate (m^3/s)

Q_u = total under-ice transport (m^3/s)

Q* = rate of total heat transfer to atmosphere per unit surface area (J/m^2 day)

ΣQ* = sum of net heat transferred (J/m^2)

q = river discharge per unit width (m^3/s m)

$\dot{q}_c$ = rate of heat transfer through a solid cover (J/m^2 s)

q_i = rate of ice production per unit area (m^3/s m^2)

q_f = heat from internal friction (J/m^2 s)

q_g = groundwater discharge per unit river length (m^3/s m)

q_t = terrestrial heat flow (J/m^2 s)

q_u = ice discharge per unit width under cover weighed under water with apparent density 0.08 (N/s m)

q_w = heat available from cooling flow (J/m^2 s)

Re = $V_u \ell \rho / \mu$ = Reynolds Number

R_H = hydraulic radius of the ice-covered flow (m)

R_i = hydraulic radius of upper flow layer (m)

S = slope of the hydraulic grade line (equal to slope of energy gradient and bed slope for uniform flow)

S_b = spacing between trashrack bars (m)

S_t = total solar radiation

S_r = solar radiation absorbed per unit ice volume

S_o = critical absorbed solar radiation for complete loss of strength

T_a = air temperature (°C)

T_f = freezing temperature of water (0°C)

T_o = initial water temperature at location ℓ_o (°C)

T_s = upper ice surface temperature (0°C)

T_w = water temperature at location ℓ (°C)

Th = cumulative degree-days of thawing

ΔT_e = equilibrium flow temperature difference from freezing (°C)

ΔT = air temperature difference from water freezing temperature (°C)

ΔT_g = groundwater temperature difference from flow (°C)

t = equilibrium ice cover thickness (m)

t_e = thickness of fragmented uniform ice cover (m)

t_i = ice cover thickness (m)

t_s = thickness of slush accumulation (m)

Δt_t = thickness of thermal ice grown in water voids (m)

U = wind speed (m/s)

U_{15} = wind velocity 15 m above the water surface (m/s)

V = mean flow velocity under open water conditions upstream of the ice cover (m/s)

V_b = water velocity past bars (m/s)

V_{cr} = mean critical upstream velocity of flow (m/s)

V_s = surface flow velocity (m/s)

V_u = mean velocity under the ice cover (m/s)

W = height of wind velocity, U, above ice surface (m)

X = length of open water reach (m)

x = distance downstream from start of supercooling (m)

x' = distance travelled by slush pan (m)

$\bar{\Delta}y$ = water level variation

z = horizontal factor for slope of riverbank corresponding to unit vertical factor

α = thermal coefficient for Stefan Equation of thermal ice growth

α_a = maximum hinge link angle limited by geometry

α_{max} = the limiting angle of the hinge link from horizontal for joint stability

γ_i = specific weight of ice (N/m^3)

γ = specific weight of water (N/m^3)

ε = porosity of slush accumulation (m^3/m^3)

μ = absolute viscosity ($N\ s/m^2$)

μ = coefficient related to ice cover friction and internal resistance of the cover

μ_h = a frictional or contact coefficient reflecting the strength of the fractured and rotating hinge joint

μ_{h1} = the friction coefficient at the higher hinge joint

μ_{h2} = the friction coefficient at the lower hinge joint

μ_M = coefficient in Michel's ice jam stability analysis typically equal to 0.26

ξ = coefficient determined by thermal growth relative to slush pan thickness

ρ = specific mass of water (kg/m^3)

ρ' = specific mass of ice (kg/m^3)

ρ_a = air density (kg/m^3)

σ_b = bending strength of the ice cover (Pa)

σ_{bs} = ice cover bending strength after exposure to solar radiation (Pa)

$\sigma_{\dot{o}}$ = ice cover strength at freezing/thawing temperature without exposure to solar radiation

τ = cohesive strength of fragmented ice pack (Pa)

τ_i = ice shear strength

τ_o = ice shear strength at 0°C without thawing (normally taken as 500 to 700 kPa)

χ, ψ = stress relaxation functions for the ice cover failure at the shoreline

EXECUTIVE SUMMARY

This report presents the results of the first phase of a research study of the stability of solid ice covers subjected to varying water levels and discharges, conducted for the Canadian Electrical Association under Research Contract 000-G-138.

An outline of data collection and field study requirements for evaluation of thermal and mechanical ice processes which govern the ice regime is presented in the Generalized Approach to Ice Studies. Because of the number of processes involved and the complex interrelationships between processes, there is no unique methodology for ice studies. Thus, a thorough understanding of all ice processes is necessary in order to identify critical processes governing the ice regime.

Application of existing criteria for mechanical stability of fragmented ice covers and thermal regime calculations, developed for steady flow conditions, to variable flows is described. It is possible to analyze and predict the response of an ice cover to water level and flow velocity variation prior to consolidation. Guidelines to limit flow variation can be established from these criteria.

This study involved a literature review and a series of interviews with hydroelectric power utilities to establish the state-of-the-art for the specific ice engineering problem of solid ice cover stability in rivers, reservoirs and lakes. The breakup of solid ice covers generally results in severe ice jams, flooding and obstruction to flow. Although the problem of solid ice cover stability is widely recognized, a coordinated effort to establish analytical criteria has not been made. Solutions are devised independently at each site based on trial-by-error operation or experience. No detailed

observations or measurements have been collected which could be used to guide and calibrate the analysis of solid ice cover stability.

A preliminary analysis of the stability of a hinged contact between the solid ice cover and shoreline was determined from the basic concept proposed by Fonseca (1979). A number of strips of ice, formed at the initial failure of the ice cover, act as linking members in an inverted arch which connects the solid central section of the ice cover to the shoreline. With this support, shearing forces of flow and wind on the ice cover are transferred to the riverbanks.

Criteria for critical water level variation for hinge stability are based on a simplified analysis of the mechanical and geometrical stability of hinge links throughout a water level cycle.

The lack of quantitative data on solid ice cover stability makes the simplified approach necessary at this time. A comprehensive field and laboratory program is recommended to calibrate the criteria developed, observe hinge evolution and guide further refinements to the analysis.

Development of analytical techniques to define the interaction of ice covers with shorelines or structures is hampered by the complexity and large number of parameters involved. Research has progressed in response to specific technological requirements and the criteria developed rely heavily on empirical coefficients to account for necessary analytical simplifications. Thus, generalized criteria do not exist at present, and designs cannot be prepared for many cases of ice/structure, or shoreline, interaction. Recommendations for research on ice action under varying flow velocity and water level are made.

1 - INTRODUCTION

The winter regime of a river, lake or reservoir depends on a complex interaction of meteorological and hydrodynamic conditions which determine the governing ice processes throughout the water body. Flow velocity is the most important parameter. Because of the difference in the nature of the ice regime during identifiable stages in the life cycle of an ice cover, three periods have been defined for further discussion and presentation of equations governing ice processes in each period. The three periods under consideration are formation, midwinter and breakup.

Ice processes are further complicated when the flow velocity and water levels vary sharply. In response to these changes, the ice regime will reflect the variation of hydrodynamic conditions. The mechanical ice regime will conform to the most severe conditions encountered, while the thermal regime will reflect the cumulative effect of hydrodynamic and meteorological conditions.

2 - GENERALIZED APPROACH TO ICE STUDIES

Standard procedures and criteria for evaluating the effects of flow and level changes on ice covers do not exist. This is a reflection of the state-of-the-art of ice engineering and of the unique nature of each water resource project with respect to its geographic and topographic settings, its purpose and operating characteristics. It also reflects the complex dependency of the ice regime on channel geometry and on the prevailing hydrologic and climatic regimes.

Nevertheless, a broad program of study can be outlined with modifications to suit the particular problems as appropriate. Details of methodologies and criteria are determined when both the scope of study and specific questions for an individual project are defined and specific information on basic data availability is at hand. In general, appropriate methodologies can be adopted from the available arsenal of hydrologic, hydraulic, climatological, heat transfer, system simulation and ice mechanics techniques.

Subsequent sections of this report outline a number of the fundamental heat transfer and ice mechanics techniques which have proven useful in previous applications. They also indicate the state-of-the-art in lesser developed areas of ice engineering, those areas where considerable ingenuity and versatility are required to find solutions for the specific problems in hand.

A general program of study can be outlined under four major activities. These are described as follows.

(a) Ice Surveys

A field program oriented to the collection of ice study data is conducted at reconnaissance level to

- obtain first hand knowledge of channel morphological and hydraulic characteristics

- identify ice cover processes occurring reach by reach in ice cover development, including channel closures, ice front locations and velocities, ice generating reach lengths and open water reaches, border ice growth and spring breakup sequence

- identify sections or reaches which will be of specific interest, e.g., locations vulnerable to flood damage with changes in flow/ice cover/water level regimes, or sections in which major changes in ice processes may be anticipated with changes in the flow regime.

Fundamental to successful execution of this level of survey is an observer with complete knowledge of the mechanics of ice cover development under different hydrologic, hydraulic and climatic regimes.

Heat budget and mechanical stability analysis require detailed surveys to collect

- water level/discharge data at key locations for open water and ice cover conditions

- river channel cross sections and profiles through key reaches

- water surface/ice cover profiles through key reaches, with and without ice cover

- ice and snow cover thickness/composition at key locations

- water temperature profiles through key reaches on selected days combined with air temperature and open-water velocity for heat balance calculations, using thermographs at key locations

- under ice velocities in key reaches or sections.

This level of survey clearly is more costly than the reconnaissance survey. However, much of the foregoing information (e.g., water level/discharge) is often collected at key locations by various agencies for other reasons and is usually adequate for studies of a preliminary nature.

The insights gained from reconnaissance surveys are particularly valuable in planning effective detailed surveys to collect data for more detailed studies. The nature of the specific questions being addressed will dictate, of course, what items of data in the foregoing list must be included in a survey program.

(b) <u>Definition of Flow Regime</u>

The single most important variable determining river stages and the governing ice processes in various river reaches is discharge. It is thus imperative to define the flow regime that will prevail at a point of interest prior to undertaking any ice mechanics analysis. This definition may be as simple as estimating maximum and/or minimum values that might reasonably be expected (e.g., spring breakup) or as complex as *most* accounting for the hourly variations of discharge *resulting from the anticipated operation of* from a water resource *project to meet ever changing electrical/power demands* ~~development~~ (e.g. hydroelectric power plant). Clearly, proper accounting may have to be made of the natural hydrologic regime, its modification by existing and proper reservoirs and/or diversions, and even the influence of externalities such as daily water supply and electrical power demand patterns. Routing of water releases from storage-diversion points to downstream points of interest to account for intermediate channel storage and friction effects on the discharge pattern under both open water and ice cover conditions may also be required.

(c) Definition of Thermal Regime

Reservoirs providing considerable regulation of winter flows have the secondary effect of modifying the thermal regime in downstream channels. In the extreme, these thermal effects will affect winter levels by completely eliminating an ice cover. In most cases, however, the rate and timing of development is all that is affected by alteration of the thermal regimes. Nevertheless, it is sometimes desirable to identify and quantify these changes as a basis for determining the potential effects of proposed projects on such things as winter ice roads, the aquatic environment and incidence of ice-generated fog.

Heat balance calculations must properly take into account local hydraulic, hydrologic and climatic conditions, e.g., whether the reach is a natural channel or a reservoir, open water or ice covered, and whether or not its thermal regime is influenced by upstream storages. The calculations may be simply limited to extreme values for design purposes. Conversely, the nature of the problem may require a day-by-day analysis. The more detailed calculations would permit definition of the length of open-water reaches day-by-day and, in conjunction with ice mechanics analysis, the length and location of both ice generating and solidly covered reaches.

(d) Ice Mechanics Analysis

With a clear understanding of the hydrologic and thermal regimes in hand, various ice mechanics analyses can be undertaken. These analyses lead primarily to stage/discharge relationships at the key point(s) of interest, as determined by the prevailing hydrology, the hydraulics of the channel and the governing ice processes. The stage/discharge relationships, of course, define the ranges and extreme water levels under ice conditions for the range of hydrologic and thermal regimes anticipated, permitting identification of potentially undesirable effects and consequent remedial measures that may be required to make a proposed project feasible.

Analysis will primarily comprise leading edge (Froude number) analysis and internal stability analyses in combination with backwater calculations for key reaches or sections to determine which process governs water levels for the range of hydrologic conditions. Sometimes quite simple "indexes" will clearly show which process dominates, while in other cases more detailed calculations may be required throughout a considerable length of river. The level of detail will also be determined to some extent by both the intent of the study (e.g. prefeasibiltiy or detailed design) and the availability of basic data. At present, various digital computer programs which incorporate many of the fundamental techniques outlined in subsequent sections are in use to facilitate such detailed analyses.

Application [illegible]

An ~~recent~~ Acres study of the effect on the ice regime at the Town of Peace River of the proposed Dunvegan power development on the Peace River in Alberta illustrates the generalized approach to ice studies. The work program included reconnaissance surveys of ice cover formation and evolution, collection of field survey data and review of existing reports on ice conditions, and thermal and mechanical ice regime analysis.

The ice reconnaissance surveys were conducted on the Peace River between the Town of Peace River and the Dunvegan dam site, 105 km upstream of the town. Observation of ice cover formation indicated that leading edge stability was the governing ice process in the vicinity of Peace River. As the ice cover advanced through the town, a critical section was reached. Further progression could not occur until the Froude number at that section was reduced by staging caused by deposition of ice under the ice cover.

Subsequent analyses of leading edge and internal stability confirmed this conclusion. Available river cross sections and meteorological data were used for input to a mathematical model which simulates the generation, upstream progression and subsequent thickening of an ice cover. The water levels at various locations in the Town of Peace River were determined using the model for the appropriate range of operating discharges at the Dunvegan development. Leading edge stability at formation continued to be the governing ice process at these higher, regulated winter discharges. Thus, the critical limit to discharge at formation corresponding to maximum tolerable water levels at flood prone areas in the Town of Peace River was determined.

Heat balance calculations were performed to assess the change in thermal regime ~~due to~~ accompanying reservoir ~~inflows~~ outflows and further, downstream flow regulation. Although the thermal effects of the reservoir on ice generation and retention of ice from upstream reaches delayed the timing and

These were completed for typical warm, normal and cold climatic years in combination with the range [illegible] reduced the rate of cover advance through the Town of Peace River, the mechanical ice processes were not changed.

The Dunvegan study conclusions were based on a comprehensive analysis of thermal and mechanical ice regimes. The dominance of leading edge stability at the critical river section was identified in the course of the reconnaissance survey. The ability to interprete survey observations of ice processes on local and global scales is essential.

3 - FORMATION

With the onset of subfreezing temperatures, ice cover formation starts in low-velocity reaches and shoreline areas where the velocity is less than 0.15 m/s in a manner similar to that on lakes and ponds. Propagation of a crystalline structure of ice begins at the water surface and continues as heat is transferred from the water through the ice to the cold air above.

If the velocity is between 0.15 and 0.3 m/s, surface turbulence may prevent crystalline propagation of sheet ice. In that case, the water becomes supercooled and frazil spicules form. The individual spicules coalesce at the surface to form "slush pans". Channel closure is achieved by a combination of rapid border ice growth and, either accumulation of slush pans at an existing ice cover, ice boom, bridge piers or other obstructions to surface ice flow, or bridging of slush pans at contracting sections, river bends or reaches of decelerating flow where the ice surface concentration, ice cohesion (freezing of slush pans) and flow depth are favorable.

In river reaches where the velocity is greater than 0.30 m/s and the Froude number is less than a critical value, the cover will progress upstream by juxtaposition of ice against the upstream edge of ice cover closures formed initially in quieter river reaches. Border ice growth can assist in closure of these reaches.

If the velocity is greater than 0.30 m/s and the Froude number is greater than the critical value for upstream progression of the ice cover, incoming slush ice will be drawn under the upstream edge of the cover and deposited downstream. The increased resistance to flow caused by thermal growth of the cover, deposited slush ice and thickening of the cover to resist increasing internal stresses with the advance of the ice front results in a steeper backwater slope. At a lower Froude number resulting from lower velocity and deeper flow depth, upstream progression of the ice cover is again possible.

At very high velocities and Froude numbers much higher than the critical value for ice cover progression, closure of the section may not be accomplished by winter's end if the open water area to generate frazil ice is limited. Thermal border ice growth and "buttering" of the border ice with slush ice will be the governing ice processes.

3.1 - Thermal Processes at Formation

An understanding of the thermal regime in a river and the ability to calculate the cooling rate of the flow and rate of frazil ice generation are necessary to predict the ice regime in a river. Determination of the mechanical stability of unconsolidated ice accumulations in the river may be irrelevant if sustained supercooling required to generate frazil ice is not achieved.

3.1.1 - Heat Balance on Open Water

Initially the cooling of water to below the freezing point must be considered. Computation of the cooling rate from an open-water surface can be based on empirical relationships or a summation of the various components of heat loss or heat gain in a heat budget computation.

Empirical equations take the form

$$Q^* = k\ (T_w - T_a) \tag{3.1}$$

where Q^* = rate of heat transfer per unit area (J/m^2 day)

k = coefficient of heat transfer representative of site conditions (J/m^2 day °C)

T_w = water temperature (°C)

T_a = air temperature (°C).

Although the coefficient k must account for all aspects of heat transfer for the river reach under consideration such as latitude and local wind speeds and exposure, excellent results have been obtained applying this simplified technique. Typical values based on field observations are presented by McLachlan (1926), Williams (1959) and Marcotte (1975). McLachlan found that for all river reaches and lakes along the St. Lawrence from Kingston to Montreal, a k value of 1.9×10^6 J/m^2 day °C is applicable. Marcotte discusses the variation of the overall transfer coefficient with meteorological parameters of wind speed and air temperature.

With a heat budget approach, the major components of heat gain or loss, convection, evaporation, radiation and heat exchange from precipitation, are summed to give the net heat transfer rate. Discussion and formulation of heat budget calculations are presented in Newbury (1966), Carstens (1970), Raphael (1962) and Michel (1971). Those of Raphael and Michel are particularly comprehensive and give a thorough understanding of the subject. The basic equations developed by Michel provide a computation method suitable for engineering applications and are presented here.

TABLE 3.1

INSOLATION COEFFICIENT VALUES, C_s, FROM MICHEL (1971) (units converted to J/m^2 day)

	Latitude			
	40°N	50°N	60°N	70°N
Sept. 1	39,730	31,070	22,400	14,190
Sept. 15	36,090	27,340	18,300	10,470
Oct. 1	31,820	22,520	13,980	6,660
Oct. 15	26,800	18,210	9,800	3,680
Nov. 1	22,520	13,610	6,150	1,260
Nov. 15	19,300	10,010	3,730	170
Dec. 1	15,570	7,750	2,300	≃0
Dec. 15	14,740	6,870	1,840	≃0
Jan. 1	14,860	6,950	1,800	≃0
Jan. 15	16,370	8,250	2,510	≃0
Feb. 1	21,480	11,100	4,440	630
Feb. 15	23,150	15,490	7,410	2,300
Mar. 1	29,060	20,010	11,260	4,520
Mar. 15	34,960	24,740	15,700	8,120
Apr. 1	39,730	30,480	21,350	12,310
Apr. 15	42,660	34,160	25,080	16,200
May 1	44,760	37,300	28,600	19,550
May 15	46,010	39,270	31,280	22,480

The heat loss on a daily basis is given by

$$Q^* = -5.87\, U_{15}\, [32 - (0.66 + 0.33\, e\, \xi)(32 + 1.8\, T_a)] + 640\, C_s\, (0.25 + 0.75\, \frac{m}{N}) - 3.44\, (33 - 1.8\, T_a) - 130\, \frac{m}{N} - 8{,}000\, i_s + 20{,}260\, q\, S + \frac{100\, q_g\, \Delta T_g}{B} + Q_{ex}$$

with $\xi = 1$ if $T_a > -18$ and $\xi = 0$ if $T_a \leq -18$ (3.2)

where Q^* = rate of total heat transfer per unit surface area (J/m^2 day)

U_{15} = wind velocity 15 m above the water surface (m/s)

e = relative humidity of air

T_a = air temperature 15 ft above the water surface (°C)

C_s = insolation coefficient (Table 3.1) which can be set to zero for overnight heat loss calculation (J/m^2 day)

$\frac{m}{N}$ = fraction of hours of clear sky during daylight hours

i_s = water equivalent of snow precipitation (m/day)

q = river discharge per unit width (m^3/s m)

S = slope of flow energy gradient

q_g = groundwater discharge per unit river length (m^3/s m)

ΔT_g = groundwater temperature difference from water (°C)

B = river width (m)

Q_{ex} = external artificial addition of heat (J/m^2 day).

The data required for this formula can be readily obtained from climatic normals published for different regions in Canada.

From the determined rate of heat transfer, the temperature change along a river reach can be calculated from

$$T_w - T_o = \frac{1}{86,400\ \rho_w\ C_p} \int_{\ell = \ell_o}^{\ell} \frac{Q^*\ d\ell}{q} \tag{3.3a}$$

where T_w = water temperature at location ℓ (°C)

T_o = water temperature at location ℓ_o (°C)

ℓ, ℓ_o = river location along flow axis (m)

ρ_w = specific mass of water (kg/m^3)

C_p = specific heat of water (J/kg °C).

If the rate of heat transfer and river width do not vary greatly along the reach this becomes

$$T_w - T_o = 1.157 \times 10^{-8}\ \frac{Q^*\ (\ell - \ell_o)}{q} \tag{3.3b}$$

For a given temperature change, the length of river reach along which cooling must take place can be determined from

$$\ell - \ell_o = 86,400\ \rho_w\ C_p \int_{T_w = T_o}^{T_w} \frac{q\ dT_w}{Q^*} \tag{3.4a}$$

which for uniform channel width and constant heat transfer can be simplified to

$$\ell - \ell_o = 8.64 \times 10^7\ q\ \frac{(T_w - T_o)}{Q^*} \tag{3.4b}$$

Dingman, Weeks and Yen (1968) present a similar formulation based on a heat budget approach to predict the length of ice-free river reach downstream from a thermal inflow site. If thermal pollution input, or a deep-water reservoir source is considered, the length of the equilibrium open water reach downstream is given by

$$X = -8.64 \times 10^3 \ \frac{\rho_w \ C_p \ q}{k} \int_{T_o}^{T_f} \frac{dT_w}{Q^*} \tag{3.5}$$

where X = length of open water reach (m)

T_f = freezing temperature of water (°C)

T_o = initial water temperature (°C)

k = coefficient of heat transfer representative of site conditions (J/m^2 day °C)

and other variables with units as previously defined.

If the rate of heat loss, Q*, is approximated by

$$Q^* = k \ (T_w - T_a) \tag{3.1}$$

equation 3.5 becomes

$$X = \frac{-8.64 \times 10^4 \ \rho_w \ C_p \ q}{k} \ \ln \left(\frac{T_f - T_a}{T_o - T_a}\right) \tag{3.5a}$$

for constant air temperature and coefficient of heat transfer. Rearranging equation 3.5a and taking antilogs gives

$$T_f - T_a = T_o - T_a \ e^{-(k/8.64 \times 10^4 \ \rho_w \ C_p \ q) \ X} \tag{3.5b}$$

Substituting the first two terms of an exponential series for the right hand side of equation 3.5b gives

$$T_f - T_a = (T_o - T_a) \ (1 - (k/8.64 \times 10^4 \ \rho_w \ C_p \ q) X) \tag{3.5c}$$

or

$$X = \frac{-8.64 \times 10^4 \rho_w C_p q}{k} \left(\frac{T_f - T_o}{T_o - T_a}\right) \qquad (3.5d)$$

If Q* in equation 3.4b is taken as the average heat transfer along the reach, between the initial section and upstream limit to the cover, given by

$$Q^* = k \left(\frac{T_o + T_f}{2} - T_a\right)$$

equation 3.4b becomes

$$\ell - \ell_o = \frac{8.64 \times 10^3 \rho_w C_p q}{k} \left(\frac{T_f - T_o}{\frac{T_o + T_f}{2} - T_a}\right) \qquad (3.4c)$$

The difference in these equations is due to the exclusion of Q* from integration in Michel's approach.

3.1.2 - Frazil Ice Generation

Frazil ice production occurs in turbulent water when the disturbance of the flow surface prevents formation of a thermal sheet of ice. As pointed out by Tsang (1976), supercooling of water below the freezing point is necessary in order that the release of latent heat does not immediately heat the water system to above freezing temperature. Michel (1971) states that maximum supercooling observed in nature is 0.06°C with a normal range of 0.01°C to 0.03°C.

Ice formed in this manner is part of a continuously evolving process. Small frazil discoids are formed uniformly throughout the turbulent flow. The invisible discoids grow into needles or spicules which rise to the surface of the flow, coalescing into frazil slush clusters. Frazil is only active in the discoid state during which it adheres readily to other ice particles or thermally conductive objects. Tsang and Michel give clear descriptions of frazil evolution. Carstens (1970) discusses the effects of rate of cooling and turbulence on laboratory generation of frazil ice and presents results of field observation programs in Norway.

Carstens concluded that the degree of supercooling is a function of the rate of heat loss from the free water surface, and of the intensity of flow turbulence transporting heat to the water surface. Field experience indicated an upper limit to surface velocity of 0.6 m/s for ice covers to form due to flow turbulence.

Anchor ice has been comparatively less studied than frazil ice. It has been shown that frazil and anchor ice have similar origins in supercooled turbulent flows, anchor ice forming at the same time as frazil runs, but it remains to determine the factors controlling the predomenance of the two ice formations under a given supercooling condition.

After the water becomes supercooled, continued heat loss results in a change of state or frazil production. The rate of frazil production is given by Michel as

$$Q_i = \frac{Q^* \, A_{o/w}}{\rho_i \, L_f} \qquad (3.6)$$

where Q_i = volume rate of ice production (m^3/day)

Q^* = heat transfer rate to atmosphere (J/m^2 day)

$A_{o/w}$ = supercooled open water generation area (m^2)

ρ_i = specific mass of ice (kg/m^3)

L_f = latent heat of fusion of ice (J/kg).

The open water area must be adjusted for the effect on width of the border ice growth variation along the reach and, as noted by Newbury, reduction of ice generation area due to insulation of the water surface by accumulating pans of slush ice travelling downstream and causing the cover to progress upstream in the reach.

3.1.3 - Border Ice Growth

In river reaches where a thermal cover cannot grow uniformly across the entire channel width, the first ice to form is shore, or border, ice in areas of low velocity. This ice grows rapidly from the banks and islands toward the center of the channels in a solid sheet.

As the edge of the sheet extends into the more turbulent part of the flow, its progress decreases markedly, but still continues to advance. The growth of ice does not require water temperature to be below freezing. The rate of advance depends on the relative rates of heat exchange between the ice sheet edge and the turbulent water and between the ice sheet and the atmosphere.

The quantity of heat given to the ice boundary by the warm water is given by

$$Q^*_w = k_w \left[\frac{dT_w}{dy}\right]_{y = o} \qquad (3.7a)$$

where k_w = thermal conductivity of water (J/m day °C)

$\left[\frac{dT_w}{dy}\right]_{y = o}$ = water temperature gradient at the ice boundary along the top water surface layer (°C/m). (y - direction)

The heat taken out by convection at the ice boundary can be similarily expressed as

$$Q^*_a = k_a \left[\frac{dT_a}{dz}\right]_{z = o} \tag{3.7b}$$

where k_a = thermal conductivity of air (J/m day °C)

$\left[\frac{dT_a}{dz}\right]_{z = o}$ = air temperature gradient at the ice boundary normal to the ice sheet (°C/m). (z - direction)

Border ice growth occurs if Q^*_a is greater than Q^*_w. Because of the values of k_a and k_w, the air temperature gradient must be 25 times greater than the water temperature gradient for ice growth. This can be satisfied for winter wind velocities and air temperature differences below freezing compared with water velocities and water temperature differences above freezing (Michel 1971).

Difficulty in obtaining practical measurements of these temperature gradients which vary appreciably with time makes it necessary to resort to empirical relationships to determine border ice growth.

Newbury (1966) developed an empirical border ice growth equation based on observations at several river sections on the Nelson River incorporating an assumption for the "tractive force" nature of the problem. Newbury observed that border ice growth may be interrupted by abrasion of passing slush ice or solid floes in the open channel. Conversely, border ice growth may be accelerated by "buttering" of slush ice onto the border ice edge in layers. The cohesive, or adhesive, strength of the slush ice and rate of heat exchange determines the thickness of slush layers adding to the border ice width.

The rate of border ice growth across the width for the Nelson River is given by Newbury as

$$B_i = 2.68 \times 10^{-5} \frac{m\ N_b\ \Sigma Q^*}{2} \tag{3.8}$$

where B_i = total border ice growth (m)
N_b = number of boundaries
ΣQ^* = sum of net heat transferred (J/m^2)
m = an adhesive parameter.

The adhesive parameter was determined as

$$m = \frac{a}{(10.76\ AS)^b} \tag{3.8a}$$

where a and b are empirical coefficients
A = cross-sectional flow area (m^2)
S = water surface slope (m/m).

For the Nelson River a = 5 and b = 2.7 and

$$B_i = \frac{1.342 \times 10^{-4}\ N_b\ \Sigma Q^*}{(10.76\ AS)^{2.7}} \tag{3.8c}$$

Rearranging,

$$\Sigma Q^* = \frac{(10.76\ AS)^{2.7}\ B_i}{1.342 \times 10^{-4}\ N_b} \quad J/m^2 \tag{3.9}$$

In the absence of field data for a given river, this can be compared with experienced rates of Q* to determine the time required for closure of a given width of river reach by border ice growth.

In spite of the number of parameters influencing border ice growth, each of which can vary with time, a simple observation program to relate cumulative degree-days of freezing to border ice growth width gives a satisfactory indication of border ice growth based only on readily available temperature data. This is due to the relatively slow thermal response of the border ice cover to daily fluctuations in hydrologic or meteorological conditions.

3.1.4 - Channel Closures

In high velocity river reaches where channel closure cannot be achieved by formation of a thermal cover, supercooling will occur and frazil ice will be generated. In reaches where the surface coverage of moving slush reaches 100 percent ($N_i = 1.0$), the blanket of slush floes may be compressed at channel constrictions forming a continuous ice-bridge between shore-fast ice boundaries. Whereas 100 percent surface coverage is necessary for such bridging, it is not sufficient. For example, Michel (1971) quotes Dutch experience on two rivers for which the required conditions were

- surface concentration $N_i \geq 1.0$ for 5 to 8 hours
- surface velocity $V_s < 0.5$ m/s for 5 to 8 hours
- average air temperature $< -9°C$.

The coverage, N_i, can only "exceed" 1.0 by compaction of the slush to form a more dense blanket or by thickening of the blanket to increase surface discharge when the product of width and surface velocity is reduced. The rate of surface ice transport is given by the continuity equation

$$Q_s = N_i \, V_s \, t_s \, B(1 - \varepsilon) \tag{3.10}$$

where Q_s = surface ice transport rate (m^3/s)

N_i = surface ice concentration (tenths)

V_s = surface velocity (m/s)

t_s = slush blanket thickness (m)

B = channel width (m)

ε = porosity of slush accumulation (m^3/m^3).

Thus, for a decelerating flow or converging section without a corresponding increase in surface velocity (i.e. simultaneous narrowing and deepening) and 100 percent surface coverage, the slush blanket will thicken or compress (N_i >1.0).

Velocities lower than 0.5 m/s correspond to a critical limit to surface shear on the underside of the cover, and therefore a lower requirement for resisting forces in the ice to arrest and hold the blanket.

The duration of 5 to 8 hours and average temperatures less than -9°C represents an opportunity to develop sufficient cohesion in the upper slush layer to provide an average cohesive strength throughout the entire depth of the blanket which is capable of resisting the drag and body forces acting to maintain slush transport.

All of the foregoing elements are represented in the stability criterion formulated by Pariset, et al, (1961) which can be applied to slush blanket stability if the following differences are considered.

- The channel width becomes the width of mobile slush blanket between border ice growth rather than the width between river banks.

- There is no downstream barrier for the moving slush blanket to "pack" against to develop internal buoyant resistance by thickening of the blanket.

Pariset's stability criterion is developed in the following section on Mechanical Stability at Formation, and the application to channel closure is discussed.

3.2 - Mechanical Stability of an Ice Cover at Formation

3.2.1. - Leading Edge Stability

Upstream progression of incoming ice occurs at an obstruction to the flow or when the conveyance at the channel is reduced below the surface inflow of mobile ice. This can occur at a variety of man-made or natural obstacles such as bridges, dams or ice booms, and bends, channel constrictions or thermal ice covers in lakes or low-velocity river reaches. When the moving slush blanket reaches the downstream end of a frazil generating reach, it can progress upstream with all ice forming a stable leading edge if conditions are favorable. If conditions for stable ice accumulation are not fulfilled, the cover can still progress with difficulty if the inflow of slush ice exceeds the rate of transport of ice under the cover from the leading edge. The rate of advance of leading edge of the cover in the latter case will depend on the net rate of ice inflow to the leading edge of the ice cover.

Initial studies by McLachlan (1926) and a later study by Cartier (1959) sought to fix a limiting velocity criterion for which a cover could advance. A wide range of limiting velocities was obtained for different ice and river characteristics indicating that further refinement was required. Kivisild proposed a limiting Froude number for upstream progression of a packed ice cover defined as

$$Fr_{cr} = \frac{V_{cr}}{\sqrt{g\ H}} = 0.08 \qquad (3.11)$$

where V_{cr} = mean flow velocity (m/s)
g = acceleration of gravity (m/s^2)
H = mean flow depth (m)

This criterion has proved to be successful for a wide variety of hydraulic, ice and meteorological conditions.

Subsequent studies by Pariset, et al (1961), and Michel (1965) served to confirm the simple criterion of Kivisild. The criteria of Pariset, et al, and Michel differ only in the inclusion of a porosity term in Michel's derivation.

Frazil slush and pans can progress upstream at a determined thickness as the ice underturns and piles up under the cover. The limiting Froude number for incipient instability of the leading edge of the thickened cover (submersion of the cover and transport of ice under the cover) is defined as

$$Fr_{cr} = \frac{V_{cr}}{\sqrt{g\ H}} = \sqrt{2\ \left(\frac{\rho - \rho'}{\rho}\right)\ (1 - \varepsilon)\ \frac{t}{H}\ \left(1 - \frac{t}{H}\right)} \qquad (3.12)$$

where V_{cr} = critical upstream mean velocity of flow (m/s)
g = acceleration of gravity (m/s^2)
H = upstream flow depth (m)
ρ = water density (kg/m^3)
ρ' = ice density (kg/m^3)
t = equilibrium ice cover thickness (m)
ε = porosity of accumulated ice cover i.e. ratio of volume of voids filled with water to the total volume of the ice accumulation

At a relative ice cover thickness to flow depth ratio t/H, of 1/3, the restriction to flow under the cover and upsetting forces on the ice become critical. The formulation can no longer apply and further thickening of the ice cover leading edge is not possible (Pariset and Hausser, 1961).

For the observed limiting ratio of cover thickness to flow depth of 1/3 and water and ice densities of 1,000 and 920 kg/m^3, respectively, this reduces to

$$Fr_{cr} = .154 \sqrt{1 - \varepsilon} \tag{3.12a}$$

The limiting case of a solid floe ($\varepsilon = 0$) is 0.154 which has been observed in laboratory studies. The more typical case encountered in field conditions is a porosity of 0.72 for a packed cover of frazil and small floes, hence the typical critical Froude number of 0.08 reported by Kivisild.

Numerous studies have been conducted to determine the stability of individual ice blocks [Ashton (1974), Larsen (1975), Chee and Haggag (1978), Uzuner and Kennedy (1972)]. From this, the stability of the leading edge of the cover is inferred. However, Michel (1971) points out that individual block stability is not generally a limiting condition as incoming frazil and slush and pans may underturn and pack at the leading edge until it is able to progress upstream at a determined thickness. The minimum thickness of the leading edge of the cover considering forces acting on the leading edge can be estimated from the following derivation of Pariset, et al,

$$t_m = \frac{V_u^{\,2}}{2g\left(\frac{\rho - \rho'}{\rho}\right)} \qquad (3.13)$$

where V_u = flow velocity under the edge of the ice cover (m/s)

From continuity, this becomes,

$$t_m = \frac{V^2}{2g\left(\frac{\rho - \rho'}{\rho}\right)\left(1 - \frac{t_m}{H}\right)^2} = \frac{0.017\, V^2}{\left[1 - t_m/H\right]^2} \qquad (3.13a)$$

where V = flow velocity upstream of the cover (m/s).

An iterative solution is required for covers which are relatively thick compared to flow depth.

The various formulas for individual block stability are not practical or applicable for the stability of a packed cover comprised of an indeterminable distribution of frazil pan and ice block fragment sizes. Field experience has demonstrated that, for engineering purposes, the limiting Froude number of 0.08 and the minimum thickness for cover progression define leading edge conditions.

3.2.2 - Internal Stability at Formation

As the ice cover progresses upstream, the internal stress increases as the hydrodynamic shearing force of the flow under the cover, the shearing stress of wind on the cover and weight of ice along the slope of the ice/water interface are added to the hydrodynamic thrust on the leading edge of the cover. The increasing forces on the cover as it lengthens must be balanced by the internal resistance of the cover created by the buoyancy of the accumulated ice and the resistance of the banks. Bank reaction is comprised of an ice-over-ice frictional term related to the internal stress transferred to the banks and a cohesive (freezing) term. If the internal stress increases beyond the internal resistance of the cover, the cover will shove or thicken until the resistance increases sufficiently.

Relationships to equate forces acting on an ice cover and resisting forces of the cover have been derived by Pariset, et al (1961), Michel (1965), and Uzuner and Kennedy (1976). These derivations vary in the treatment of the resisting forces leading to different forms of coefficients for internal friction and cohesion. Experience in field application and verification of coefficient values favour the formulation of Pariset, et al. The general relationship is given by

$$\frac{BV^2}{C^2H^2} = \left(\left[\frac{2}{\rho g}\frac{\tau}{H} + \mu\frac{\rho'}{\rho}\left(1 - \frac{\rho'}{\rho}\right)\frac{t}{H}\right]\frac{t}{H} + \frac{f_w B}{\rho g H^2}\right)\frac{\left(1 - \frac{\rho'}{\rho}\frac{t}{H}\right)^3}{\left(1 + \frac{\rho'}{\rho}\frac{t}{H}\right)} \qquad (3.14)$$

where B = channel top width (m)

V = mean flow velocity under open water conditions (m/s)

see note next page !! C = Chezy roughness coefficient, assumed equal for river bed and underside of ice cover ($m^{1/2}/s$)

H = open-water flow depth (m)

τ = cohesive strength of ice (Pa)

t = ice cover thickness (m)

μ = coefficient related to ice-over-ice friction and internal resistance of the cover.

g = gravitational acceleration (m/s^2)
ρ' = density of ice (kg/m^3)
ρ = density of water (kg/m^3)
f_w = wind drag on the upper surface of the cover (Pa) (positive if acting in the downstream direction)

Pariset, et al, reported values of Manning's n-value for the composite roughness on the St. Lawrence of 0.06 to 0.05 at formation and 0.04 to 0.03 at later stages of a stable cover. The Chezy roughness parameter can be related to Manning's n-value by

$$C = \frac{1}{n_i} R_H^{1/6} \qquad (3.15)$$

where R_H = the hydraulic radius of the ice-covered flow (m).

It should be noted that the open-water velocity and flow depth are those that would exist at the same hydrostatic depth as the flow with an ice cover. Thus, the open-water depth must consider staging due to the presence of the ice cover downstream. The ratio of winter open-water flow depth to summer open-water flow depth obtained from Manning's equation with constant discharge equals

$$\frac{H}{H_o} = 1.32 \left(\frac{n_c}{n_b}\right)^{3/5} + \frac{\rho' t}{\rho H_o} \qquad (3.16)$$

where H_o = summer open-water flow depth (m)
n_c = composite roughness with ice cover (see Section 3.2.6)
n_b = channel bed roughness.

For thin covers ($t \ll H_o$) with ice roughness near that of the river bed, a 30 percent increase in stage can be anticipated. If staging and flooding are not important, ice cover stability can be determined conservatively from open-water hydraulic parameter values at the appropriate discharge.

The wind drag can be estimated from the Von Karman equation by

$$f_w = U^2 \rho_a / (5.75 \log (W/K_i) + 8.5)^2 \qquad (3.17)$$

where U = wind speed (m/s)

ρ_a = air density (kg/m^3)

W = height of wind velocity, U, above ice surface (m)

K_i = roughness height of ice (m).

For a wind speed height of 10 m, ice roughness height of 0.4 m and air density of 1.28 kg/m^3 recommended by Michel, this reduces to

$$f_w = 10.14 \times 10^{-5}\, U_{10}^{\,2} \qquad (3.19a)$$

Solution of the stability equation for the stable ice thickness, t_i, yields

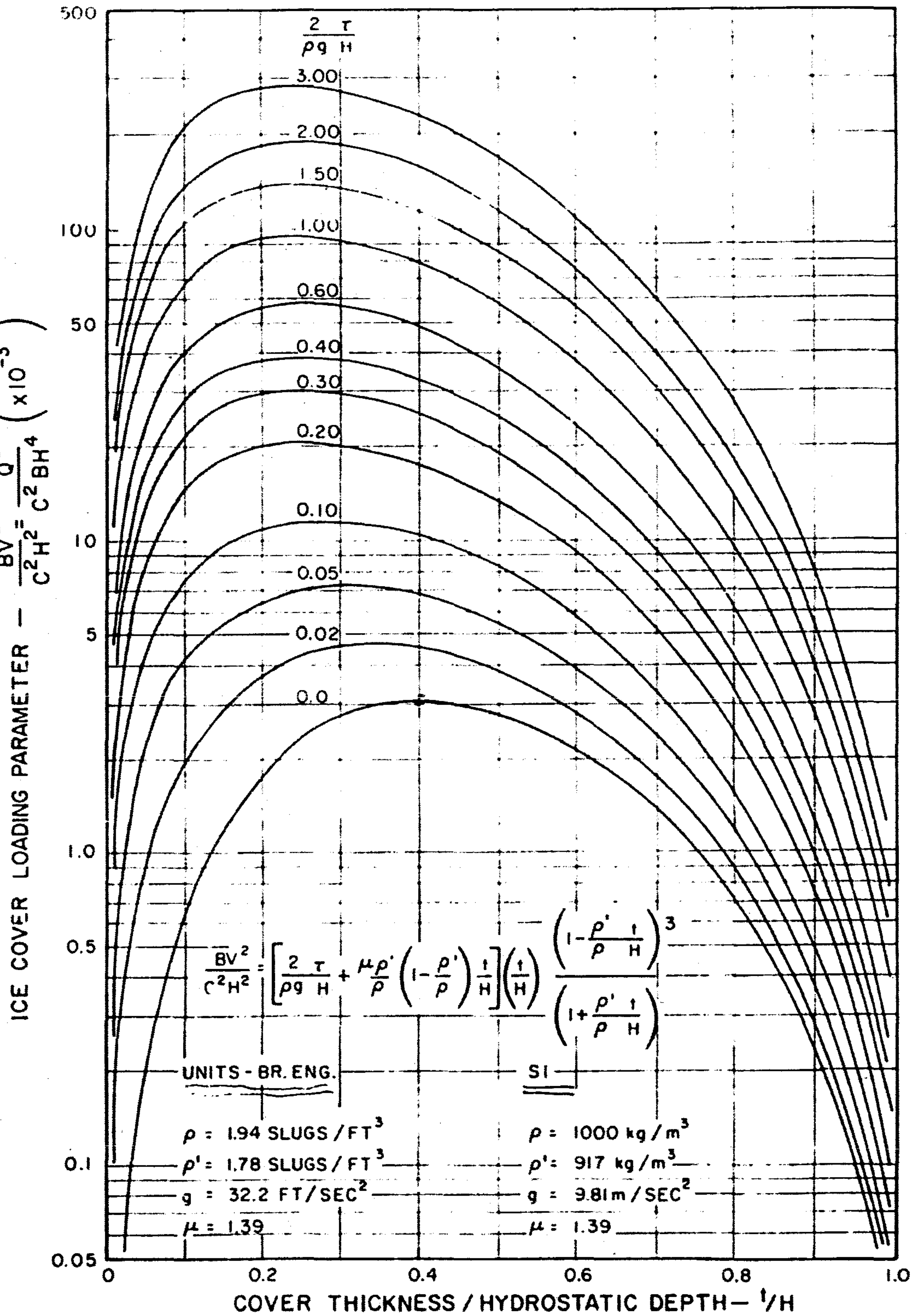
$$t_i = \frac{-\dfrac{2\tau}{\rho g H^2} \pm \sqrt{\left(\dfrac{2\tau}{\rho g H^2}\right)^2 + 4\,\dfrac{\mu \rho'}{H^2 \rho}\left(1 - \dfrac{\rho'}{\rho}\right)\left[\dfrac{BV^2}{C^2 H^2}\,\dfrac{\left(1 + \dfrac{\rho' t_i}{\rho H}\right)}{\left(1 - \dfrac{\rho' t_i}{\rho H}\right)^3} - \dfrac{f_w B}{\rho g H^2}\right]}}{2\,\dfrac{\mu \rho'}{H^2 \rho}\left(1 - \dfrac{\rho'}{\rho}\right)} \qquad (3.13)$$

An iterative solution to the above equation with assumed values of t_i is required. At this stage of complexity, a computer solution of the equation is recommended. Manual approximations are available, however, as will be outlined in a following paragraph.

Solution of the stability equation has been plotted in Figure 3.1. For a given value of cohesion, the area under the corresponding curve, defined by the parameter $2\tau/\rho gH$, represents all stable combinations of relative ice thickness, t/H, and the internal stability ratio, BV^2/C^2H^2. For cover progression at formation , the limit to the relative thickness at the leading edge is $t/H<1/3$. (At a greater thickness, basic assumptions for leading edge stability become invalid.) As the cover progresses upstream, a minimum thickness at the lower, or left-hand t/H, limit at the appropriate $2\tau/\rho gH$ curve is required for internal stability corresponding to the ratio BV^2/C^2H^2. Thus, the ice cover thickness for progression of the leading edge can be less than or greater than the eventual internal stability thickness, indicated by the values of BV^2/C^2H^2 and t/H, falling above or below the appropriate $2\tau/\rho gH$ curve, respectively.

Pariset, Hausser and Gagnon (1966) were unable to separate the product τt. They reported values of τt from 1,100 to 1,300 N/m and a value of μ equal to 1.28. Lavender (1973)* was able to reanalyse the data of Pariset, Hausser and Gagnon (1966) and separate τ and t. This method showed that τ could vary from 0 to 4,070 Pa. A value of 3,110 Pa is recommended for typical ice cover formation conditions. The range of μ for the data ranged from 1.39 to 1.60. Thus, the value reported by Pariset et al is conservative. A minimum design value of 1.39 is recommended for design, and is used in this report in Figure 3.1.

*Unpublished notes of Acres Employee Development Committee Seminar, January 1973



$$\frac{BV^2}{C^2H^2} = \left[\frac{2}{\rho g}\frac{\tau}{H} + \frac{\mu\rho'}{\rho}\left(1-\frac{\rho'}{\rho}\right)\frac{t}{H}\right]\left(\frac{t}{H}\right)\frac{\left(1-\frac{\rho'}{\rho}\frac{t}{H}\right)^3}{\left(1+\frac{\rho'}{\rho}\frac{t}{H}\right)}$$

FIG. 3.1

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

ICE COVER STABILITY DIAGRAM

ACRES

For a given ice cover cohesion, the maximum value of the ice cover loading parameter, BV^2/C^2H^2, occurs at a relative thickness, t/H, from 0.2 to 0.4. Above this relative thickness, the increased stress imposed on the cover exceeds the increase in stability due to increased cover thickness and the upper limit to BV^2/C^2H^2 decreases with increasing t/H.

If the BV^2/C^2H^2 value is higher than the maximum point on the $2\tau/\rho gH$ curve, cover stability cannot be achieved. Thickening of the cover by internal collapse, and transport and deposition of ice downstream causes staging which reduces the value of BV^2/C^2H^2 sufficiently for cover stability.

Deposition of ice transported under the cover from upstream can cause instability if the relative ice thickness increases beyond the upper, or right-hand, t/H limit to the $2\tau/\rho gH$ curve. In this case as well, an increase in depth resulting from staging under the thicker cover serves to reduce stresses in the cover.

If some reasonable assumptions are adopted, the solution for internal stability is greatly simplified. For design purposes, it is generally appropriate and conservative to assume a cohesionless cover, i.e., $\tau = 0$. Also, it is generally desired to design the channel so that the ice cover is relatively thin, i.e., $t \ll H$, and the equation can then be simplified to

$$\frac{BV^2}{C^2H^2} = \mu \frac{\rho'}{\rho} \left(1 - \frac{\rho'}{\rho}\right)\left(\frac{t}{H}\right)^2 \pm \frac{f_w B}{\rho g} \tag{3.14a}$$

Variable wind direction makes it difficult to account for the effect of wind drag because f_w can contribute to instability or stability of the cover. For that reason, wind stress is normally ignored for ice cover stability in rivers. However, for the design of ice boom installations in wide river reaches and determination of river closure by slush bridging, wind stress can play a major role and should be considered. With the preceding assumptions ($\tau = 0$, $t \ll H$, $f_w = 0$), the stable ice cover thickness, t_i, is given by

$$t_i \approx 3.12 \sqrt{\frac{BV^2}{C^2}} \qquad (3.18a)$$

for the substituted values

Imperial units? ok for metric.

$\rho'/\rho = 0.92$

$\mu = 1.39$ (estimated minimum value obtained by Lavender from Pariset's field data).

Pariset, et al, (1961), showed that rivers could be classified as "wide" or "narrow" depending on the governing stability criterion for stable ice cover thickness. If leading edge stability requires a greater thickness than internal stability, the river is "narrow". Conversely, if internal stability requires a greater thickness of cover than does leading edge stability, the river is "wide". To determine the transition between wide and narrow rivers, leading edge stability ice thickness is equated to the approximate solution to internal stability ice thickness. Thus, for a wide river

$$t_m = \frac{V^2}{2g\left(\frac{\rho - \rho'}{\rho}\right)} < t_i = 3.12 \sqrt{\frac{BV^2}{C^2}}$$

which yields

$$\frac{V^2C^2}{B} < 38.94 \; g^2 \left(\frac{\rho - \rho'}{\rho}\right)^2 \qquad (3.19)$$

Thus, a river is

"wide" if $\frac{V^2C^2}{B} < 24$ (3.19a)

"narrow" if $\frac{V^2C^2}{B} > 24$ (3.19b)

This ratio is a useful indicator of ice conditions in the reach. If an ice cover can progress through the reach, further calculation for internal stability and ice cover thickness is not necessary if the river is narrow.

3.2.3 - River Classification by Governing Ice Process

The combinations of leading edge and internal stability criteria yield the following classifications of river reaches. The leading edge criterion of Michel (1971) has been reformulated by substitution of the Chezy flow relationship as

$$Fr_{cr} = C \frac{\sqrt{R_H S}}{\sqrt{gH}} = 0.154 \sqrt{1 - \varepsilon} \qquad (3.12b)$$

where S = slope of the hydraulic grade line (equal to bed slope for uniform flow).

For a wide channel

$$R_H \simeq \frac{H}{2}$$

and

$$CS^{1/2} = 0.682 \sqrt{(1 - \varepsilon)} \qquad (3.12c)$$

If the greater value of porosity recommended by Michel of 0.72 is used, this becomes

$$CS^{1/2} = 0.361 \tag{3.12d}$$

The ice cover can progress upstream without staging if the Froude number is less than critical and the product, $CS^{1/2}$, is less than 0.361. This yields the following Table 3.2 when combined with the internal stability criterion in equations 3.19a and 3.19b.

3.2.4 - Application to River Closure

As discussed in Section 3.1.4, closure conditions for a river reach covered with a moving slush blanket can be predicted using the Pariset stability equation. Because no internal resistance can be developed in the moving ice, the internal stress must reduce in the downstream direction within the cover until it becomes zero. At that point, cohesion at the river boundaries (border ice or river banks) can support the cover. The modified formula for this condition is

$$\frac{BV^2}{C^2H^2} = \left(\frac{2\tau\ t_s}{\rho g\ H^2} \pm \frac{f_w\ B}{\rho g\ H^2}\right) \frac{\left(1 - \frac{\rho'\ t_s}{\rho\ H}\right)^3}{\left(1 + \frac{\rho'\ t_s}{\rho\ H}\right)} \tag{3.14b}$$

where t_s = thickness of the moving slush blanket (m).

The combination of factors which will then contribute to bridging are

(a) a reduction in discharge to reduce drag and body forces in the blanket

(b) a wind blowing upstream to help balance the downstream forces

TABLE 3.2

RIVER CLASSIFICATION BY ICE COVER PROCESS

Leading Edge Stability (open water)	Internal Stability: $\frac{V^2C^2}{B} > 24$ (narrow)	Internal Stability: $\frac{V^2C^2}{B} < 24$ (wide)
$Fr_{cr} < 0.154\sqrt{1-\varepsilon}$ $CS^{1/2} < 0.682\sqrt{1-\varepsilon}$ (stable without staging)	CLASS A - cover forms easily by juxtaposition without staging and without subsequent shoving and packing.	CLASS B - cover advances easily by juxtaposition and subsequently shoves and packs to maintain internal equilibrium
$Fr_{cr} > 0.154\sqrt{1-\varepsilon}$ $CS^{1/2} > 0.682\sqrt{1-\varepsilon}$ (stability requires staging)	CLASS C - cover can advance only with staging but has sufficient thickness to maintain internal stability	CLASS D - cover advances with difficulty only after staging and subsequently shoves and packs to maintain internal equilibrium

(c) a reduction in blanket width, B, by border ice growth

(d) increased heat transfer from the blanket to increase cohesion, caused by low temperature and high wind

(e) increased slush blanket thickness, t_s, at a surface contraction (N_i >1.0).

3.2.5 - Accumulation of Slush and Frazil

Frazil ice generation depends on the rate of heat loss over a supercooled open-water area, as discussed in Section 3.1.2. The rate of ice production per unit area in an open-water reach is given by

$$q_i = \frac{Q^*}{\rho_i \; L_f} \qquad (3.20)$$

where q_i = rate of ice production per unit area ($m^3/s\ m^2$)

Q^* = rate of heat transfer from open-water surface ($J/m^2\ s$)

ρ_i = specific weight of ice (kg/m^3)

L_f = latent heat of fusion of ice (J/kg).

As stated previously, the determined surface area for generation of frazil must consider the presence of border ice growth and slush pans. If the ice is assumed to coalesce into pans at the surface having a typical thickness, t_s, the surface concentration of ice, N_i, will increase from zero at the upstream end of the supercooled reach to a limiting value of 1.0, at which point frazil ice generation will effectively cease.

The surface ice concentration at any point x downstream in the reach is given by*

$$N_i = 1 - \exp\left[-\frac{Q^*(\varepsilon + 1)\ x}{\varepsilon\ \rho_i\ L_f\ t_s\ V_s}\right] \qquad (3.21)$$

where N_i = surface area concentration of slush pans (m^2/m^2)

Q^* = rate of surface heat transfer from open water ($J/m^2\ s\ °C$)

x = distance downstream from start of supercooling (m)

ρ_i = specific weight of ice (kg/m^3)

L_f = latent heat of fusion of ice (J/kg)

ε = porosity of slush accumulation (m^3/m^3)

t_s = thickness of slush accumulation (m)

V_s = surface flow velocity (m/s).

The total rate of frazil ice production along the reach is given by the integral along the river reach of open-water area multiplied by the rate of open-water ice production per unit area. The integral for total frazil ice production in the river reach upstream of any point along the reach gives

$$Q_i = (1 - \varepsilon)\ t_s\ V_s\ B\ N_i \qquad (3.22)$$

which is, of course, equal to the rate of surface ice flow at that point in the river reach.

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*Complete derivation for Dunvegan Power Project to be published. Report in preparation.

The frazil ice produced along the reach in this manner coalesces and rises to the surface to form slush pans. These pans occupy a fraction of the total surface area equal to, N_i, at a thickness, t_s, and porosity, ε. As the pans move downstream, they thicken through thermal growth nucleated by the frazil crystals. Employing the algorithm of Michel and Bérenger (1975) for ice growth in broken ship tracks where part of the surface is covered with ice, the thickness of ice grown in the water filled voids is given by

$$\Delta t_t = \frac{\varepsilon}{1 - \varepsilon} \left\{ t_s + \xi \left[\frac{\alpha}{294} \sqrt{\frac{\Delta T x'}{V_s}} - t_s \right] \right\} \tag{3.23}$$

where Δt_t = thickness of thermal ice grown in water voids (m)

α = thermal coefficient for Stefan Equation of thermal ice growth (see Section 4.1.2)

ΔT = air temperature difference from water freezing temperature (°C)

x' = distance travelled by slush pan (m)

ξ = coefficient determined by thermal growth relative to slush pan thickness.

If $\frac{\alpha}{294} \sqrt{\frac{\Delta T\ x'}{V_s}} \leqslant t_s$, $\xi = 1$ then

$$\Delta t_t = \frac{\varepsilon}{(1 - \varepsilon)} \frac{\alpha}{294} \sqrt{\frac{\Delta T\ x'}{V_s}} \tag{3.23a}$$

If $\frac{\alpha}{294} \sqrt{\frac{\Delta T\ x'}{V_s}} > t_s$, $\xi = \varepsilon$ then (3.23b)

$$\Delta t_t = \frac{\varepsilon^2}{(1 - \varepsilon)} \frac{\alpha}{294} \sqrt{\frac{\Delta T\ x'}{V_s}} + \varepsilon\ t_s.$$

The total volume of additional thermal ice produced along the reach to a point x is the sum or integral of thermal ice production in the open-water voids of slush pans travelling downstream, or thickness times area. It is assumed that, from the time frazil ice is generated at some point in the reach until it reaches the point x, thermal growth will take place in the open-water voids. The time to coalesce at the surface has been ignored. This integral is given by*

$$Q_t = \frac{Q^* \, B \, \varepsilon}{\gamma_i \, L_f \, (1 - \varepsilon) \, t_s \, x} \int_o^x (1 - e^{-cx}) \, \Delta t_t \, dx \qquad (3.24)$$

where $c = \dfrac{Q^*}{t_s \, V_s \, \gamma_i \, L_f \, (1 - \varepsilon)}$

As there is no definite integral solution, a computer solution of a series expansion is required.

The total volume of ice produced along an open-water reach is given by the summation of frazil ice production in open water and subsequent thermal ice growth, given by

$$Q_s = Q_i + Q_t. \qquad (3.25)$$

If the moving blanket of slush ice and thermal growth reaches a river closure where the Froude number is less than that critical for progression, the ice may pack against the closure in part or in total. The rate of upstream progression will be determined by the net rate of ice inflow to the leading edge of the ice cover. The volume of ice underturning and carried away from the leading edge is assumed to be equal to the under-ice transport capacity of the flow. This transport of ice

*Complete derivation for Dunvegan Power Project to be published. Report in preparation.

away from the leading edge was treated as an inverted sediment transport problem by Pariset and Hausser (1961). For the hydraulic radius equal to half the mean depth, the adapted Meyer-Peter formula is

$$3{,}281 \frac{V_u^2}{C^2} = 12.3\, d_s + 0.84\, q_u^{2/3} \tag{3.26}$$

where V_u = mean velocity under the ice cover (m/s)

C = Chezy roughness coefficient for water passage ($m^{1/2}/s$)

d_s = characteristic dimension of ice fragments taken as floe thickness or slush blanket thickness (m)

q_u = ice discharge per unit width under cover <u>weighed</u> under water with apparent density 0.08 (N/s m).

Rearranging and substituting t_s for d,

$$q_u = \left(3{,}920 \frac{V_u^2}{C^2} - 14.7\, t_s\right)^{3/2} \text{ N/s m.} \tag{3.26a}$$

The total transport is given by

$$Q_u = \left(3{,}920 \frac{V_u^2}{C^2} - 14.7\, t_s\right)^{3/2} \frac{B}{\frac{(\rho - \rho')g}{\rho}} \tag{3.27}$$

where Q_u = total under-ice transport (m^3/s)

B = channel width (m)

ρ = water density (kg/m^3)

ρ' = ice density (kg/m^3)

g = acceleration of gravity (m/s^2).

The cover will progress upstream at the thickness required, given by

$$t_m = \frac{V^2}{2g\,\frac{(\rho - \rho')}{\rho}} \qquad (3.13)$$

Individual floes thicker than t_m will come to rest against the leading edge of the cover. Thinner floes and slush in excess of the carrying capacity of the flow will pack at the leading edge at a porosity, ε', between the porosity of the slush blanket and the porosity of an accumulation of solid floes, depending on the amount of thermal ice present and the hydraulic thrust on the leading edge. So, while individually the floes are too thin to be stable, the leading edge will progress. [The porosity of a packed accumulation of ice fragments and slush reaches a minimum value of 0.4 according to Michel (1978).]

The rate of ice cover progression is given by

$$\Delta L = \frac{Q_s - Q_u}{B\, t_m\, (1 - \varepsilon')} \qquad (3.28)$$

The time required for ice cover progression through a river reach can be determined from the rate of cover progression accounting for decreasing ice inflow, Q_s, as the ice production area reduces with upstream cover progression.

The under-ice transport rate calculated by the Meyer-Peter method is noted to be no more accurate for ice transport than it is for sediment transport. The calculation depends on the characteristic dimension used to define the distribution of ice fragment sizes.

Due to the lack of field data on this size distribution, more simplified criteria may be adopted for frazil ice deposition.

In that case, all ice is assumed to pack at the leading edge at Froude numbers less than critical for cover progression. Conversely, at higher Froude number, all ice is carried under the cover at the leading edge. Where conditions are favourable, the ice is deposited under the ice cover increasing cover thickness and roughness. In some cases where very thick accumulations occur, a "hanging dam" is formed. A hanging dam may occur without a great head loss which distinguishes this phenomenon from an ice jam caused by unstable collapse of the cover and high head loss.

The simplest criterion for deposition of ice under an ice cover was a maximum velocity limit proposed by Newbury (1966). Based on observations on the Nelson River in Manitoba, frazil will be deposited at under-ice velocities less than 0.8 m/s. At velocities above 1.5 m/s, erosion of unconsolidated slush will occur.

Tesaker (1975) studied frazil ice accumulation under a solid ice cover for two winters at a Norwegian power plant. He found that frazil ice accumulated until the flow velocity reached 0.4 to 0.6 m/s. This corresponded to a limiting Froude number of the flow under the ice of 0.08 to 0.14 which compares with limiting Froude numbers for progression of an ice cover. At the suggestion of Lavender (1975), a densimetric Froude number defined as

$$F_r' = \frac{V}{\sqrt{g\,H\,\frac{(\rho - \rho')}{\rho}}} \qquad (3.29)$$

was examined. A densimetric Froude number accounts for the bouyancy of the particles which is important for stability. For ρ'/ρ of solid ice equal to 0.92, the densimetric Froude numbers are 0.28 to 0.49 corresponding to Froude numbers of 0.08 to 0.14, respectively. If a porous slush pan is considered (ε = 0.75) and ρ'/ρ equals 0.98, the densimetric Froude number equals 1.0 (corresponding to a Froude number of 0.14), a well known critical Froude value in other applications. To arrive at a useful and general stability criterion, more knowledge on the density of slush accumulations is required.

As discussed earlier, the Meyer-Peter approach has serious limitations related to the determination of a suitable length parameter for the transported particles. While this length selection is difficult for slush ice transport, the method is even less appropriate for solid ice pans with surface dimensions much different from pan thickness. Uzuner (1975) analyzed the stability of solid ice pans under an ice cover and the rate of transport. The analysis paralleled the stability of prismatic blocks resting on a river bed. The stability criterion for block transport obtained from moment equilibrium is given by

$$Fr_{it} = \frac{V}{\sqrt{\left(1 - \frac{\rho'}{\rho}\right) g\, t_e}} = \left[1.596 - \frac{1.277}{\left[5.8 + \frac{H}{t_e} - \left(1 - \frac{\rho'}{\rho}\right)\right]^{1/3}}\right] \frac{\ell_b}{t_e} \left[1 - \left(1 - \frac{\rho'}{\rho}\right) \frac{t_e}{H}\right] \quad (3.30)$$

where Fr_{it} = densimetric Froude number at incipient block transport

V = flow velocity upstream of the ice cover (m/s)

ρ' = ice density (kg/m^3)

ρ = water density (kg/m^3)

g = acceleration due to gravity (m/s^2)

t_e = equilibrium thickness of fragmented uniform ice cover (m)

H = depth of approach flow (m)

ℓ_b = length of ice block (m).

A lack of field data to substantiate this criterion limits its application.

At present, accumulation of sluch ice and floes should be determined from the simple limiting velocity criteria until field data have been collected to verify the analytical criteria for slush and block transport.

3.2.6 - Backwater with Ice Cover

The effect of an ice cover on the slope of the energy gradient is two-fold. In a wide river, the hydraulic radius is equal to half the mean flow depth under the cover rather than the full flow depth. Secondly, the composite roughness of the flow boundary is often rougher than the natural channel bed roughness when a packed cover of slush and floes is formed. An additional factor in determining stage is the hydrostatic water level within the ice cover.

Numerous researchers have attempted to derive a composite roughness Manning's n-value or Darcy-Weisbach friction factor to account for the additional roughness of the ice-covered channel. An excellent summary of this research is provided by Uzuner (1975) who concluded that the derivation of Larsen (1973) was the most complete and rigorous. In a similar summary by Pratte (1976), that conclusion was also reached. However, Pratte noted that the simpler derivation of Torok-Sabaneev (1948) was more appropriate for engineering use. The composite roughness of the cross section is given by

$$n_c = \left(\frac{(n_i)^{3/2} + (n_b)^{3/2}}{2} \right)^{2/3} \quad (3.31)$$

$$n_i = \left[2 n_c^{3/2} - n_b^{3/2} \right]^{2/3}$$

where n_i = Manning's n-value under the ice cover

n_b = Manning's n-value of the channel bed.

For the ranges of roughness encountered, the formulation of Larsen simplifies to this equation. Furthermore, insufficient field data exist to support the assumptions of the various researchers. In application, the errors introduced by selection of single bed and ice roughness values outweigh the errors in any analytical method. Pratte discusses application of composite roughness in channels with variable bed characteristics. Without detailed field measurements of cross section bed profile and ice thickness variation, a simple approach is necessary and adequate.

The most complete compilation of field data on Manning's n-values is reported in Nezhikhovskiy (1964). For smooth ice covers with no loose ice accumulated underneath, the values were found to be consistently

$0.010 < n_i < 0.012$ (at beginning of freeze-up)

$0.008 < n_i < 0.010$ (middle of winter).

The tendency for covers to become smoother as winter progresses is due to the greater heat transfer caused by turbulence around projections under the cover combined with faster growth rate in thinner sections.

Carey (1966, 1976), Ashton and Kennedy (1972) and Larsen (1973) investigated the formation of ripples on the underside of smooth ice covers. As heat transfer changes near the end of winter, the smooth ice cover may erode creating a ripple pattern. The roughness increases to 0.014 in some cases.

For ice covers with loose ice accumulations underneath, roughness values were classified by type and depth of accumulation as follows

- frazil slush
- compact frazil slush with small ice floes
- ice floes.

Following formation of accumulations at freeze-up, n-values were as given in Table 3.3. Very rapid decreases in this roughness have been noted, although a gradual decrease in roughness through the winter is normal. On the Dniester River, the n-value dropped from between 0.07 to 0.08 to a final value of 0.008 to 0.0012 in 15 to 25 days. The decrease in roughness varies dramatically from year to year even on one river, depending mainly on meteorological conditions but also on ice material in the accumulation. As expected, solid floes decrease in roughness more quickly than loose frazil slush.

TABLE 3.3

VALUES OF ICE COVER ROUGHNESS, n_i, WITH UNDER-ICE ACCUMULATIONS (from Michel, 1971)

Initial Ice Accumulation Thickness (m)	Type of Accumulation: Frazil Slush	Compact Slush With Ice Fragments	Ice Floes
0.1			0.015
0.3	0.01	0.013	0.04
0.5	0.01	0.02	0.05
0.7	0.02	0.03	0.06
1.0	0.03	0.04	0.07
1.5	0.04	0.06	0.08
2.0	0.04	0.07	0.09
3.0	0.05	0.08	0.10
5.0	0.06	0.09	

Michel (1971) notes that spring jams formed from solid floes had lower values of n_i than for freeze-up accumulations of solid floes. Therefore, a value of n_i for compact frazil slush and small ice floes is recommended for spring jams by Michel.

Beltaos (1978a, 1978b) investigated spring jams at breakup and concluded that the freeze-up values reported by Nezhikhovskiy represented extreme values of roughness. The calculation of composite n-value with the Torok-Sabaneev equation may not be applicable for extreme roughness, the detailed fluid mechanics of which is not known at present.

The determination of composite roughness for an ice-covered channel is limited by a lack of field data. The roughness values listed provide some guidance, but an engineer requires a great deal of experience to determine backwater effects under ice-cover conditions, even when detailed open-water levels and cross section data are available.

3.3 - Application to Variable Flow and Water Level

The preceeding formulas have been developed for simple steady, uniform flow conditions. For mechanical stability, the unfrozen cover can react to variable water levels and hydraulic forces. Thus, stability calculations should be made at extreme discharges and water levels. For thermal calculations, the effect of discharge variation is cumulative, and average values should be used.

4 - MIDWINTER

The characteristics of the ice cover formed in a river reach are determined by hydrodynamic and meteorological conditions at freezeup. In high velocity reaches, a thick, packed cover is required to develop sufficient internal strength to resist the stresses imposed by forces acting on the cover.

The strength of the cover increases greatly after the cover begins to consolidate and then a much thinner cover is required for stability. In rivers where winter flow is typically low and relatively constant, the ice regime is dominated by thermal processes.

A floating, fragmented ice cover can respond to changes in discharge, attaining the required thickness and level for stability. A solid cover has greater strength but does not have the same degree of flexibility to adapt to changes in water level. Large and rapid water level variations caused by hydroelectric peaking or spring runoff can break up the solid cover. High discharge and the sudden reduction in strength from that of a solid cover to that of a fragmented cover can result in the most severe ice jams. Variation of river discharge must be regulated tc ensure that support of the cover at the river banks is maintained and rupture of the solid cover is avoided.

4.1 - Midwinter Thermal Processes

4.1.1. - Extent of Open Water

Heat balance on open water and frazil ice generation, discussed in Section 3.1, determine the time for cover formation and extent of open water. The cover progresses upstream covering the supercooled section of the river reach and frazil generation ceases. The leading edge position migrates upstream and downstream with changing meteorological conditions following the position of freezing water temperature. Donchenko (1975) gives examples of rivers in Russia where the position of the leading edge varies by as much as 60 km throughout one winter season as a result of variations in meteorological conditions and flow regulation. Progression of the cover can be calculated from the formulas in Sections 3.1.1 and 3.1.2. Recession of the leading edge can be calculated from the formulas in following sections.

4.1.2 - Solid Ice Cover Growth

The growth of an ice cover depends upon the net rate of heat transfer from the cover. Heat is taken away from the cover to the atmosphere and normally heat is transfered to the cover from the flow. Many factors influence this heat transfer, as will be discussed later.

It has been found in practice, however, that ice cover growth can be reasonably represented by a simple relationship originally derived for lakes and reservoirs by Stefan and simplified by others (see Michel 1971). The formulation is

$$t_i = \overset{0.0341}{\cancel{341}}\,\alpha \sqrt{D} \qquad (4.1)$$

where t_i = thickness of solid ice (m)

α = a form of local heat transfer coefficient

D = accumulated degree-days of freezing (°C-days).

The coefficient α is obtained from experience to represent the time average values of the various physical and thermal properties of the ice and water, as well as the highly variable and complex heat transfer between the ice and the atmosphere. Heat transfer at the ice/water interface is also considered in the selection of α. Values of the coefficient, derived from field experience, are

windy lakes with no snow	$\alpha = 0.8$
average lake with snow	$\alpha = 0.5$ to 0.7
average river with snow	$\alpha = 0.4$ to 0.5
sheltered river with rapid flow	$\alpha = 0.2$ to 0.4.

In spite of the extreme simplification of this technique, the method has been used successfully. However, this relationship predicts growth of the ice cover as long as the degree-days of freezing continue to accumulate and heat is extracted from the underlying body of water. This implies that ice cover growth is only limited by duration of winter when, in fact, heat inflow from the under surface may limit total ice growth. This applies particularly to recession of the leading edge of the cover when melting of the undersurface begins.

A complete analysis of heat transfer at the ice/water interface can be performed to determine ice growth. The starting assumption is a constant temperature of 0°C at the under surface of the ice and continuous melting or freezing depending on the direction of net heat transport. The constant temperature of 0°C facilitates heat transfer calculations by making transfer processes on either side independent of each other.

Heat transfer from the lower ice surface to the atmosphere occurs in two stages. Heat is first conducted to or from the lower ice surface as described by

$$\dot{q}_c = \frac{k_c}{t_i} (T_f - T_s) \qquad (4.2)$$

where $\dot{q}_c$ = rate of heat transfer (J/m^2 s)

k_c = conduction coefficient (J/°C m s)

t_i = ice cover thickness (m)

T_f = freezing temperature of water (0°C)

T_s = upper ice surface temperature (°C).

Transfer of heat from the upper surface to the atmosphere occurs by convection and radiation dependent on such factors as wind speed, air-ice temperature difference and ice properties. Meteorological conditions have the strongest influence on heat transfer.

In general, it is possible to assume that the upper surface is at air temperature and calculate heat transfer from conduction alone. Baines (1961) notes that this practice is equivalent to assuming an overcast sky condition and a strong wind, whereas the average winter condition is one of moderate sunshine and light wind which tends to produce thinner covers.

The validity of the assumption is supported, however, by data collected on the Nelson River as reported by Newbury (1966). Newbury's ~~analysis of the~~ data showed independence of ice growth from wind conditions and a transfer coefficient very near the generally accepted value for conduction through ice (i.e., 2.18×10^4 J/m^2 s °C per meter of thickness). Thermal inertia in the cover no doubt figures significantly in making the approximation reasonable, particularly when using average temperatures over a period of one week or more.

Heat transfer from the flow in a river to the ice cover is a function of the thermal and flow characteristics of the river. The heat transfer is comprised of three components; terrestrial heat flow, q_t, internal friction, q_f, and cooling of the river flow, q_w. The total heat flow to the cover, $\dot{q}_i$, is the sum of these components.

$$\dot{q}_i = q_t + q_f + q_w \tag{4.3}$$

Very little data is available for the terrestrial heat flow. Initially, the return flow of heat stored in the river bed during summer inflow is high. McLachlan (1926) provides estimates of 17.2 - 19.7 J/m^2 s in the St. Lawrence River early in winter. In late winter, just prior to break up, the heat inflow is typically 2.09 - 2.51 J/m^2 s. Nybrant (1945) estimated corresponding rates of 11.72 J/m^2 s in October to zero in April for pond studies in Sweden. A rough average of 4.61 J/m^2 s based on Nybrant's work is recommended for an initial estimate of ice cover thermal equilibrium. Thus,

$$q_t \simeq 4.61 \text{ J/m}^2 \text{ s} \tag{4.3a}$$

The heat from internal friction can be calculated from the head loss

$$q_f = \frac{Q\gamma S}{BJ} \tag{4.3b}$$

where q_f = heat from internal friction (J/m^2 s)
- Q = river discharge (m^3/s)
- γ = specific weight of water (N/m^3)
- S = slope of energy gradient equal to slope of hydraulic grade line in uniform flow
- B = river width (m)
- J = thermal equivalent = 1.00 N-m/J.

Heat from cooling of river flow is given by

$$q_w = -\frac{Q}{B} c_p \, \rho \, \frac{dT_w}{dx} \tag{4.3c}$$

where q_w = heat available from cooling flow (J/m^2 s)

c_p = specific heat of water (1 J/kg °C)

ρ = specific mass of water (kg/m^3)

T_w = water temperature (°C)

x = distance downstream along the river (m).

With temperature measurements taken under ice-covered conditions, the cooling of river flow and the rate of change of ice cover thickness can be calculated from

$$\frac{dt_i}{dt} = \frac{\dot{q}_c - \dot{q}_i}{L_f \ \gamma_i} \tag{4.4}$$

where $\dot{q}_c$ = rate of heat transfer by conduction through the ice cover (J/m^2 s)

$\dot{q}_i$ = rate of heat transfer from the flow to the ice cover (J/m^2 s)

If water temperature measurements or estimates are available, the rate of ice cover growth can be calculated. Integration of the differential equation yields the change in ice cover thickness

$$\Delta t_i = \frac{\overline{(\dot{q}_c - \dot{q}_i)}}{L_f \ \gamma_i} \ t \tag{4.4a}$$

where $\overline{\dot{q}_c - \dot{q}_i}$ = time average net heat inflow to the ice cover determined from average flow and air and water temperatures

t = time for change in ice thickness (s).

In a long river reach where equilibrium heat flow to and from the water is achieved, water temperature will be determined by total heat flow expressed as

$$\dot{q}_i = h_e \ \Delta T_e = q_t + q_f \tag{4.5}$$

where h_e = convective heat transfer coefficient (J/m^2 s °C)

ΔT_e = equilibrium temperature difference from freezing (°C).

Convective heat transfer from the flow to the smooth under surface of the cover can be described by the Reynold's Analogy (Baines 1961, Ashton 1973, Cowley and Lavender 1975)

$$Nu = \frac{f}{2} Re^{a} Pr^{1/3} \tag{4.6}$$

where the Nusselt number, $Nu = h_e \ell / k_f$

the Reynolds number, $Re = V_u \ell \rho / \mu$

the Prandtl number, $Pr = c_p \mu / k_f = 13.6$ for water at 0°C

a = an exponent equal to 0.8 or 1.0

and f = friction coefficient.

For the dimensionless ratios

h_e = convective heat transfer coefficient (J/m^2 s °C)

c_p = specific heat of water (J/kg °C)

ρ = specific mass of fluid (kg/m^3)

k_f = thermal conductivity of the fluid (J/m s °C)

ℓ = characteristic length (m)

μ = absolute viscosity ($kg\ s/m^2$)

V_u = average velocity under the ice cover (m/s).

The Nusselt number provides a measure of the relative importance of heat transfer by turbulent mixing and simple conduction. The Reynolds number characterizes flow turbulence. The Prandtl number represents thermal properties of the fluid.

For a two-layer flow, the layer depths are proportional to the shear coefficients of the respective layers.

For a wide channel, the hydraulic radius of each layer is approximately equal to the layer depth. In the Reynolds Analogy the friction factor is equal to one-fourth of the Darcy-Weisbach friction factor which can be related to the Manning coefficient by

$$f = \frac{f_D}{4} = \frac{19.6\ n_i^{\,2}}{R_i^{\,1/3}} \qquad (4.6a)$$

where n_i = Manning's n-value for the ice cover

R_i = hydraulic radius of upper flow layer (m).

In natural river channels where flow is always turbulent, the Manning equation is applicable. Controversy in applying the Reynolds Analogy is related to the appropriate exponent for the Reynolds number which depends on the treatment of shear stress. For example, Baines, Cowley and Lavender suggest favour- the use of the exponent "a" equal to 1.0 while Ashton and Michel argue that an exponent of 0.8 is appropriate for fully rough turbulent flow.

A practical limitation to the heat transfer determination is the determination of the appropriate n-value and hydraulic radius for the unknown flow distribution. Much field and laboratory work is required before the proper form can be determined. It is sufficient at this point to solve the Reynolds Analogy in the form

$$N_u = \frac{9.8\ n_i^{\,2}}{R_i^{\,1/3}}\ Re\ Pr^{1/3} \qquad (4.6b)$$

The layer depths of the upper (ℓ_i) or lower (ℓ_b) layers are proportional to the friction factor or the square of the corresponding n-value. It follows that the ratio of the layer thickness can be written as

$$\frac{\ell_i}{\ell_b} = \left(\frac{n_i}{n_b}\right)^2 \tag{4.7}$$

The total flow depth under the cover is given by

$$H = \ell_i + \ell_b \tag{4.7a}$$

Thus, $\ell_i = H/[1 + \left(\frac{n_b}{n_i}\right)^2]$ (4.7b)

An estimate of convective heat transfer coefficient can be obtained from

$$h_e = 1.73 \times 10^6 \; [1 + \left(\frac{n_b}{n_i}\right)^2]^{1/3} \; \frac{V_u \, n_i^{\,2}}{H^{1/3}} \tag{4.8}$$

for $\mu/\rho = 1.78 \times 10^{-6}$ m^2/s

$k_f = 0.566$ J/m s °C

Pr = 13.6 for water at 0°C.

The instantaneous local growth rate of the ice cover can then be determined from

$$\frac{dt_i}{dt} = \frac{h_e \, (T_w - 32) - \frac{k_c}{t_i} \, (32-T_a)}{\gamma_i \; L_f} \tag{4.9}$$

If this term is negative, melting of the cover occurs and water temperature falls. As t_i becomes very small, the melting rate becomes infinite and the cover recedes downstream. The melting also transfers heat from the flow which brings the water temperature down to the freezing point. A new equilibrium may be established downstream.

4.2 - Mechanical Stability of a Solid Ice Cover

Research on the mechanical stability of ice covers has been devoted to the stability of fragmented covers at formation and breakup. Hydroelectric power plant operation is usually modified to conform to the stability criteria required for stable cover formation. After the ice cover has solidified, normal operation is resumed.

In some cases, rapidly varying velocity and water level changes associated with peaking operations continually cause breakup of the cover. As there are no general stability criteria available to guide winter peaking operation, this procedure must be determined by trial-and-error or suspended for the duration of the winter.

The criteria presented here have yet to be verified by field observations. They will not be suitable for design or operational guidelines until field and laboratory programs have been conducted to establish coefficient values.

4.2.1 - Solid Covers - General Stability

Michel (1975) reported the results of experiments conducted on the stability of a solid ice cover. The study involved laboratory tests using a poured wax layer floating on the water surface to simulate a solid ice layer subjected to hydraulic forces of flowing water under the cover. The experimental tests yielded two criteria; a limiting Froude number of 0.275 for the stability of the leading edge of the cover and a limiting Froude number for failure of the entire cover by a process of "oscillatory bending". This Froude number is defined below.

$$Fr = \frac{V}{\sqrt{gH}} = 0.113\, e^{-\left[\frac{\sigma_b\, t_i}{g\, (\rho - \rho')\, B^2}\right]} \qquad (4.10)$$

where V = flow velocity under open water conditions (m/s)
g = acceleration due to gravity (m/s^2)
H = open water flow depth (m)
σ_b = bending strength of the ice cover (N/m^2)
t_i = ice cover thickness (m)
ρ = water density (kg/m^3)
ρ' = ice density (kg/m^3)
B = channel top width (m).

The results were obtained in a small-scale model of a prismatic channel. The range of parameters tested was limited, particularly with respect to channel geometry. The effect of water level variation causing failure of the cover at the channel banks was not considered. Further testing and field verification are required before the criteria can be used with confidence.

Only qualitative descriptions of solid ice cover disruption at hydroelectric peaking generating stations could be found so verification of these criteria with field observations could not be made.

4.2.2 - Solid Covers -
Shoreline Hinging

A hinging failure mechanism of the ice cover contact at the river banks caused by large variations in stage related to hydroelectric power plant peaking operations was identified in a field study by Fonseca (1979).

The mechanism is characterized by a minimum number of three failure lines in the ice cover parallel to the river banks. Repeated failures and reformation of the ice cover at varying water levels produce a cover width dimension greater than the horizontal width of the river water surface at maximum water level. The central portion of the cover remains intact. With the rise and fall of the ice cover following changes in water level, the fragments or strips of ice parallel to the river banks form an inverted arch which supports the central portion of the ice cover, illustrated in Figure 4.1. The "hinges" between the fragments, the solid ice cover and immobile ice in contact with the river bank were observed to break or crack with each cycle in the water level.

Kartha (1977) noted the formation of hinge cracks on the Winnipeg River. A series of tests were conducted to investigate ice cover stability under peaking operation conditions. Measurements of water and ice cover levels, and observations of cover stability were made for three peaking discharge ranges at seven river cross sections. Unfortunately, detailed observations of the hinging mechanism were not given.

Hinging is mentioned specifically in a report by Donchenko (1975). It is not clear if the same phenomenon is considered for this definition of shoreline contact.

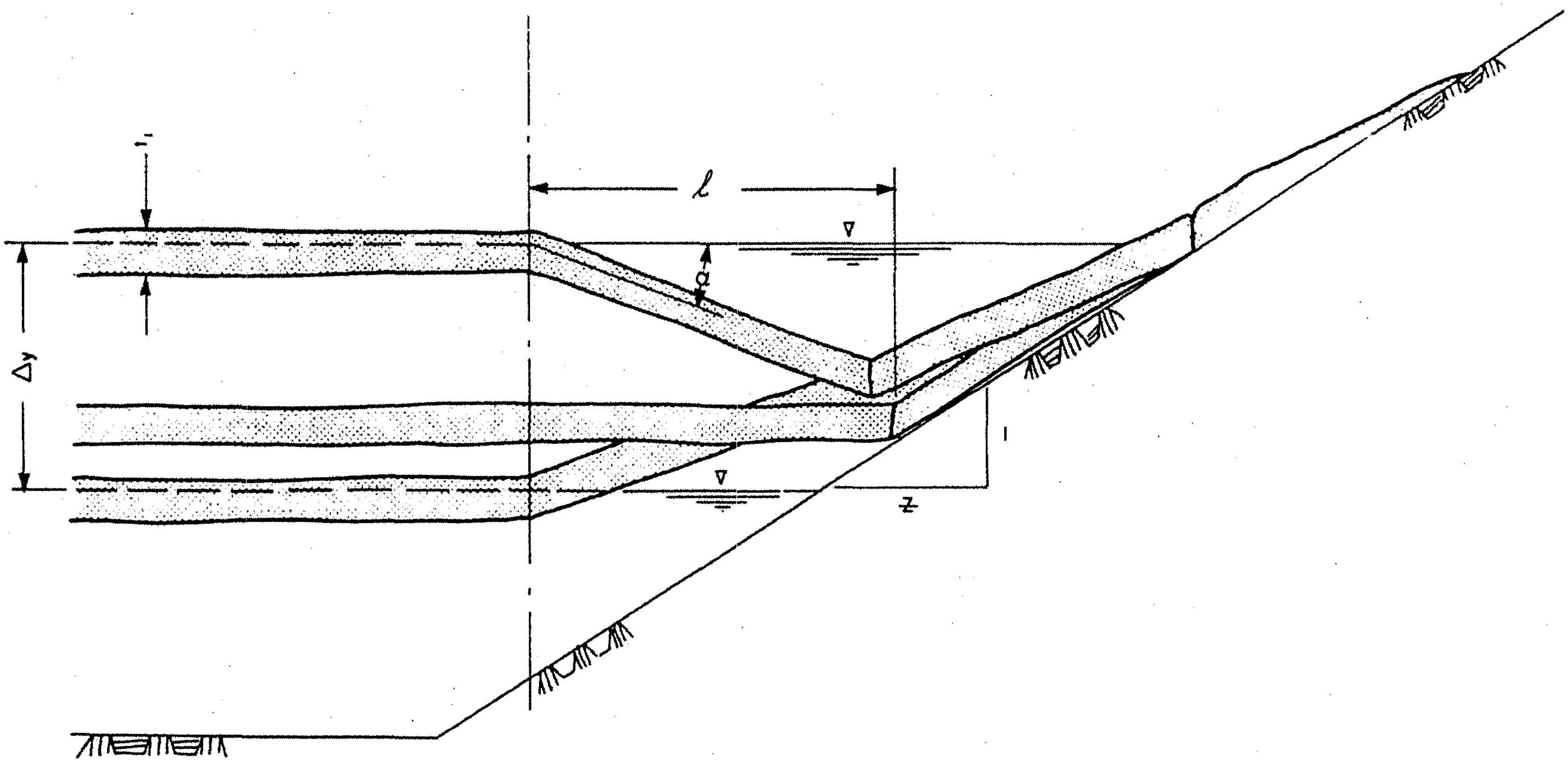


FIG 4.1

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

DEFINITION SKETCH OF HINGING MECHANISM

ACRES

Initial Cover Failure

After the formation of a solid cover at a fixed water level, the inital failure occurs at the shoreline. The rise or fall in water level creates a distributed load in the cover due to the difference between ice weight and buoyancy of the partially submerged cover. Bending failure at the shoreline occurs if the water level exceeds a maximum value. After the initial failure at the shoreline, a second failure occurs at a point offshore. This failure sequence has been observed by Shaitan (1967) and Sinotin and Sokolov (1968).

The initial peaking operation following cover formation will determine the failure lengths for hinge link formation. Subsequent failures will occur at the initial failure points where the cover is weak (Michel 1970).

Various formulas based on elastic and plastic analysis have been presented for critical water level variation at cover failure and vertical loads on structures caused by water level variation (Michel 1970, Shaitan 1967, Donchenko 1975). However, attempts to analyze this phenomenon have been hampered by a lack of knowledge of the basic creep properties of ice (Michel 1970). The failure of the ice cover is highly dependent on the rate of loading, meteorological conditions, and frequency and range of water level variations. Shoreline configuration and ice adhesion to the shoreline play a significant role in the failure as well. The strength of adhesion can be as strong as the shear strength of ice itself for contact with concrete, steel or timber structures. For natural or armoured riverbanks, the adhesion will be reduced. The strength of the contact depends on the depth of freezing of the riverbank (Rossinskij and Nisar-Muhamedova, 1977).

The generalized formulas do not account for the highly variable conditions encountered in nature. In fact, with simplifying assumptions, the form of the equations derived for plastic and elastic failure is the same so the difference in the application of the formulas is the choice of elastic or plastic ice strength for the loading conditions.

The problem is further complicated by the lack of appropriate field data. As Tsang (1975) notes, there are no techniques at present to estimate bending and compression strength in the field from readily available meteorological and hydrological data. These formulas will be of limited use until standardized testing methods are adopted such as those suggested by the Working Group of the IAHR Section on Ice Problems (1980). Calculations do support observations of initial failure for water level variation less than the ice thickness.

Finally, there are no criteria for the second failure of the ice cover which determines the hinge link lengths. These lengths are important for the overall stability of the solid ice cover.

The engineering capability to predict the initial cover failure is inadequate at present. A systematic program of field data collection and observations is required to guide the analysis of the phenomenon. However, for the determination of overall ice cover stability, a rigorous theoretical solution is not required because

- the initial failures of the cover are only important for determination of hinge link lengths

- it will be impractical to maintain solid ice cover contact with the shoreline when failure occurs for water level variation less than ice thickness.

Therefore, the recommended goal at this time is to establish a simple empirical correlation between hinge link length and failure water level variation which depends on ice thickness and basic meteorological conditions (such as antecedent degree-days of freezing/thawing as an ice strength index). This relationship can be calibrated in the field with simple data requirements.

Hinge Stability

On the basis of preliminary analyses of the hinging mechanism, failure of the hinge through buckling of the individual fragments or crushing of the ice was rejected because of the narrow parameter ranges for conditions critical to stability. Subsequent analysis of the hinge development utilizing a solid mechanics approach has yielded the following criteria for stability of hinge sections. Details of the analyses are given in Appendix A.

The stability of the hinge depends on the frictional or shearing resistance of the hinge joints to changing buoyant force resolution on the hinge members. For a statically determinant solution, a frictional resistance approach was adopted. If only one hinge link is free to move, as shown in Figure 4.2, the limiting water level variation for stability is given by

$$\frac{\Delta y}{\ell} = \sin \alpha_{max} \tag{4.11}$$

where α_{max} is the maximum angle from horizontal for the hinge link which is in turn given by

$$\alpha_{max} = \tan^{-1} \left(\frac{\mu_h}{2}\right) \tag{4.11a}$$

where μ_h = a frictional or contact coefficient reflecting the strength of the fractured and rotating hinge joint.

The hinge link length at formation is required for substitution into the above relationship.

The stability of the one-link hinge reveals the basic stability relationship at the hinge joint. However, from a practical standpoint, one-link hinge stability cannot be achieved without elongation of the center section of the cover or the hinge link. Two or more links are required to maintain a continuous connection between the shoreline and free-floating center section of the ice cover as the water level fluctuates. Various configurations are shown in figures 4.3 and 4.4.

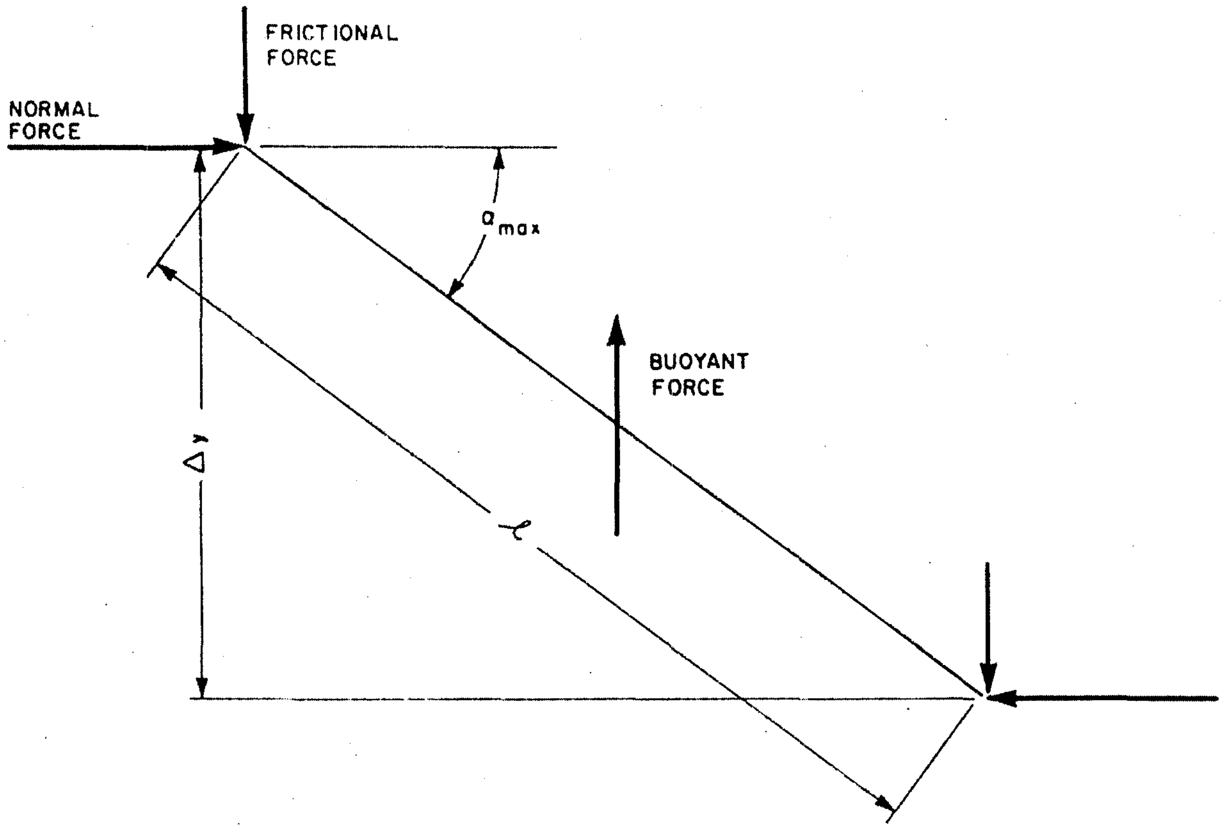


FIG. 4.2

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

STABILITY OF AN INDIVIDUAL HINGE LINK

AGRES

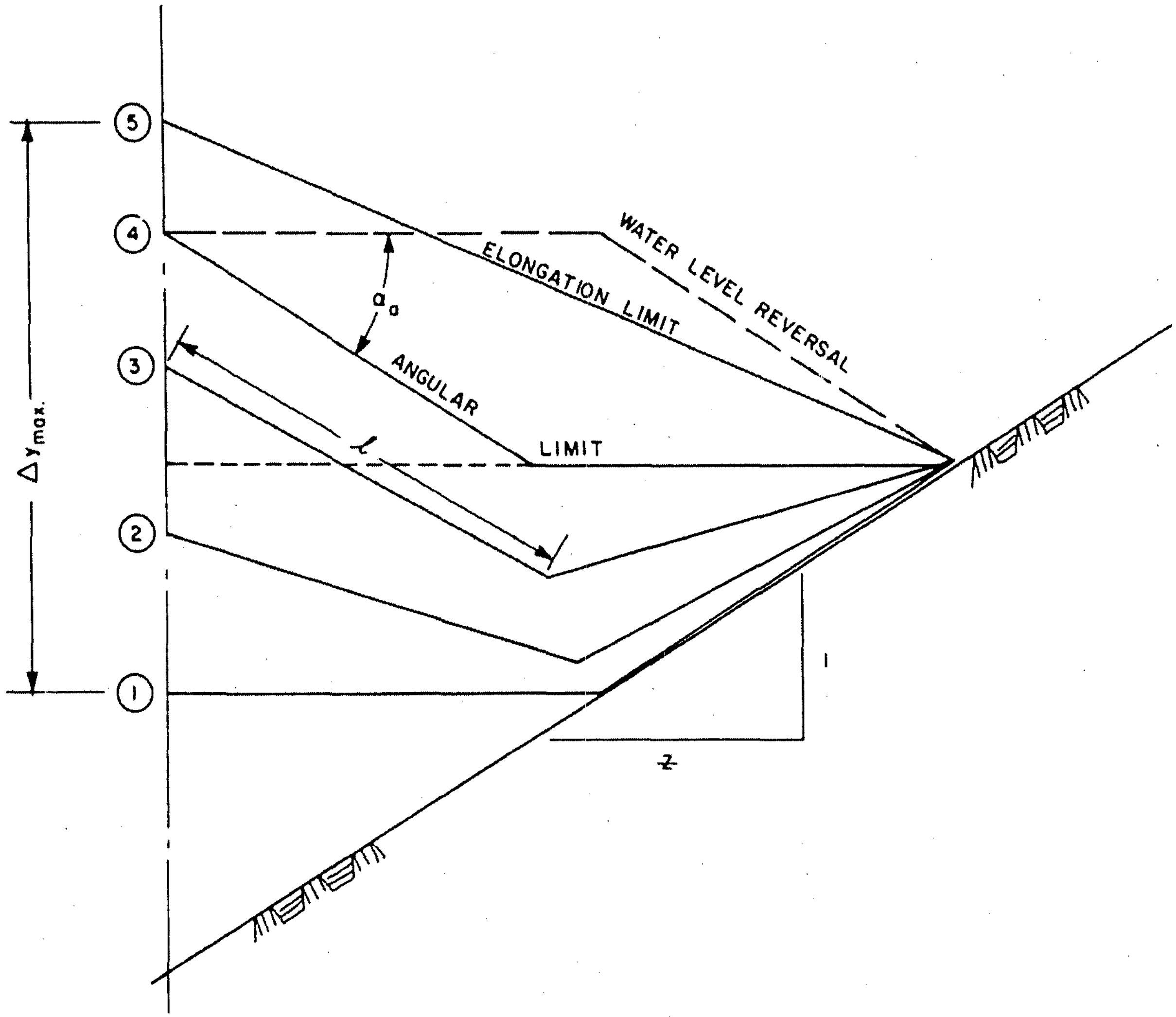


FIG 4.3

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

SKETCH OF TWO-LINK HINGE CONFIGURATIONS FOR RISING WATER LEVEL

ACRES

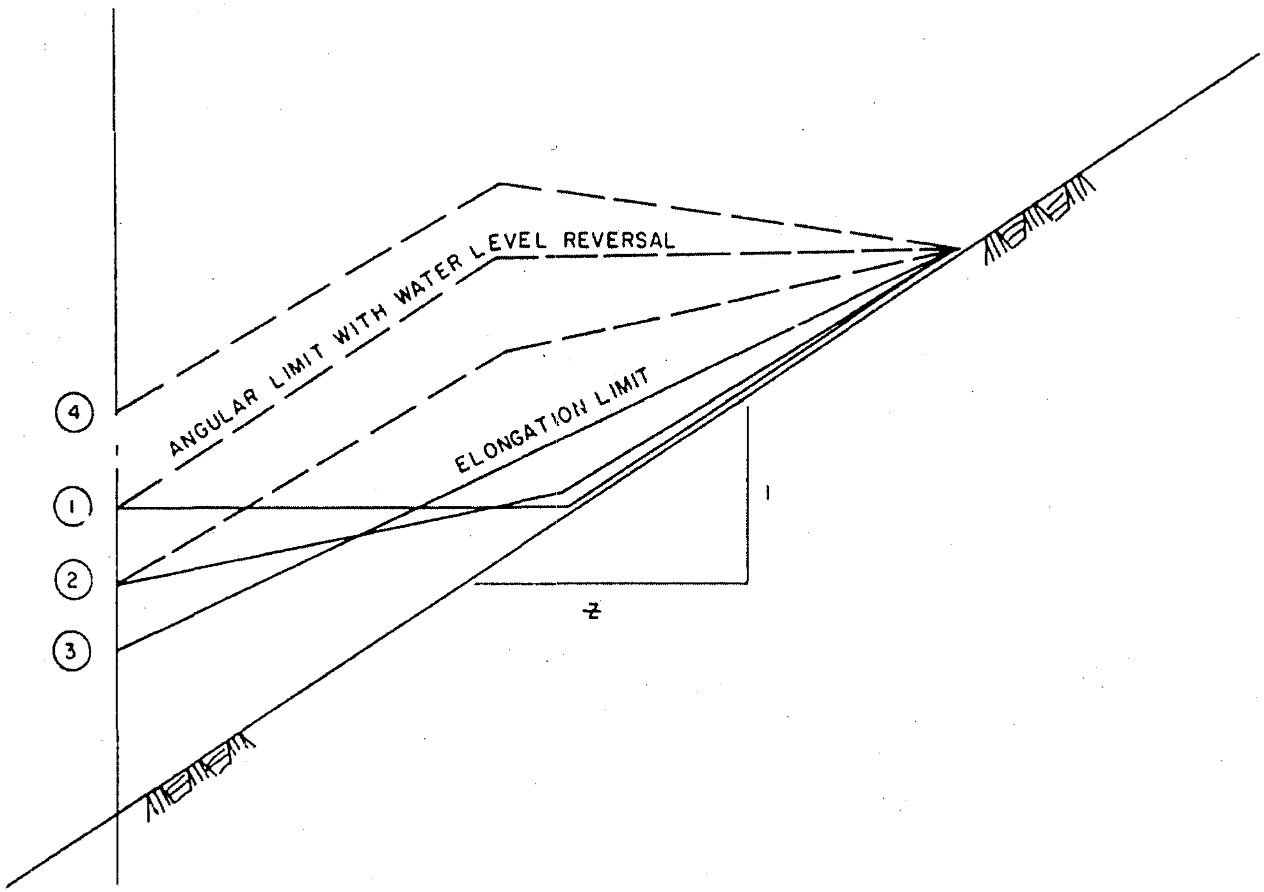


FIG 4.4

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

SKETCH OF TWO-LINK HINGE CONFIGURATIONS FOR FALLING WATER LEVEL

ACRES

Rising and falling water level as they relate to hinge stability are defined in terms of the arbitrary starting position for hinge analysis shown in figures 4.3 and 4.4. In this position, one hinge link lies on the channel bank and the other link is horizontal. A rising water level causes the central section of the ice cover to move above the horizontal link level. The horizontal link follows the cover movement and the second link rotates off the channel bank.

If two hinge links are free to move, the limiting criteria become more complex. Based on the basic stability of a one-link hinge and geometric relationships for bank slope, hinge link lengths and link angles, the limiting water level variation for hinge stability for a rising water level

is given by

$$\frac{\Delta y}{\ell} = C_{r1} \pm \sqrt{C_{r1}^{\,2} - 2\, C_{r2}} \qquad (4.12)$$

where

$$C_{r1} = \sin \alpha_{max} + \frac{1}{\sqrt{z^2 + 1}}$$

$$C_{r2} = 1 - \cos \alpha_{max} + \frac{(z + \sin \alpha_{max} - z \cos \alpha_{max})}{\sqrt{z^2 + 1}}$$

z is the horizontal component of bank slope for a unit vertical rise.

For a falling water level

$$\frac{\Delta y}{\ell} = C_{f1} \pm \sqrt{C_{f1}^{2} - 2\, C_{f2}} \qquad (4.13)$$

where

$$C_{f1} = \sin \alpha_{max} - \frac{1}{\sqrt{z^2 + 1}}$$

$$C_{f2} = 1 - \cos \alpha_{max} + \frac{(z - \sin \alpha_{max} - z \cos \alpha_{max})}{\sqrt{z^2 + 1}}$$

Hinge stability depends on the length of hinge link for conditions at the initial failure of the ice cover.

The criteria for hinge stability are summarized in Table 4.1. In order that the criteria for rising and falling water level could be compared, plots were prepared for the geometrical relationship between $\Delta y/\ell$, α and z.

Hinge link stability can be expressed in terms of water level variation and ice cover thickness by substitution of the appropriate hinge link length for rising or falling water level at formation.

TABLE 4.1

SUMMARY OF SOLID ICE COVER STABILITY FOR VARIABLE WATER LEVEL

Hinge Stability

One-Link Hinge	Two-Link Hinge
$\frac{\Delta y}{\ell} =$	$\frac{\Delta y}{\ell} =$

Rising Water Level

$\sin \alpha_{max}$

$$C_{r1} \pm \sqrt{C_{r1}^{2} - 2C_{r2}}$$

where

$$C_{r1} = \sin \alpha_{max} + \frac{1}{\sqrt{z^2 + 1}}$$

$$C_{r2} = 1 - \cos \alpha_{max} + \frac{(z + \sin \alpha_{max} - z \cos \alpha_{max})}{\sqrt{z^2 + 1}}$$

Falling Water Level

$\sin \alpha_{max}$

$$C_{f1} \pm \sqrt{C_{f1}^{2} - 2C_{f2}}$$

where

$$C_{f1} = \sin \alpha_{max} - \frac{1}{\sqrt{z^2 + 1}}$$

$$C_{f2} = 1 - \cos \alpha_{max} + \frac{(z - \sin \alpha_{max} - z \cos \alpha_{max})}{\sqrt{z^2 + 1}}$$

For rising water level, maximum water level variation is governed by both the frictional stability of the joint and the physical limitation of hinge elongation. As the water level increases, it can be seen in figures 4.3 and 4.5 that α, the angle from horizontal of the hinge link, increases to a maximum value α_a and then decreases until the point of hinge elongation occurs. For each value of z there is a corresponding maximum value of α_a that can be achieved, given by

$$\sin \alpha_a = \frac{1}{\sqrt{z^2 + 1}} \tag{4.14}$$

This corresponds to

$$C_{r1}^{\ 2} - 2\ C_{r2} = 0$$

hence,

$$\frac{\Delta y}{\ell} = \sin \alpha_a + \frac{1}{\sqrt{z^2 + 1}} = \frac{2}{\sqrt{z^2 + 1}} \tag{4.15}$$

The angular limit line is shown in Figure 4.5. The physical significance is illustrated in Figure 4.3 also.

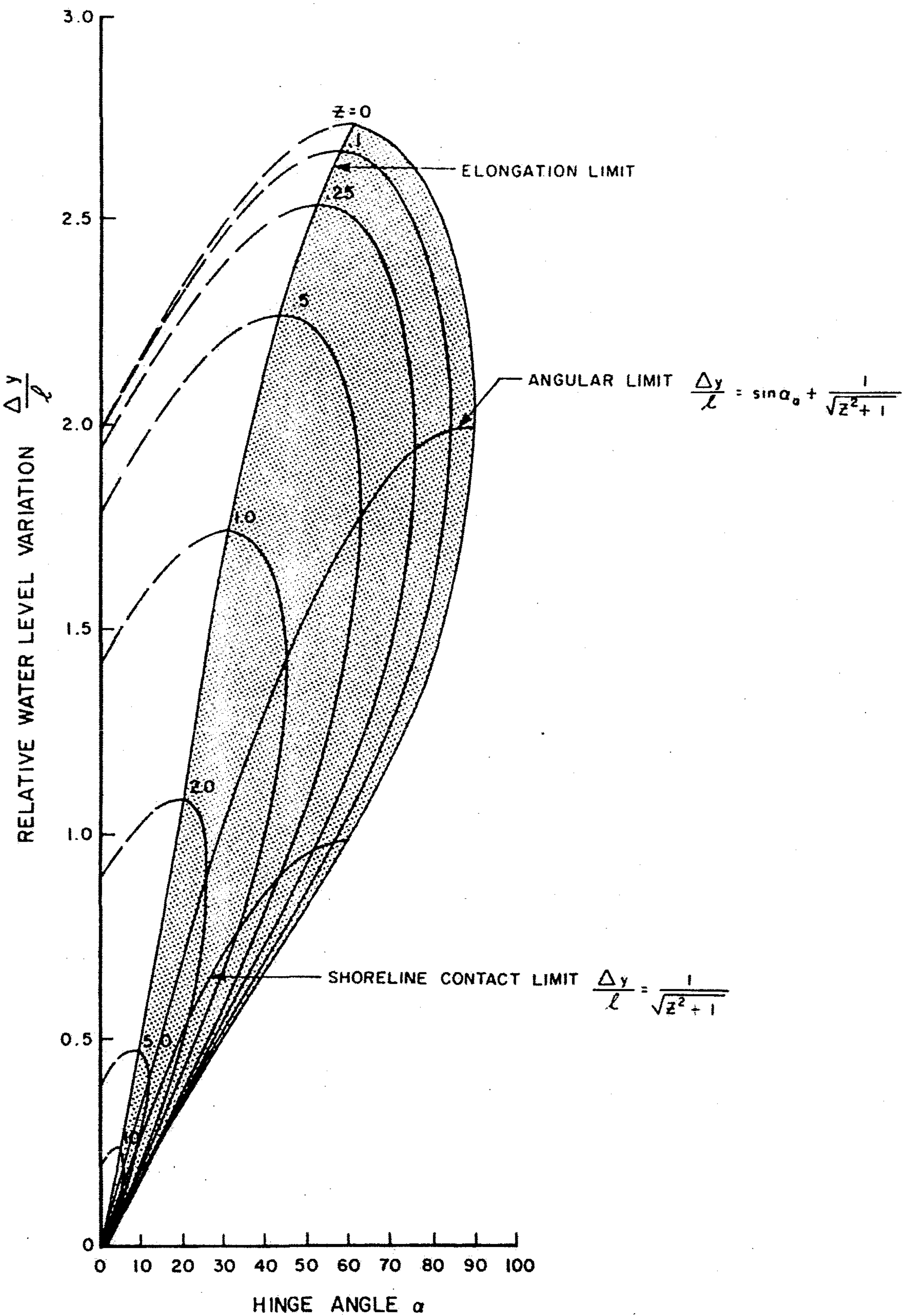


FIG. 4.5

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

TWO – LINK HINGE CONFIGURATIONS FOR RISING WATER LEVEL

ACRES

Another possible limit corresponds to the point at which the ice cover is level with the hinge contact at the river bank. If the contact is poor and the water level rises above the contact point, the hinge may detach and the entire cover would become free floating. Thus, the limit to water level variation equals y_o and

$$\frac{\Delta y}{\ell} = \frac{y_o}{\ell} = \frac{1}{\sqrt{z^2 + 1}} \qquad (4.16)$$

The line of shoreline contact is shown on Figure 4.5.

It should be noted that the outer hinge link is considered to be critical for stability because of the steeper angles attained when both links are subjected to buoyant uplift forces.

If α_{max}, the friction angle for hinge joint stability, is less than α_a, the geometrical angular limit, frictional joint stability will govern hinge stability and the hinge will fail when α corresponding to $\Delta y/\ell$ and z reaches α_{max}. If α_{max} is greater than α_a and the hinge does not fail when α_a is attained, an increase in water level up to the elongation limit is possible if the hinge does not detach at the shoreline.

The shaded area in Figure 4.5 represents the geometrical configurations for rising water level stability of a two-link hinge for the range of limiting hinge angles α_{max} and bank slopes z possible. The dashed lines in Figure 4.5 represent geometric solutions to the stability equation which would only occur after a drop in water level from the elongation limit.

The maximum frictional contact angle α_{max} must be determined from field and laboratory experiments. With this angle, the limiting rising water level variation can be determined by following the appropriate slope curve z to α_{max}. If α_{max} is greater than the geometric angular limit α_a, the limiting value of $\Delta y/\ell$ is at the elongation limit.

For a falling water level, hinge stability is more restricted, as seen in Figure 4.6. As in the case of rising water level, the shaded area represents possible hinge configurations for various bank slopes and hinge link angles. The elongation limit is reached before the maximum angular limit is reached. Unless limited by joint friction, the elongation limit governs hinge stability. The dashed lines represent hinge configurations that could only be achieved by a rise in water level from the elongation limit, as illustrated in Figure 4.4. The maximum angle which can be formed occurs in this region of the graph when

$$\sin \alpha_a = \frac{1}{\sqrt{z^2 + 1}} \qquad (4.17)$$

At that point,

$$\frac{\Delta y}{\ell} = \sin \alpha_a - \frac{1}{\sqrt{z^2 + 1}} = 0 \qquad (4.18)$$

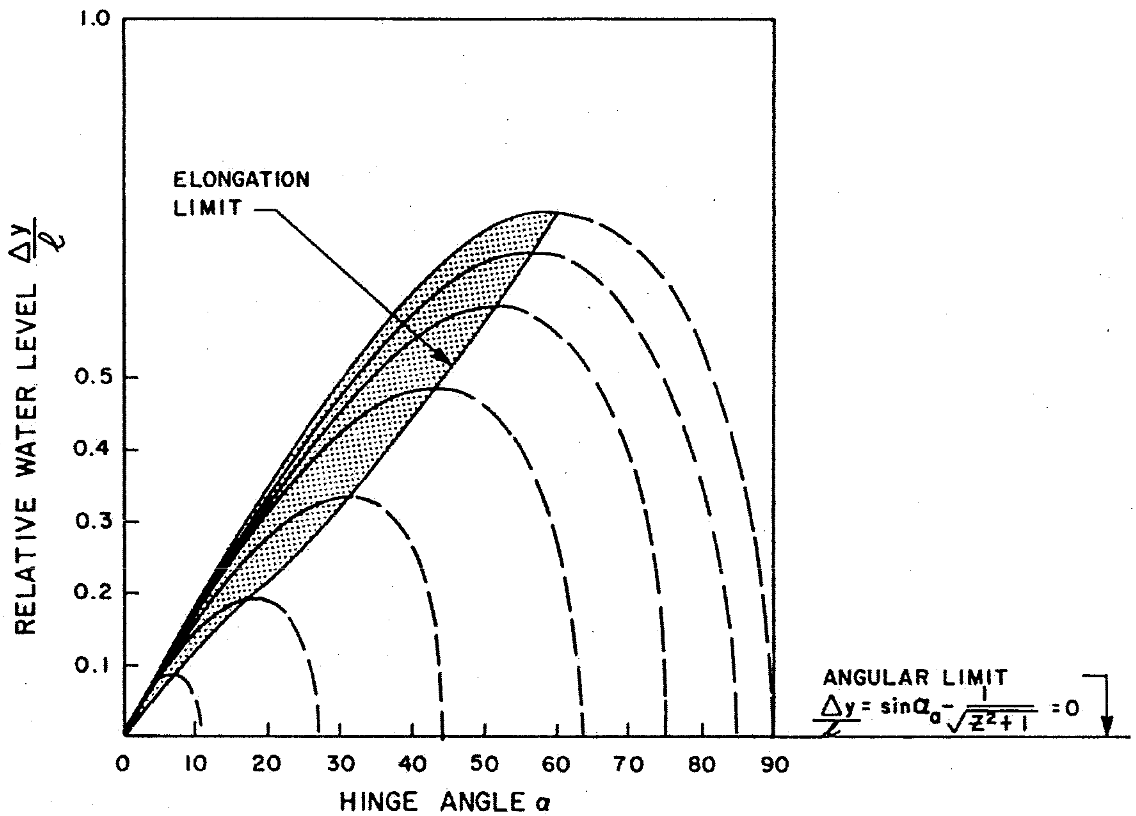


FIG. 4.6

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

TWO-LINK HINGE CONFIGURATIONS FOR FALLING WATER LEVEL

With the limiting hinge frictional angle and bank slope, the maximum falling water level variation can be determined from Figure 4.6.

The combined limits to rising and falling water level determine the total limit to water level variation under peaking operation. With α_{max}, a limiting hinge angle, determined from field and laboratory observations, and z, the bank slope measured in the field, the relative operating range for peaking operation can be established from the sum of the rising and falling water level limits. The absolute range in water level depends on the length of the hinge links resulting from ice conditions at cover formation and operating procedure immediately after cover formation. Hinge link lengths can be measured in the field or calculated for the rise or fall in water level following cover formation. As discussed previously, a rising water level will create longer hinge links and permit a larger peaking range after hinge formation.

The analysis of falling water level below the starting position may be irrelevant if the hinge joints have no tensile strength and the links separate before the elongation limit is reached. In this case, the lower limit to hinge stability is the starting position with one link lying on the channel bank and the second link horizontal.

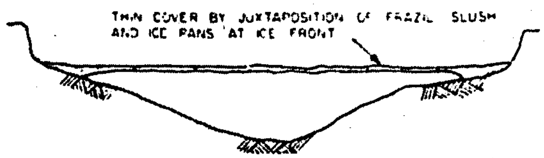
In the static case of a reservoir with no flow velocity, the failure of hinge links is less important. At high water levels, hinging may support the cover when subjected to wind stress preventing piling. At low levels, the hinge links lie on the reservoir banks. In some cases, the ice freezes to the banks and a new cover freezes above, (Sinotin and Sokolov 1968, Sokolov 1970), but for the most part the ice refloats and assumes its original position in the ice cover at higher water levels (LeVan and Fonseca 1977, LeVan 1977, Littlefield interview).

With a flow velocity, hinging contributes to cover stability by transferring forces on the ice cover to the river banks. If the hinges do not support the cover, the stresses on the cover accumulate in the downstream direction and ice jams can result. Donchenko (1978b) notes that water level variation with peaking greater than three to four times the ice cover thickness results in jamming. Pentland (1973) attributes ice jams on the South Saskatchewan River to variations in flow velocity and water level during peaking. Smelyakova (1970a) cites cases where failure of the cover occurred with varying water level at discharges that normally would not cause ice jams. Thus, the loss of ice cover support from the banks can cause ice jamming. Finally, Sinotin and Sokolov (1968) describe the jamming of the ice cover in the connecting canal at the Kiev pumped storage power station where flow velocity approaches 3 m/s, while the ice cover in the reservoir remains stable for the same water level variation.

The formulation of Pariset and Hausser (1961) can be used to determine cover stability between shoreline hinges. The width of the river between hinges is used for the channel width, B. An appropriate value of cohesion, τ, must be determined for contact at the hinge. If the hinges fail due to excessive water level variation, the accumulated forces on the ice cover must be resisted by crushing strength of the cover where it contacts the shoreline, structures or intact cover sections.

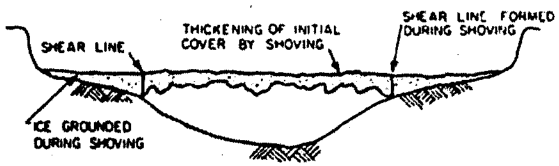
Another form of hinging which may be considered a boundary condition of this analysis is the phenomenon described by Schulte (Calgary Power interview). Vertical shear lines in the ice cover caused by shoving at formation provide shoreline contact. The upper limit to hinge stability is the water level which floats the cover from between the confines of the grounded ice shear walls. At that point, forces on the ice cover are no longer transferred to the river banks through the grounded ice. This type of hinging may be typical of covers which shove at formation while the arching hinge formation may be typical of covers which form in narrow channels where leading edge stability governs. The formation sequence of shear walls is shown in Figure 4.7.

For this initial investigation of hinge stability, the limiting range in water levels has been related to an arbitrary starting position with the offshore link horizontal and the nearshore link lying parallel to the river bank. The relationship of the onshore link position to a normal or maximum operating water level must be determined. The formation of the hinge after the initial cover failure and the frictional contact of hinge joints must be investigated. The type of hinge formed must be investigated in terms of flow regulation and the regime or river classification.



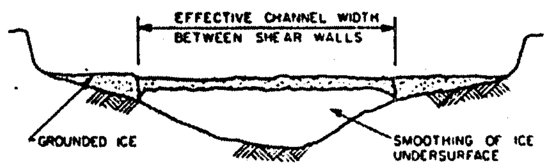
STAGE 1 DEVELOPMENT

LEADING EDGE STABILITY DETERMINES COVER THICKNESS INITIALLY AS ICE FRONT ADVANCES THROUGH SECTION



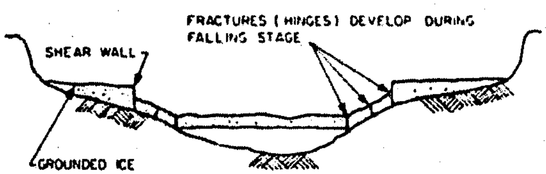
STAGE 2 DEVELOPMENT

COVER THICKNESS INCREASES SUBSEQUENT TO PASSAGE OF ICE FRONT AS STRESSES INCREASE IN COVER AT SECTION DUE TO BODY AND DRAG FORCES ACTING ON COVER UPSTREAM OF SECTION. COVER MAY ALSO THICKEN BY DEPOSITION IF ADDITIONAL ENERGY LOSS REQUIRED TO CREATE SUFFICIENT STAGE FOR STABLE ICE FRONT AT SOME UPSTREAM SECTION. SHEAR LINES FORM AS COVER TELESCOPES TO INTERNAL EQUILIBRIUM THICKNESS.



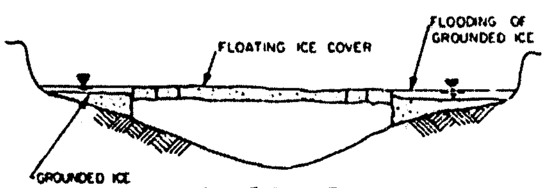
STAGE 3 DEVELOPMENT

FLOW UNDER COVER OR SLUSH/FRAZIL DEPOSITS SMOOTH UNDER SURFACE OF ICE COVER, WITH CONSEQUENT DROP IN STAGE FOR A GIVEN FLOW.



LOW FLOW STAGE

LARGE DECREASES IN FLOW RESULT IN SAGGING AND FRACTURING OF COVER. HINGE MECHANISM MAY FORM, DEPENDING ON HOW RIGIDLY COVER HAS FROZEN BEFORE LARGE DROP IN WATER LEVEL. COVER WILL FLOAT BACK UP WITH FLOW INCREASE, EITHER WITH HINGE MECHANISM OR AS SHOWN FOR HIGH FLOW STAGE, DEPENDING ON RIGIDITY OF COVER.



HIGH FLOW STAGE

LARGE INCREASES IN FLOW WILL FLOAT COVER. COVER WILL NOT FAIL PROVIDED ITS INITIAL INTERNAL EQUILIBRIUM THICKNESS WAS DETERMINED AT HIGH FLOW AND FLOW DOES NOT REACH STAGE WHICH FLOATS COVER OUT OF CONFINES OF GROUNDED ICE SHEAR WALLS.

FIG. 4.7

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

FORMATION OF SHEAR LINES AND HINGES

ACRES

5 - BREAKUP

The breakup process on rivers occurs rapidly and often with such violence that documentation and measurement are impossible. The process is further complicated by the number of meteorological and hydrodynamic parameters involved. An excellent description of breakup is provided by Deslauriers (1966).

The simultaneous weakening of the ice cover and sharp increase in river discharge resulting from warm weather result in a sudden release of a large volume of ice at high discharge. Early breakup when the ice is strong and has no cohesive strength creates the most severe jams.

5.1 - Thermal Processes at Breakup

Thermal processes differ from midwinter because conduction reverses and heat is supplied to cover from the atmosphere. The effect of thermal processes on ice cover strength is more important than the reduction in cover thickness. After the cover breaks up, the ice is assumed to have no cohesion and the stability of the ice is dominated by hydrodynamic conditions. For that reason, prediction of breakup is based only on statistical records of meteorological conditions (Barbridge and Lauder 1957) and ice jams are predicted on the basis of typical spring hydrodynamic conditions. No general attempt has been made to formulate the dominant meteorological and hydrodynamic parameters that initiate breakup.

Buzin and Kozitskiy (1975) presented a technique to predict the strength of a river cover during spring thaw. The relationship took the following form

$$\sigma = \sigma_o \left(1 - \sqrt{\frac{S_r}{S_o}}\right)^2 \tag{5.1}$$

where σ = ice cover bending strength after exposure to solar radiation

σ_o = ice cover strength at freezing/thawing temperature without exposure to solar radiation

S_r = solar radiation absorbed per unit ice volume

S_o = critical absorbed solar radiation for complete loss of strength.

The recommended value of σ_o was given as 5.5×10^4 N/m^2 (80 lb/in.^{2}). The critical value of solar heat absorption was given as 184 J/m^3 (4,944 Btu/ft^3). The absorbed solar radiation is given by

$$\frac{S_r}{S_t\ (1 - Al)} = e^{-ct_i^{\,n}} \tag{5.1a}$$

where S_t = total solar radiation

Al = ice albedo (average value 0.25)

t_i = ice thickness (m)

c,n = experimental coefficients.

The values of the coefficients were found to range between c = 0.541, n = 0.6 and c = 0.243, n = 0.6 for extreme ranges of ice properties.

With the prediction of ice strength from the above method, the cover stability criteria of Michel (1975) in Section 4.2.1 can be used to predict the breakup of the ice cover for forecast meteorological and hydrodynamic conditions. Without solar radiation data, the spring time ice bending strength recommended by Michel (1970) and forecast hydrodynamic conditions can be used to predict breakup from oscillatory bending.

Similarly, the formulation of Pariset and Hausser (1961) can be used to forecast breakup. As the cover shear strength deteriorates rapidly during the spring thaw, a critical point will be reached and the cover will shear away between the banks. The empirical Russian formulation reported in Tsang (1975) gives the thawing effect on ice shear strength, τ_i, as

$$\tau_i = 98.1 \left(\frac{\tau_o}{98.1} - \left(\frac{Th}{29} \right)^2 \right) \approx \tau_i = \tau_o - 0.1166(Th)^2 \qquad (5.2)$$

where τ_o = shear strength at 0°C without thawing, normally taken as 500 to 700 kN/m^2 (i.e. kPa)

Th = cumulative degree-days of thawing (°C days).

Prediction of spring breakup can be made with forecasted cover strength based on meteorological conditions and hydrology. On regulated rivers, discharge can be controlled during critical periods to prevent or prolong breakup, thereby avoiding serious ice jams.

5.2 - Mechanical Stability at Breakup

5.2.1 - Ice Jamming

After the cover has broken and the ice drive begins, prediction of jamming at obstacles to ice passage can be made using the formula of Pariset and Hausser with cohesion set to zero. This becomes

$$\frac{BV^2}{C^2H^2} = \left[\mu \frac{\rho'}{\rho} \left(1 - \frac{\rho'}{\rho}\right) \frac{t^2}{H^2} \right] \frac{\left(1 - \frac{\rho' t}{\rho H}\right)^3}{\left(1 + \frac{\rho' t}{\rho H}\right)} \qquad (5.3)$$

At maximum value of t/H equal to 1/3, $\rho'/\rho = 0.92$, and $\mu = 1.39$

$$\frac{BV^2}{C^2\ H^2} = 2.9 \times 10^{-3} \qquad (5.3a)$$

as indicated by the zero-cohesion curve on Figure 3.1.

The criterion developed by Michel (1965) for stability of ice jams at breakup is

$$\frac{H\ \sqrt[4]{B}}{\mu_M \sqrt{Q}} \geqslant 3.18 \qquad (5.4)$$

where H = flow depth (m)
B = channel width (m)
Q = discharge (m^3/s)
μ_M = coefficient typically equal to 0.26.

These criteria correspond for a Chezy roughness value C equal to 27 $m^{1/2}/s$. Stability of ice accumulations at breakup can be determined with either criterion.

As in the case of formation, extreme water levels and flow velocities should be considered for mechanical stability and time-averaged values for thermal effects.

6 - INTERACTION BETWEEN ICE AND STRUCTURES

Interaction between ice covers and structures has been widely reported for static and dynamic horizontal loads on structures and for uplift forces on structures for vertical ice cover movements. These analyses depend on the geometry of the structure, ice conditions and the nature of the ice/structure interaction. It is beyond the scope of this study to summarize all work on the many complex types of ice cover/structure interactions. Comprehensive summaries are presented in Michel (1978) and Korzhavin (1962).

Numerous studies of ice piling on sloping structures have been conducted as well. Croasdale et al (1978), Bruun and Johanesson (1971), Tsang (1974) and Allen (1970), to mention a few, have described this phenomenon. Readers are referred to an excellent summary of sea ice observations and analysis of ice piling by Kovacs and Sodhi (1980) for details.

In the above mentioned studies, the total force developed on the structure and stability of the entire structure have been formulated. The total force is related to the ultimate strength of the ice cover for the appropriate ice failure mechanism (bending, compression, shear) or to the kinetic energy of individual ice floes or ice fields. In general, forces imposed by a frozen, static ice sheet are limited by the ultimate strength of the ice while forces imposed by moving ice are determined from impulse-momentum or work-energy relationships (Starosolsky, 1970). For example, compressive force from thermal expansion is given as a product of ultimate stress in crushing multiplied by ice thickness times the length of structure-ice contact. The force of impact by an ice floe is determined by the kinetic energy of the floe, and the work done in deformation of the structure and ice floe as the floe is crushed and stopped.

Iyer (1978) presents a summary of existing ice codes in effect in North America, Scandinavia and the Soviet Union. Design practices for static and dynamic loads, and the various methods to account for the effect of ice and structure geometry to arrive at the appropriate ice pressures are compared.

Typical static design loads in North American practice are expressed as a uniform, distributed load at the water line; 15,000 - 22,500 N/m for rigid structures and 7,500 - 15,000 N/m for flexible members where creep of the ice and deflection of the structure relieve the load (Acres, 1970).

Dynamic loads are more difficult to determine because of the many complex variables involved. Normally, a crushing strength of 275 Pa is used for dynamic ice pressure and the type of failure mechanism and contact area are considered in the application of work-energy principles to determine the total load.

These methods are considered to be too conservative but cannot be improved until the actual mechanisms are known. Structural elements which cannot be designed practically by this method to resist dynamic loads are operated to avoid ice impact entirely. Thus, sluice gates are operated fully open or fully closed.

These general design practices are found to be inadequate when applied to specific or unique cases, as illustrated by the failure of stoplogs in the Montreal Ice Control Structure (Acres, 1970). The stoplogs were designed to withstand a load of 7,500 N/m. However, impact of a large ice floe punctured the steel stoplog and then caused it to buckle. Clearly, the present design practices must be expanded to include a greater variety of loading and failure conditions.

Whereas the design practice for determining total loads on structures is too general and, therefore, inadequate, there is nothing published to formulate or describe explicitly, the effect of localized ice action on structure surfaces, protective rip-rap or natural riverbanks.

6.1 - Damage Related to Water Level Variation

The theoretical determination of vertical forces on structures by solid ice covers is difficult due to the time dependency of ice properties (strength, elastic/plastic behavior, etc). The result is a lack of knowledge on the complex behavior of a floating ice sheet (Michel, 1978). Many assumptions have been made to develop formulation for two analyses--deformation of an infinite plate on an elastic foundation and brittle failure, or plastic failure at the ultimate strength of the ice sheet. The formulation for failure of an infinite plate on an elastic foundation, developed for ice forces on a straight vertical wall or an isolated pile, are reported in Michel (1978). The maximum forces and moments on the structure are equal to the "loading" which causes the failure of the ice cover. The initial failure is the critical loading on the structure. Subsequent failures occur at reduced loads at the weakened initial failure location and are more difficult to analyze.

Shaitan (1967) observed that the central portion of a solid cover will follow the smallest changes in water level. The portion of the ice cover attached to the shore will deform and acts as a cantilever beam. The analysis of Shaitan calculates the reactions and moments of the beam transmitted to the shoreline. The reactions are expressed in terms of design values of ice stresses which are in turn related to yield stresses of the ice, allowing for time reduction of stresses.

No indication is given, however, of the method of determination of the appropriate values for ice viscosity and elastic modulus. Shaitan does note that the initial failure is critical and that subsequent failures occur when the ice bears on the initial failure crack. This conclusion reached by both Shaitan and Michel supports the fundamental assumption of successive cover failures for the formation of hinge links in Section 4.2.2 of this report. Considering the difficulty in determining appropriate ice properties for basic analysis (elastic modulus, ice strength) and the effect of shoreline contact, the simplified analyses presented in Michel (1970) seem most appropriate to estimate the moment and vertical reaction of the ice cover on the shore.

Michel (1978) notes that the vertical reaction of the ice cover may be limited by the strength of adhesion of the ice to the structure surface. There is little data available on this ice strength which, in the limiting case, may be as high as the shear strength of the ice itself. This approach is satisfactory for concrete or steel structure facings but is inadequate for rip-rap or natural river banks where the degree of freezing of the banks determines the shoreline resistance (Rossinskij and Nisar-Mahamedova, 1977). There is no method to determine the resistance of natural or placed material to ice action.

Korzhavin (1962) notes that considerable damage to river and marine protection can be caused by ice action. Individual stones can be worked from stone masonry by freeze/thaw cycles, rubble armour lifted from protective layers, and piles lifted and carried away by floating ice. The damage caused by impact, rocking and lifting of armour units by ice can be much greater than the hydrodynamic action on rubble-mound structures. More damage is caused by spring ice movements (impact) than by midwinter ice action (thermal expansion, thrust, lifting). It may be more appropriate to alter ice conditions than to design a structure to resist the ice forces. Ice dusting has been used to reduce albedo of the ice surface. The result is a thinner, weaker cover. Mechanical means such as blasting, ice cutting and ice breaking with ships have been used as well. Sommerville and Barnes (1966) describe the use of an air bubber to prevent further damage, attributed to rising and falling water level, to the 15-cm thick concrete reservoir lining of a Winnipeg reservoir. Pekhovich et al (1970) recommend coating structures with hydrophobic materials to prevent ice buildup. In the Soviet Union, it is common practice to coat sea walls and other marine structures with tar (Tsang, 1975). This has a number of favourable characteristics. The ice cannot freeze to the tar eliminating vertical forces and moments, heat absorption by the tar melts the ice away from the structure reducing horizontal forces due to thermal expansion, and the layer of tar reduces the freeze-thaw action by warming and partially sealing the face of masonry structures.

To prevent ice buildup on isolated structures or mechanical equipment such as stoplog guides, spillway gates or trash-racks, electrical heating can be used. Heating must begin before ice forms on the structure. It is practically impossible to get rid of the ice once there is a strong deposit (Michel, 1971). The power required to prevent freezing is given by

$$P = 2.34 \frac{(V_b d_b)^{0.52}}{S_b} \tag{6.1}$$

where P = power per unit area (W/m^2)

V_b = water velocity past bars (m/s)

d_b = bar diameter (m)

S_b = spacing between bars (m)

6.2 - Damage at Shorelines

Numerous authors discuss the tremendous destructive action on shorelines due to impact and abrasion of moving ice. The great mass of moving ice accumulations results in damage even at low velocities (Tsang, 1975).

Russell, quoted in Bolsenga (1968), relates the observations of ice damage made in early Arctic geological expeditions. In shallow areas, the river bed and bank topography is dictated by ice action which thoroughly removes all vegetation and creates tremendous furrows in banks and shoals. Bed material is transported over great distances, huge boulders pushed up on the riverbanks, and trees scarred above high water level. No material except bedrock can resist the main thrust of ice. Numerous other authors reported in Bolsenga offer similar descriptions of great damage and dominance of ice on the river morphology. Korzhavin (1962) presents many examples of shoreline and structure damage as well. Details include cutting of piles by ice abrasion and scour of concrete structures to depths up to 8 cm.

A distinction should be made between the effect of ice on river morphology and isolated events of ice damage. Newbury (1968) describes the indirect and direct effects on river morphology.

Ice affects river morphology indirectly by changing the hydraulic regime to accommodate ice cover stability requirements. A two-layer flow distribution is formed under the ice cover. If a thin ice cover forms, the change in shear stress at the river bed is small. If the leading edge Froude number controls winter flow depth, the flow velocity may be reduced by staging below summer values and sediment deposition will occur. Conversely, if a thick cover forms by shoving or ice deposition, the velocity under the cover will rise and channel bed erosion may occur.

The direct effects of ice are more obvious. Bed or riverbank material is lifted or scoured by moving ice. Bed material carried by moving ice is deposited when the ice melts. The movement of bed material depends on local ice movements, not on the bed material. On the Nelson River, movement of boulders ranging in size from 0.15 to 4.93 tonnes was monitored. The movement of these boulders ranged from 1 m to more than 180 m, taking place in shallow bed areas and riverbanks, particularly on islands and in rapids sections where major shoving and ice jams took place during formation and breakup.

Evidence of ice action on riverbanks is indicated by the vegetation trim-line at the maximum level of ice action. Below this level, a shallow, concave-up groove along the riverbank is formed by the ice.

Another common feature of ice-covered rivers is side channels, or terraces, formed by flow bypassing ice jams or staging due to downstream jams.

Smith (1979) adds that ice processes dominate the river morphology by enlarging the river cross section. Ice scour from the ice drive at breakup and ice jams effectively increase channel cross-sectional area at bankfull flow by as much as three times. Considering increased velocity, the cross sections can carry an average of 4.7 times more discharge for the 24 rivers studied in Alberta than for comparable rivers in warmer climates. Thus, the recurrence interval for bankfull flow is increased from 1.6 to 16.7 years.

For an established hydrology, ice processes dominate river morphology. Subsequent ice damage occurs when extreme combinations of hydrologic and meteorologic conditions cause correspondingly severe ice jams and movement at break-up or formation. If the natural hydrology is altered by flow diversion, flow regulation or river basin development, new "damage" may occur as the river morphology adapts to the new hydrology.

The domination of ice over riverbank characteristics can be seen at Churchill Falls. With the hydroelectric power development, the regulated winter discharge of 1,416 m^3/s is much higher than the natural winter flow of 280 - 425 m^3/s. As a result of the increased winter discharge, much thicker ice covers form at higher water levels. The scouring action of the ice can be seen as high as 5 m above the normal open-water elevations. Trees on riverbanks and low islands that were previously above the ice action have been flattened by the ice.

In the analysis of ice piling, the kinetic energy of ice floes is resisted by the frictional force on the sloping banks (Croasdale et al, 1978). The total force of the pileup is expressed as a pressure, analogous to the pressure of a granular soil. Michel notes that the initial impact of the ice is generally a greater force than the pressure of the ice pile. The most significant effect of the ice is to increase the effective cross-sectional area of the structure in front of the advancing ice pack.

Agalakov et al (1974) observed that the frequency and limits to ice pileup on smooth and rough slopes are essentially the same. The forces on rough slopes, however, are greater. If the slope roughness is defined in terms of rock armour diameter to ice thickness, then the force of the ice piling on the slope on a smooth slope ($D_R/t_i = 0$) is 39 kN/m, and on rougher slopes, 49 kN/m ($D_R/t_i = 1$) and 59 kN/m ($D_R/t_i = 2$).

The frictional resistance of the slope is assumed on the basis of limited relevant field data for ice/rubble friction coefficient. This is used to determine the limit of ice rideup on the sloping structure face. Damage to the face of the structure is not considered but is generally assumed to be minor. Michel (1978) states that rock, sandbag or concrete block bank protection have poor resistance to scour when exposed to ice action. Less damage to shorelines occurs than would be expected, however, because naturally occurring shorefast ice and ice pileups absorb energy and protect slopes from scour (Tsang, 1975 and Bruun and Johanesson, 1971).

Foulds (1974) describes ice piling on the Great Lakes near several water intakes. At Douglas Point on Lake Huron, ice mounds reached 13.7 m above water level. On Lake Erie near Buffalo, ice piled to a height of 9.1 m above water level in 8 m of water.

Kovacs (1980) reviewed reports of ice piling relevant to arctic or antarctic beaches. Numerous observations of ice piling in excess of 10 m high were made. Kovacs distinguishes between ice rideup of sheets of ice and pileups of broken ice. Pileup seldom occurs more than 10 m inland from the sea but rideup frequently extends 50 m or more inland, regardless of ice thickness.

At breakup as well, ice jams wash out between vertical shear lines in the ice cover leaving heavier ice accumulations on the riverbanks. This ice, grounded or frozen to the riverbanks, protects the banks from scouring action of the moving ice at breakup. Thus, shorefast ice shculd be promoted, There are many case studies that attest to the tremendous forces and damage caused by ice (Bolsenga 1968, Kovacs, 1980). The reports, however, are site-specific, qualitative accounts which cannot be used to formulate criteria for ice action on structures because of the number of parameters involved and the lack of detailed measurements. Because design criteria are inadequate and the ability of structures to withstand and direct ice forces requires prohibitively massive structures, the design approach taken is to eliminate ice action all together. Protective coatings, mechanical action and air bubblers are used to eliminate forces from solid covers. Large hydraulic works are designed to reduce ice impact by creating low-velocity areas for ice retention or by using diversion canals, ice booms and ice sluices to direct the ice away from vulnerable structures.

7 - CALIBRATION OF SOLID ICE COVER STABILITY PROCESSES

Michel (1971) states that the most severe jams occur when the ice cover breaks up prematurely. Ice jams formed at an early breakup are exceptionally strong because the ice cover has not weakened and a large increase in water level is required to create sufficient pressure to wash out the jam. This was a major factor in the 1968 ice jams on the St. John River (Atkinson, 1973).

The violence and rapid progression of spring breakup are responsible for the damage and for the lack of quantitative data. Data have been collected for calculation of leading edge stability and internal stability for spring ice jams but data are not available at the present on ice cover properties and hydraulic conditions during the sequence of events leading up to the breakup of the solid cover. It is not possible, therefore, to calibrate the analysis of the initial failure of the ice cover at the shoreline or the stability of hinge formation on the basis of the qualitative descriptions of solid ice cover breakup given in the few references located.

7.1 - Observations of Solid Ice Cover Breakup and Hinging

The following observations demonstrate the effect of fluctuations in flow on the stability of solid ice covers and support the assumptions made in hinging analysis.

The analysis of Shaitan (1967), described in section 5.1, gives an expression for the critical water level variation causing failure of a solid ice cover frozen to the river banks. The influence of temperature and rate of loading on ice strengths and stress relaxation is not clearly defined.

Shaitan states that the initial failure is most critical for structure stability. Subsequent failures occur when the ice cover bears on the initial failure plane. The formation of several hinge links can be explained in this way.

There are several reports in the literature that attribute premature breakup of a solid ice cover to discharge variation accompanied by water level fluctuation (Townsend 1971, Gotlib and Razorenov 1970). Smelyakova (1970a) notes more specifically that ice jams occur at velocities lower than normally critical for ice jamming when accompanied by fluctuating water level. Many cases are reported where increases in discharge approaching 100 percent of the normal flow rate cause disruption of the cover. The zone of influence for peaking extends downstream of the hydroelectric power plants over 60 km. No mention of ice problems in forebay reservoirs or upstream river channels is made. The cases reported are extreme in terms of discharge variation and the disruption of the ice cover. Insufficient hydraulic data are presented to permit analysis of these events or calibration of the conceptual model.

Donchenko (1978a, 1978b) states that a water level variation greater than three or four times the ice thickness will break the bond between the river banks and the ice cover, and result in ice jams. The cover initially fails in longshore cracks. As the ice cover lifts with increasing stage, transverse cracks form and large pans are broken from the leading edge which then raft and pack. The solution to this instability is to restrict peaking until a thick cover forms in the area influenced by peaking.

The criterion given for critical rise in water level for failure of the ice cover contact with the shoreline is

$$\Delta y_i = C \frac{\sigma_b}{\chi\psi} \sqrt{\frac{t_i}{E}} \tag{7.1}$$

where σ_b = bending strength of the ice (Pa)

t_i = ice thickness (m)

E = elastic modulus of ice (Pa),

This criterion has the same form as the relationships presented by Michel (1970). The coefficient, C, has a value of 34.6 for "partial connection" to the shoreline and 17.3 for "so-called hinged attachement". The functions of relaxtion, χ and ψ, are determined by graphical means, which are not presented in Donchenko's report but are referenced to the original Russian report by Kachanov (1947).

The definitions of partial attachment and hinged attachment are not sufficiently clear. Undoubtedly, the hinged attachment does not correspond to hinging as defined in this report. It may represent a pinned joint in the beam analysis and partial attachment may represent a beam with a fixed end.

Flow variation for peaking operation in early winter causes breakup and ice jams on the South Saskatchewan River (Pentland, 1973). The solution to this problem is a steady, high discharge until a strong, solid cover forms to provide maximum conveyance. Minor flow variation is permitted for the rest of the winter. No clearly established criteria have been developed.

In 1969, a variation of 28 m^3/s from the regulated discharge of 283 m^3/s caused breakup of a thin cover shortly after freeze-up. However, a fluctuation of 113 m^3/s in December 1972 caused only minor jamming. Further studies of the relationship between ice cover breakup and flow variation were recommended.

On the Winnipeg River, a series of three peaking tests were performed to evaluate the response of the ice cover to water level and flow fluctuations (Kartha 1977). The daily peaking ranges tested were 710 to 1,130 m^3/s, 565 to 1,360 m^3/s and 480 to 1,615 m^3/s. Previous peaking operation with a range of 710 m^3/s did not cause ice cover instability so no observations of ice cover behaviour were made.

At the smallest peaking range tested (710 to 1,130 m^3/s), the ice cover rose and fell with the water level. No cracks or openings in the cover were observed. The maximum level variation observed was 0.61 m with an average variation of 0.20 m.

At the middle peaking range (565 to 1,360 m^3/s), hinge cracks 0.15 m wide formed near the shoreline but no breakup of the cover was observed. No flow over the ice was reported. The maximum ice cover level variation was 0.86 m.

For the final test (480 to 1,615 m^3/s), the hinge cracks opened up to a width of 0.76 m. The ice cover broke up at critical cross sections. Major cover disruption occurred where the maximum level variation was 1.19 m. Thus, it was concluded that the critical peaking maximum discharge and flow variation on the Winnipeg River falls between the second and third test values.

The interviews with various hydroelectric power utilities failied to provide quantitative data for calibration of the conceptual model. The observations made by the various utilities confirm the disruption of the ice cover and formation of shoreline hinging at peaking stations. The general solution to cover breakup is a strict curtailment of peaking operation. The criteria for winter plant operation are established on the basis of experience. These criteria may be overly conservative and, due to their site specific nature, are of little help in eliminating ice cover problems at other existing or proposed developments.

A number of promising field sites for data collection were identified and are listed in Appendix B. Bearspaw Dam on the Box River upstream of Calgary specifically mentions problems with fragmentation and sagging of solid covers (Mr. P. Dole). Shulte describes the formation of vertical shear walls on Alberta rivers between which the solid cover rises and falls with varying water level. He recommended that covers should be formed at highest discharge anticipated during the winter season and that the cover should be "exercised" to maintain a stable response to fluctuating water levels.

Very few observations of ice problems in reservoirs were reported. Sokolov (1970) and Sommerville and Burns (1968) report some damage to concrete reservoir linings and structures as a result of water level variation. These localized problems can be solved by weakening the contact with the ice cover with hydrophobic coatings, air bubblers, heat application or mechanical action. Similarily, no serious ice problems in reservoirs were reported at the Niagara pumped storage plants (Yorke and Cummmons interviews), the Hollingsworth reservoir (Littlefield interview) or the Northfield Mountain and Blenheim-Gilboa pumped storage reservoirs.(LeVan 1975). Ice problems with fluctuating water levels would appear to be a problem only in the presence of a flow velocity.

Determination of ice cover strength is necessary to apply the criteria developed in this study. Michel (1978) states that ice can experience a five-fold decrease in strength in a single day at breakup. The relationships developed for compressive strength and ice temperature under laboratory conditions cannot be applied to field situations until a remote method of average ice temperature determination is available. There are no empirical relationships for bending or compressive strength and degree-days of thawing at present although such a relationship exists for shear strength (Tsang, 1975).

7.2 - Calibration Data for Hinging

Field data on ice cover hinging are taken from the report by Fonseca (1979) and a subsequent interview. Field observations of ice cover levels were taken at two locations on the Gatineau River downstream from the Paugan Falls generating station.

At the location identified as KM 37.4, ice cover elevations were taken at three points across the width of the river on January 29 and January 30, 1979. These elevations are shown in Figures 6.1 and 6.2 with water level readings taken at KM 38. Partial river cross sections are shown as well.

The position of the ice cover on the shoreline at KM 37.4 on both days was the same at low morning water level. The change in ice cover levels offshore and onshore on each day reflected the different water level ranges.

At the location identified as KM 42.7, near the Farrelton Bridge, horizontal and vertical measurements were taken at eight locations on the ice cover (see Figure 6.3). In this case, water level differences, but not absolute water levels, can be compared with ice cover elevations.

From these figures, the following conclusions can be drawn.

- As the water level rises, the central section of the ice cover follows with an equivalent change in elevation.

- Successive hinge links float free from the banks after the ice cover offshore rises and the hinge link becomes partially submerged.

These initial data and the photographs in Fonseca's report identify the hinging process. Some of the assumptions in the hinging analysis such as submersion of the links and rotation of links with rising water level have been verified. There are some questions that cannot be answered about these field data such as the apparent shortening of the hinge link between points 5 and 6 in Figure 6.3. There is a need for more photographs and descriptions of the the hinge link and joint movements before complete calibration of the analysis of hinging is possible. More field data are required with wider water level variations to test the extreme limits to hinge stability.

Observations made by Schulte of ice cover fluctuation between vertical shear line boundaries illustrate the complexity and range of hinging action. Any field program is site specific and, therefore, is limited by the natural range of parameters at that location. Thus, a laboratory program to test the complete range of all parameters is required as well as selective field programs to relate laboratory results to prototype field conditions.

ICE COVER LEVELS AT km 37.4 (GATINEAU RIVER) JANUARY 29, 1979

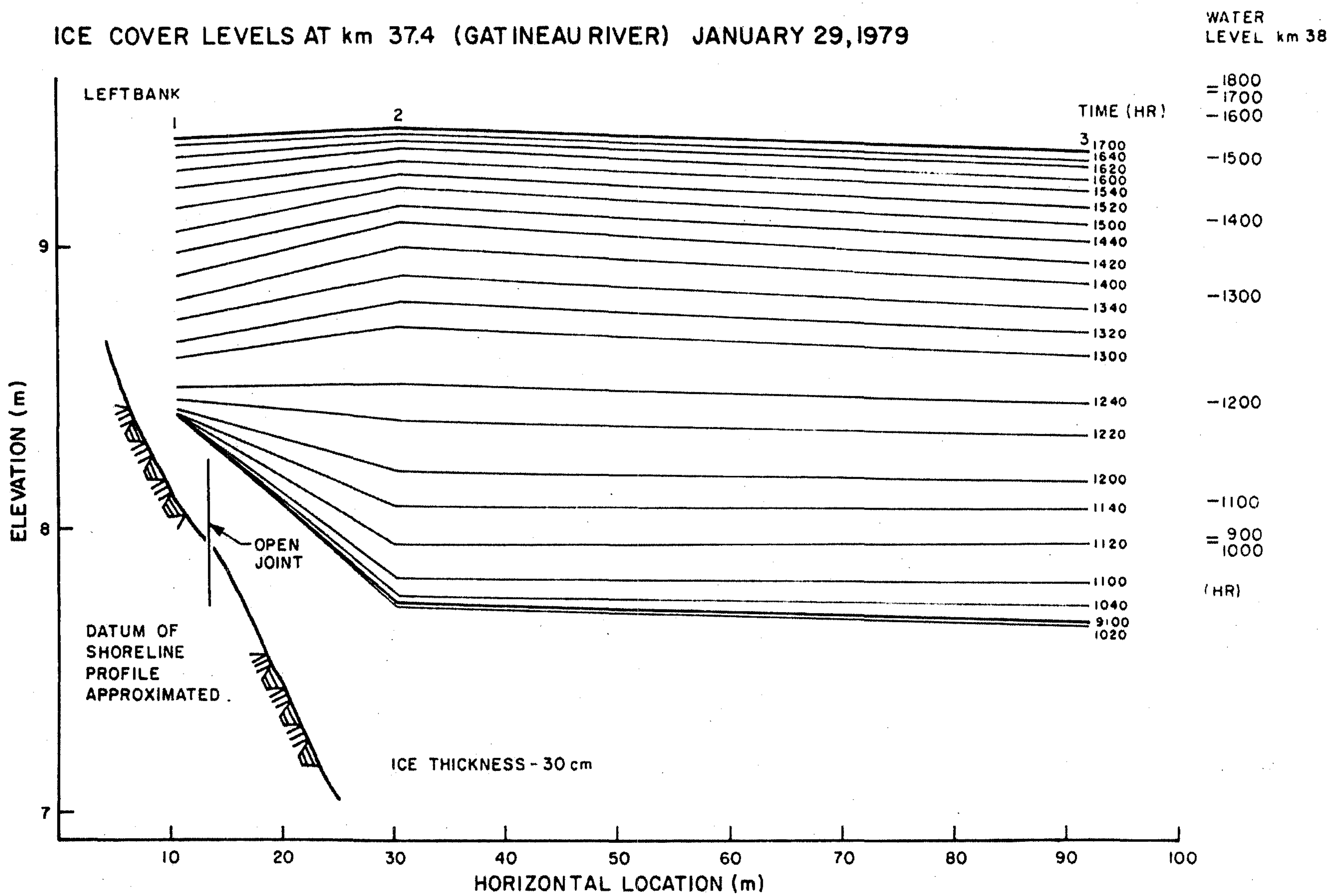


FIG. 7.1

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

FIELD MEASUREMENTS OF COVER MOVEMENT

ACRES

ICE COVER LEVELS AT km 37.4 (GATINEAU RIVER) JANUARY 30, 1979

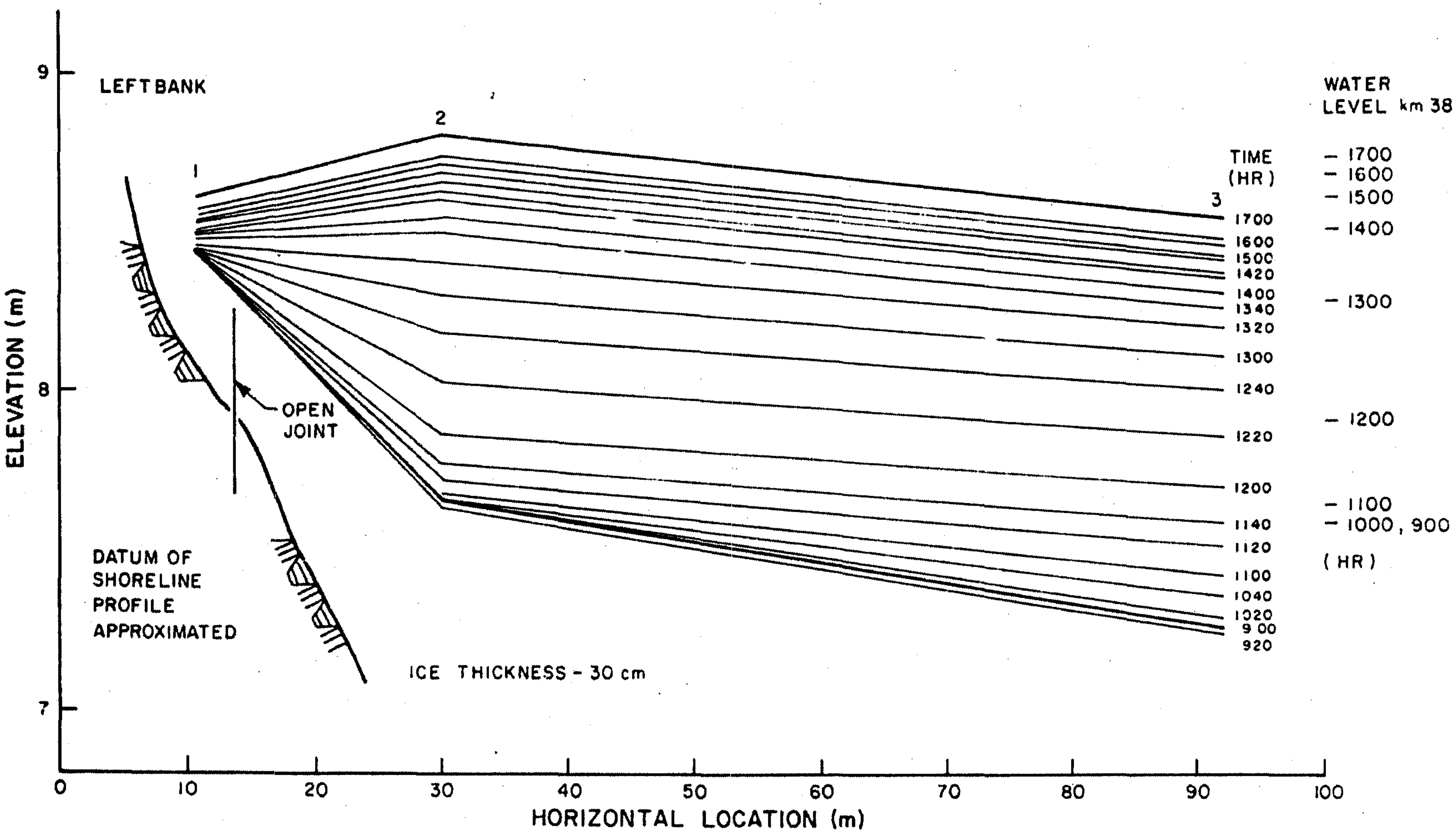


FIG 7.2

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

FIELD MEASUREMENTS OF COVER MOVEMENT

ACRES

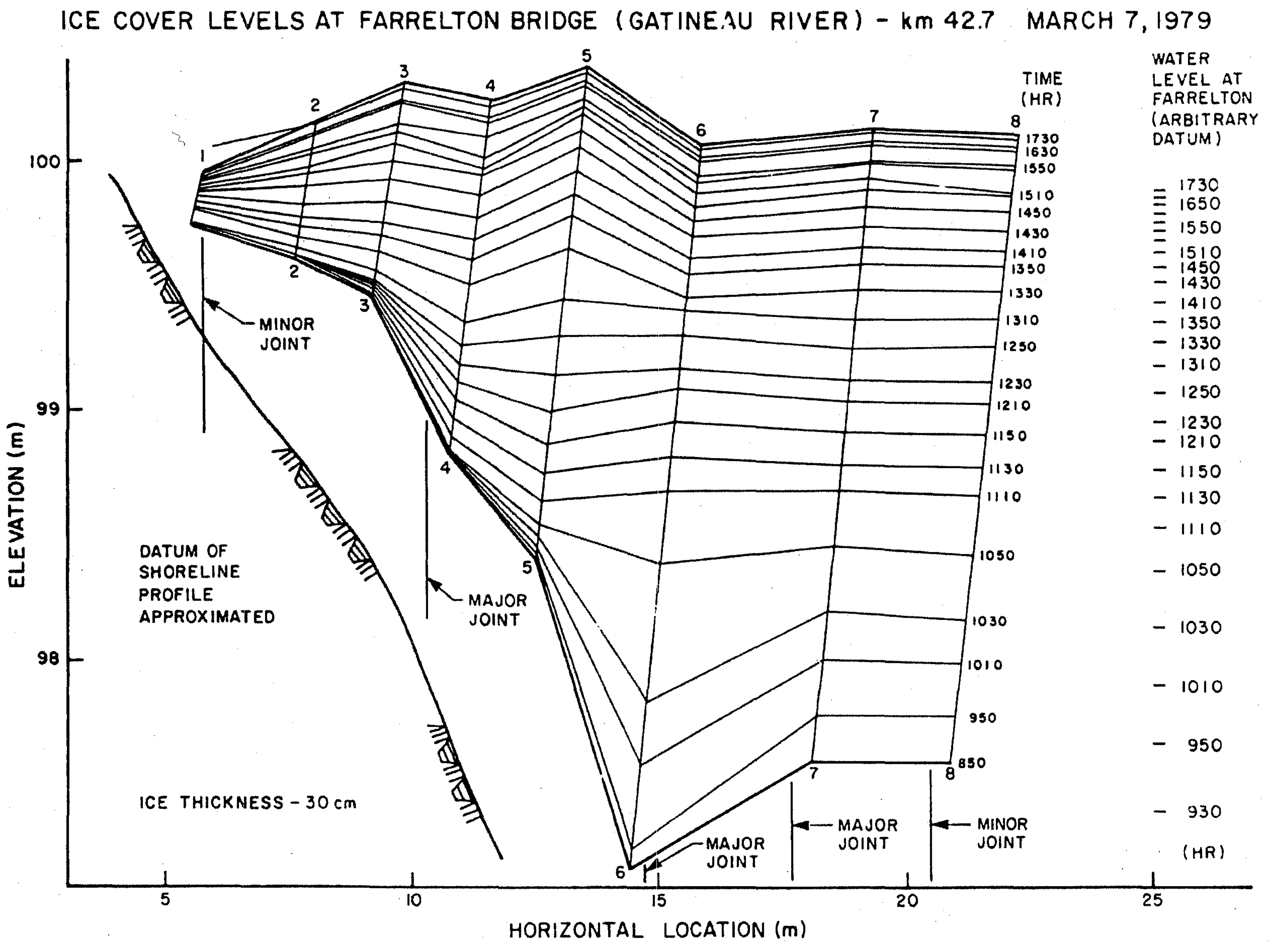


FIG 73

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

FIELD MEASUREMENTS OF COVER MOVEMENT

ACRES

8 - CONCLUSIONS AND RECOMMENDATIONS

8.1 - Conclusions

Ice processes may be categorized into those occurring during initial formation of a solid cover, the continuing competition of forces in maintaining a dynamic equilibrium of an existing ice regime with ambient hydrological and meteorological conditions, and finally those which govern the breakup and melting of the cover. Characteristic processes pertain to each channel reach, lake or reservoir depending on the geometry, climate and the flow management steps which are imposed.

An ice regime is affected by flow velocity and water level fluctuations during each stage of cover growth, change or decay. Such effects must be carefully controlled in the operation of water resource developments in order to avoid untoward results which could impair the output or benefits of the project. Similarly, during the design of future developments, consideration of ice management techniques is an important part of maximizing the utility of winter operations.

Based on research performed by many investigators, most of the ice processes normally encountered can be described by mathematical formulations and recognized limits. Some are the result of detailed theoretical analyses, while others are empirical rules derived initially from field observation. Some are more thoroughly calibrated and verified than others. The foregoing sections of this report have outlined those relationships which are most valuable for definition of ice processes related to typical Canadian utility winter operations. Taken together, the various formulae and limits form a conceptual model of ice cover behavior which can serve as a tool to be applied to both operational and design problems.

Much research has gone into describing the sequence of initial ice cover formation, growth and consolidation. Both dynamic channel processes, and lake effects are well covered in available literature. The mathematical formulations currently in use as design techniques adequately predict prototype behavior in most cases. The effects of varying flow velocity and/or water level can be assessed by application of proven relationships for specific circumstances. Appropriate formulae and guidelines are presented in Sections 3, 4 and 5 of this report.

Midwinter processes during which an ice regime is gradually modified by ambient climatic or hydraulic events are also reasonably well described insofar as the changes in extent of surface coverage and thickness of a cover are concerned. Empirical observations have also indicated that a solid cover is much less sensitive to changes in flow velocity than a cover during initial formation. Less clear, however, is the effect of rapid water level changes on an already-established ice cover. Those relationships pertaining to midwinter ice processes are covered in Section 4 of this report.

Because of their dynamic nature, breakup processes are less well defined than other ice regime stages. Both statistical and empirical approaches have been outlined in Section 5 of this report.

Solid ice covers may be susceptable to premature breakup when subjected to varying water levels which destroy the ice cover contact with the channel banks. In light of the flooding, power loss and structural damage caused by sudden and severe ice jams typical of this type of breakup, it is both surprising and unfortunate that so little quantitative or qualitative data exist to guide the analysis of solid ice cover breakup or ice hinge formation and stability. The preliminary analysis presented herein (see Section 4.2.2 of this report) is formulated from a basic hinging concept to develop simple criteria which can be applied with only basic data requirements. This analysis is appropriate for the present rudimentary understanding of the ice mechanics involved, and the present level of ability to determine field values of the parameters needed. Comprehensive laboratory and field studies are required to confirm the assumptions made in this analysis and guide its subsequent calibration.

The annual damages to shoreline structures, water resource developments and the like are a reluctantly accepted part of life in northern latitudes.

Although cases of severe damage to natural river banks and man-made structures are reported in the literature, techniques to forecast ice loadings and assess the probable extent of damages are unsatisfactory at present. Descriptions of typical ice effects on structures and shoreline are contained in Section 6 of this report.

Due to the complexity of ice/structure interaction, it is likely that research in this area of ice engineering will continue to be very problem-oriented in scope, addressing only ad hoc technological requirements. This has been the case for bridge piers and for artificial islands subject to drifting Arctic ice pack. In recent years, increases in hydroelectric peaking operations have created a need to study the problem of solid ice cover stability in river peaking and pumped storage hydroelectric plants. Data regarding ice/structure interaction has, to date, been either site-specific or only qualitative in nature. Neither approach is of assistance to the design of more satisfactory shoreline appurtenances. In the course of future studies, however, damage to structures, channel linings and reservoir slopes caused by ice cover movement must also be addressed.

8.2 - Recommendations

On the basis of this literature review and analytical study, the following recommendations are proposed for further investigation of solid ice cover stability.

A critical water level variation derived from theoretical considerations can be calculated from geometrical and mechanical stability analysis of hinge link joints to maintain ice cover/ channel bank contact.

Several simplifying assumptions were made in the derivation and many features of the ice cover contact at the shoreline could not be expressed mathematically because of the complexity of the phenomenon and lack of field observations. A comprehensive laboratory and field program is required to calibrate the analytical criteria developed. The laboratory program should preceed the field testing. A full range of parameters can be tested under controlled conditions in order that extreme values of parameters and the various cover failure mechanisms can be evaluated. Of particular interest is the evolution of hinge links from initial ice cover failure to the final hinge configuration, and the influence of ice cover formation processes and channel geometry on hinge characteristics. In the course of the laboratory and field tests, the application of various steady-flow formulae for fragmented ice cover stability to variable water level and flow velocity conditions should be investigated. Assumptions made in the analysis should be verified or new assumptions made based on test observations and consideration of analytical limitations.

The criteria calibrated in laboratory tests should then be verified in field tests to check possible scale effects in model testing. Field tests, however, are limited by cost, schedule and their site-specific nature, as well as the unacceptable danger of extreme test ranges. Hence, the need for laboratory work.

Application of the Pariset and Hausser stability equation for fragmented ice cover stability to stability of a solid cover between hinge links should be investigated. The appropriate values of frictional and cohesive contact across the hinge links for transfer of cover stresses to the riverbanks must be determined.

The initial ice cover failure is critical for ice loading on structures. Ice contact with concrete structures can be determined from ice adhesion which is limited to ice shear strength. The ice contact with armoured and natural slopes is more complex. A large number of tests is required under static and flow conditions to account for variations in parameters such as slope angle, slope covering and underlayer material, and meteorological conditions affecting ice cover properties and slope freezing.

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APPENDIX A

ANALYSIS OF HINGE STABILITY WITH WATER LEVEL VARIATION

A.1 - STABILITY OF A SINGLE-LINK

Detailed observations of hinge joint behaviour are not available so the mechanisms of joint contact and failure are not known. The fractured ends of hinge links may assume some combination of an interlocking shear pin connection and an ice-over-ice frictional contact. Stability of a single-link is based on the resistance of hinge joints to buoyant forces on the submerged hinge links.

The force diagram shown in Figure 4.2 shows that the system is statically indeterminant with pinned joint connections and cannot be solved unless a simplified assumption of frictional resistance is assumed. However, a solution still cannot be obtained if the frictional coefficient at each joint is the same. By assuming that the reaction at the higher joint has a higher frictional resistance than the lower joint, the force diagram becomes as shown in Figure A-1. For stability, the summations of forces and moments are zero and the following equality holds

$$\tan \alpha_{max} = \frac{\mu_{h1}}{2} \left(1 - \frac{\mu_{h2}}{\mu_{h1}}\right) \tag{A1.1}$$

where α_{max} = the limiting angle of the hinge link from horizontal

μ_{h1} = the friction coefficient at the higher hinge joint

μ_{h2} = the friction coefficient at the lower hinge joint.

One limiting case is when μ_{h2} equals μ_{h1} and α_{max} equals zero. The other limit is when μ_{h2} equals zero and α_{max} equals $\tan^{-1}(\mu_{h1}/2)$. The stability of the statically indeterminant link can only be determined by laboratory and field observations of the maximum stable link angle. As it is difficult to separate the individual joints, the limiting case when μ_{h2} equals zero and α_{max} equals $\tan^{-1}(\mu_{h1}/2)$ was selected for further consideration.

A.2 - STABILITY OF A TWO-LINK HINGE

The hinge mechanism will be stable if the frictional resistance of the joints is not exceeded through the water level cycle. As shown for the case of a single-link hinge, the joint will be stable if the critical link angle is not exceeded. The geometrical relationship between water level change, hinge link length and bank slope was examined starting at the position shown in Figure A-2. The hinge link lengths were assumed to be equal, a reasonable assumption if all the links were formed within a short time period. The three cases shown in Figure A-2 were identified as

- a falling water level

- a rising water level up to the level of the river bank contact

- a rising water level above the level of the river bank contact.

For the third case, shown in Figure A-3, the following relationships hold

$$\ell + \frac{z}{\sqrt{z^2 + 1}} \ell = y_1 + \cos\alpha\ \ell = \text{constant} \tag{A2.1}$$

$$y_i = \sqrt{\ell^2 - x_1^{\ 2}} \tag{A2.2}$$

$$x_1 = x_3 - x_2 \tag{A2.3}$$

$$x_2 = \Delta y - \sin\alpha\ \ell \tag{A2.4}$$

$$x_3 = \frac{\ell}{\sqrt{z^2 + 1}} \tag{A2.5}$$

Substituting A2.2 through A2.5 into A2.1

$$\ell + \frac{z}{\sqrt{z^2+1}}\,\ell = \cos\alpha\,\ell + \sqrt{\ell^2 - \left(\frac{\ell}{\sqrt{z^2+1}} - \Delta y + \sin\alpha\,\ell\right)^2} \qquad \text{(A2.6)}$$

Expanding terms, rearranging and dividing by ℓ, this becomes

$$\left(\frac{\Delta y}{\ell}\right)^2 - 2\left(\sin\alpha + \frac{1}{\sqrt{z^2+1}}\right)\frac{\Delta y}{\ell} + 2\left(1 - \cos\alpha + \frac{(z + \sin\alpha - z\cos\alpha)}{\sqrt{z^2+1}}\right) = 0 \qquad \text{(A2.7)}$$

Therefore, for a rising water level as in Case 3,

$$\frac{\Delta y}{\ell} = C_{r1} \pm \sqrt{C_{r1}^2 - 2\,C_{r2}} \qquad \text{(A2.8)}$$

where $C_{r1} = \sin\alpha + \dfrac{1}{\sqrt{z^2+1}}$

$$C_{r2} = 1 - \cos\alpha + \frac{(z + \sin\alpha - z\cos\alpha)}{\sqrt{z^2+1}}$$

The same solution was obtained for the analysis of Case 2. For Case 1, a falling water level, the solution obtained was

$$\frac{\Delta y}{\ell} = C_{f1} \pm \sqrt{C_{f1}^2 - 2\,C_{f2}} \qquad \text{(A2.9)}$$

where

$$C_{f1} = \sin\alpha - \frac{1}{\sqrt{z^2+1}}$$

$$C_{f2} = 1 - \cos\alpha + \frac{(z - \sin\alpha - z\cos\alpha)}{\sqrt{z^2+1}}$$

The solutions to these equations represent geometrically

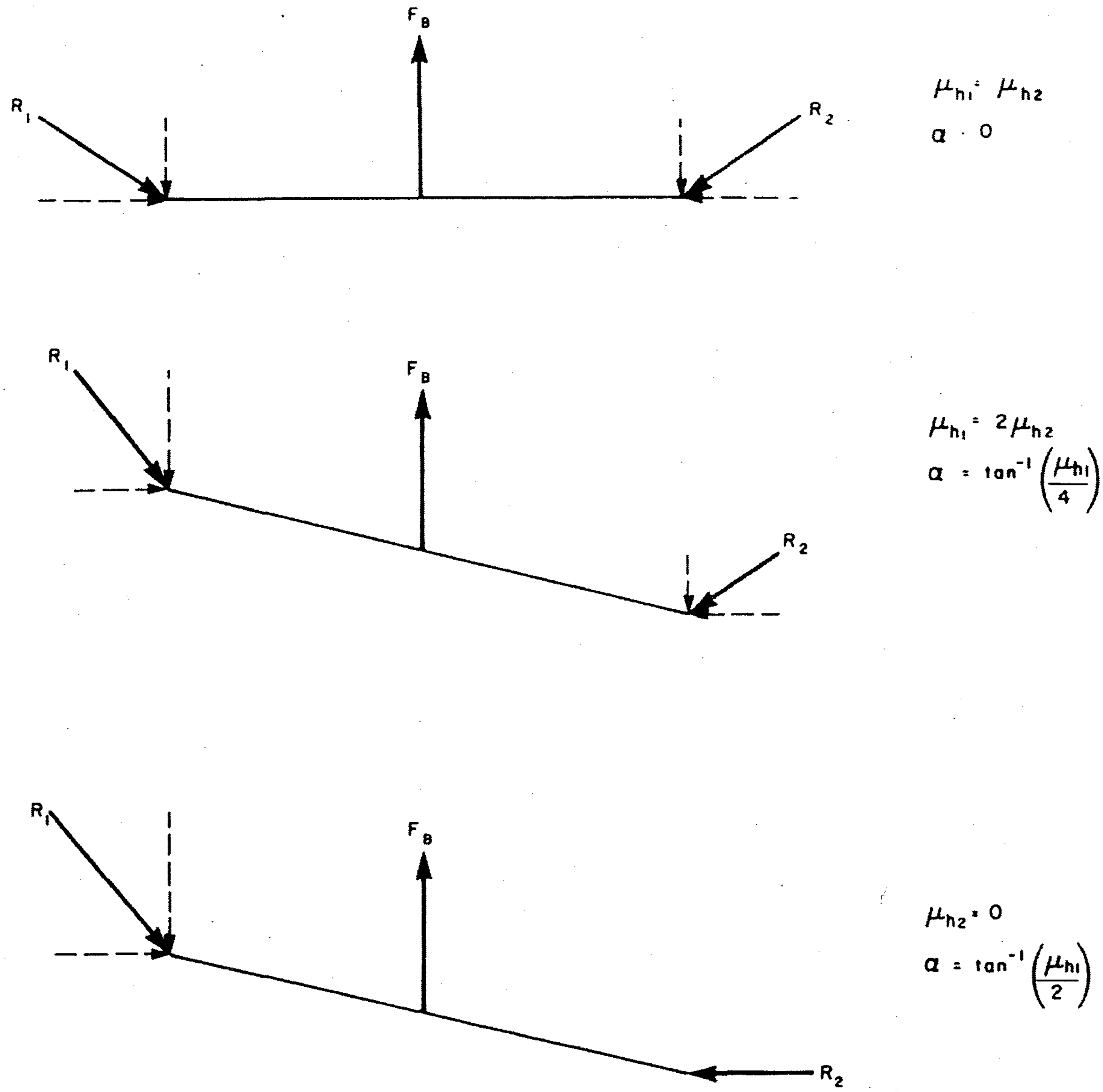

FIG. A1

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

STABILITY OF A SINGLE HINGE LINK

ACRES

possible configurations. The solutions are plotted in figures 4.5 and 4.6. Interpretation of the curves and their limitations are discussed in the text of the report.

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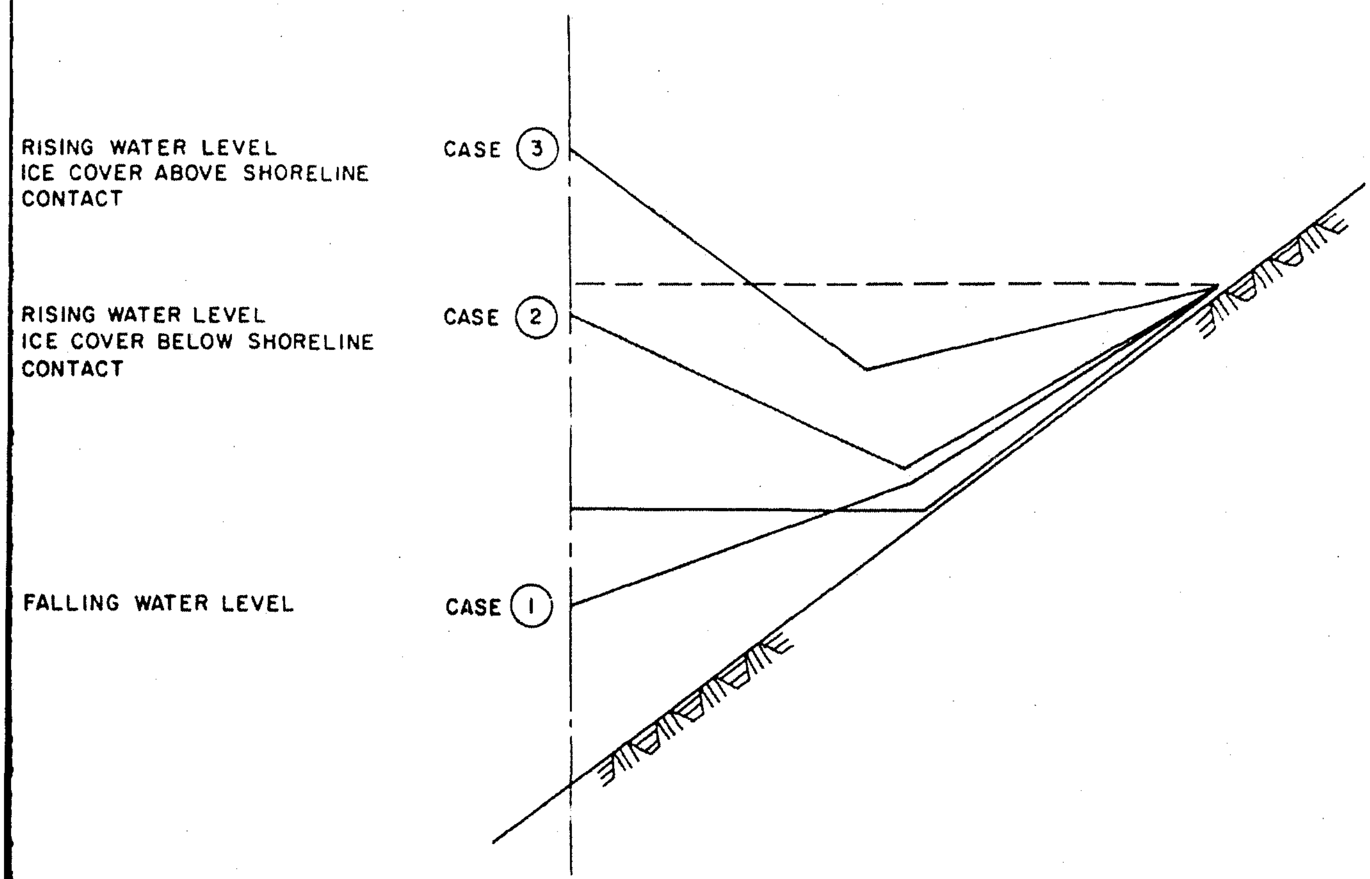

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

DEFINITION SKETCH FOR TWO-LINK STABILITY

FIG. A2

ACRES

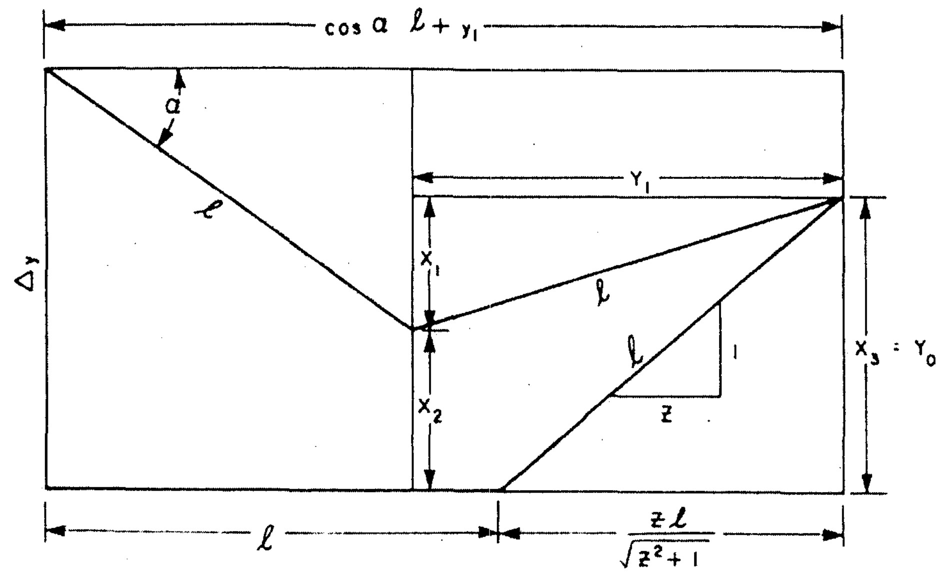

FIG. A3

CANADIAN ELECTRICAL ASSOCIATION

STUDY OF ICE COVERS SUBJECT TO VARYING WATER LEVEL

TWO-LINK STABILITY - CASE 3

ACRES

APPENDIX B

SUMMARY OF INTERVIEWS

APPENDIX B

B.1 - SUMMARY OF INTERVIEWS

Interviews with hydroelectric power utilities were conducted to identify locations where field data could be obtained to calibrate the conceptual model. The utilities were questioned regarding the extent and type of ice problems encountered, solutions to these problems, field data or documentation available at present, and specific observations or comments related to variation in discharge and water level. A list of Interview Contacts follows this section.

The response of the utilities was favorable and a willingness to supply available data or to assist in field studies was expressed, although funding for field studies could not be promised. The observations made by the various utilities confirm the occurrence of ice cover failure under water level variation and formation of some forms of shoreline hinging. However, there is very little documentation available and that is largely qualitative. Solutions to most severe ice problems experienced by the utilities have been developed on the basis of operational experience. Critical limits to cover stability are now rarely exceeded so assumed stability limits will be hard to verify.

A specific, follow-up questionnaire on hinge formation was sent to the utilities where suitable field sites were identified.

Hinging has been observed in some form on the Peace River in British Columbia, on various rivers in Alberta, on the

Winnipeg River in Manitoba, and the Gatineau River in Quebec. Breakup of solid ice covers has been observed on the Peace River in British Columbia, on the Box River in Alberta, on the South Saskatchewan River in Saskatchewan, on the Sturgeon, Mississagi and Abitibi rivers in Ontario, and on the St. John River in New Brunswick. The following sites have been identified as field sites for data collection on the basis of these interviews.

British Columbia - Peace River downstream of G.M. Shrum generating station.

Alberta - Box River upstream of Calgary at Bearspaw Dam.

Saskatchewan - South Saskatchewan River at Coteau Creek/ Gardiner Dam power plant.

Manitoba - Winnipeg River from Slave Falls to Lac du Bonnet.

Ontario - Sturgeon River near Sturgeon Falls at the Crystal Falls generating station.

- Mississagi River north of Thessalon at Aubrey Falls generating station.
- Abitibi River near Fraserville between Abitibi Canyon and Otter Rapids generating stations.
- Niagara River pumped storage generating station.

Quebec - Gatineau River between Paugan and Chelsea.

New Brunswick - St. John River at Mactaquac, Beechwood and Grand Falls generating stations.

Nova Scotia - Wreck Cove generating station.

In response to questions on pumped storage generating stations, no reports of damage or serious ice problems were reported. This is undoubtedly because the ice covers, which are not subjected to shear stress on the underside, simply fail at the shoreline and move freely with water level fluctuations.

B.2 - INTERVIEW CONTACTS

The following utility personnel were interviewed for this study.

Mr. W. B. Bancroft, Manager, System Operations, Nova Scotia Power Corporation

Mr. K. Barrett, Newfoundland Light and Power

Mr. E. Bradford, Assistant Regional Operator, Northeastern Region, Ontario Hydro

Mr. R. W. Carson, Acres Consulting Services, Winnipeg, Manitoba

Mr. R. Cotanen, Northwestern Region, Ontario Hydro

Mr. Cummons, Lewiston Operations, Niagara Mowhawk Power Corporation

Mr. L. B. Davies, British Columbia Hydro and Power Authority

Mr. J. Dobson, Saskatchewan Power Corporation

Mr. P. Dole, Calgary Power

Mr. F. Fonseca, Hydraulic Works Division, Hydraulic Service, Hydro-Quebec

Mr. D. M. Foulds, Water Planning and Management Branch, Environment Canada - formerly with Ontario Hydro

Mr. B. Kartha, British Columbia Hydro and Power Authority

Mr. S. T. Lavender, Acres Consulting Services Limited, Niagara Falls

Mr. J. Littlefield, Utilities Division, Great Lakes Power Corporation Limited

Mr. J. Long, Director of Operations, Newfoundland and Labrador Power

Mr. J. Long, General Manager, Northern Canada Power Commission

Mr. T. Schulte, Calgary Power

Mr. T. Wigle, River Control Engineer, Ontario Hydro

Mr. J. Woodcroft, Eastern Region, Ontario Hydro

Mr. R. Yorke, Niagara River Control, Ontario Hydro