SUSITNA HYDROELECTRIC PROJECT

SURVEY OF EXPERIENCE IN OPERATING HYDROELECTRIC PROJECTS IN COLD REGIONS

VOLUME 3 - APPENDIX D

SUPPLEMENTAL MATERIAL INCLUDED IN RESPONSES

Prepared by
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Prepared for
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6. Surges from Ice Jam Releases: A Case Study by S. Beltaos and B.G. Krishnappen published in the Canadian Journal of Civil Engineering.

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IAHR Ice Symposium 1984
Hamburg

STRATEGIC HYDRO POWER OPERATION AT FREEZE-UP REDUCES ICE JAMMING

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ABSTRACT

In order to facilitate ice cover formation and thereby reduce ice jamming at freeze-up on the river reach downstream of Vittjärv hydro power station in northern Sweden, exstensive excavations have been undertaken In spite of the work carried out, ice jamming still occured resulting in head-losses and inundations.

As a measure against the ice jamming it was suggested to decrease the discharge when ice formation starts and keep it low until the river is satisfactorily ice covered. However, the problem is to reduce the flow in the lower part of the river and still be able to generate electricity at high capacity in stations along the upper parts of the river. This requirement can partly be met by prelowering of a "mid-river" reservoir.

Considering the planning needed for the operation of the whole river system it is extremely important that the right time for the low discharge period is correctly predicted. The method worked out to cope with this problem includes monitoring of water temperatures and water levels, weather fore-casting, ice surveys etc.

To speed up ice cover formation a specially designed ice boom has been tested. Also ice breaking and ice-sawing on reaches with rapid shore ice formation have been used. The paper presents details of the indicated method together with two years of practical field experience.

INTRODUCTION

Vittjärv hydro power plant is located in the lower part of Lule River, in the north of Sweden. See Figure 1. The station has a head of 6 m and a capacity of 690 m^3/s and it was taken in operation during the winter 1974/75. Already the first year of operation serious ice jamming occurred during freeze-up. Downstream of the station ice jams caused a head-loss of more than 2 meters.

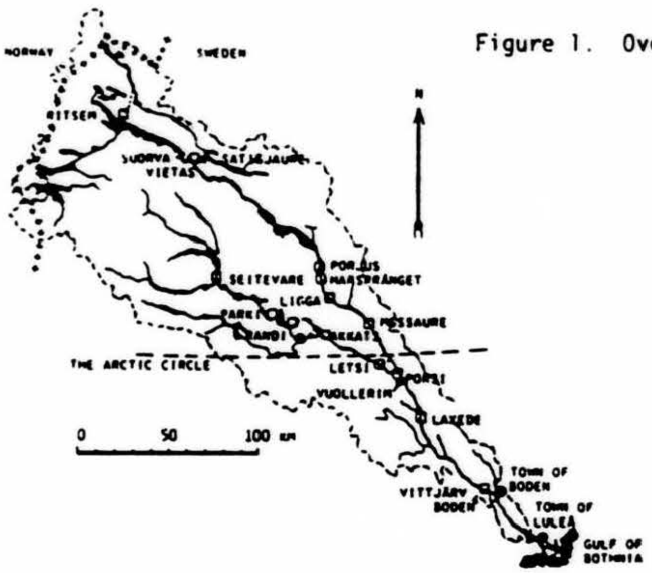

Figure 1. Overall map of Lule River.

To facilitate ice cover formation and thereby reducing frazil production and ice jamming, extensive excavations have been undertaken in the river. These works were terminated 1978. The details regarding these works as well as experienced ice problems are reported by Jensen (1981). A sketch of the river between Vittjärv and Boden power stations is shown of Figure 2.

In spite of the work carried out in the river, ice jamming still occurred after 1978. These ice jams resulted not only in head-losses in Vittjärv power station but also in inundations and inflow of water to pump stations and houses located on the banks of the river. Figure 3 shows water profiles between Vittjärv and Boden measured after the excavations were finished. The profiles refer to somewhat different discharges but clearly show that some years were much worse than others. For example, due to incomplete ice cover formation ~~in the early winter 81/82, frazil~~ was produced along the ice free reaches upstream the Trångfors bridge. The ice deposited into

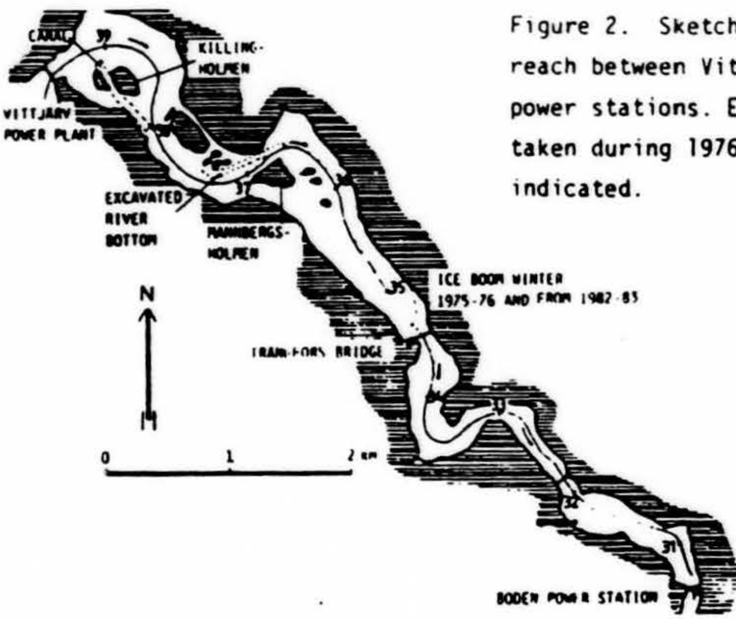

Figure 2. Sketch of the river reach between Vittjärv and Boden power stations. Excavations undertaken during 1976 to 1978 are indicated.

hanging dams and it was necessary to temporarily decrease the flow from 600 m^3/s to 400 m^3/s in order to stop further rise of the water level. The water profile dated 811218 on Figure 3 shows the effect of the hanging dam created downstream of Trångfors. The flow reduction lasted for 6 days and the extra cost for alternative power production was estimated to 4 million Swedish Crowns (about 0.5 million dollars).

The experienced ice problems cause economic losses. More important, though, are the plans for winter-time peak power generation. Ice problems might increase if strong peak power regulation is introduced (Billfalk, 1982). It is therefor most important to find methods whereby also peak power operation can be handled without causing serious ice troubles. To meet this requirement further excavations as well as an increase of the maximum permissible reservoir level at Boden power station are considered. To improve the situation before such measures could be undertaken a procedure for strategic operation of the river during freeze-up has been worked out. This procedure will probably be required even as a complement to further excavations etc

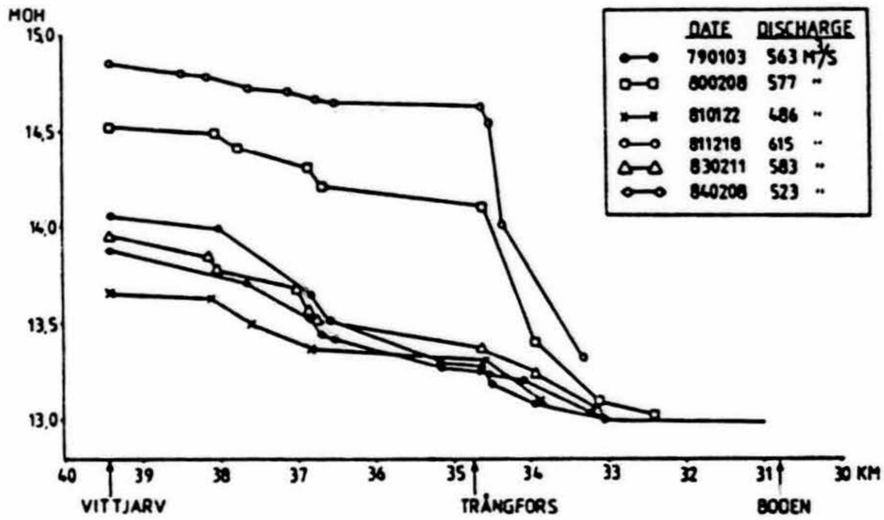

Figure 3. Measured water surface profiles for winters after 1978.

PROPOSED METHOD

After the difficulties experienced 1981/82, an investigation about the causes of the problems was undertaken. This investigation clearly indicated the role of the discharge rate. Low discharge at freeze-up results in rapid ice cover formation and negligable ice jamming while high discharges result in incomplete ice cover formation and ice jamming. As a measure against ice jamming it was therefor suggested to decrease the discharge when ice formation starts and keep it low until the river is satisfactorily ice covered. However, this simple principle can be hard to accomplish from a power production point of view.

Upstream of Vittjärv a number of the most important hydro power stations in the country are situated; see Figure 1 The problem is to reduce the flow in the lower part of the river and still be able to generate electricity at high capacity in stations along the upper part of the river. This requirement can be met by pre-lowering one or two major reservoirs, where the surplus of water from the upper part of the river can be stored, while low discharge is maintained along the lower part. For example, running Letsi power station in the southern branch of the river adds 200 m^3/s to the main river. Requiring a flow rate of 300 m^3/s

at Vittjärv means that most of the water coming from the stations in the upper part of the main river must be stored in the Messaure reservoir. By lowering the Messaure reservoir prior to such an operation the required discharge rate at Vittjärv can be kept for a couple of days. More extended periods with restricted flow may require reduced production even in the stations upstream of Messaure. This might be possible without economic losses if hydro power installations in other rivers are not fully utilized.

Considering the planning needed for the management of the whole river system, it is extremely important that the appropriate time for the low discharge period is determined with highest possible certainty. A second attempt with repeated lowering of the Messaure reservoir etc., may probably not be possible. The method worked out to cope with this problem includes monitoring of water temperatures and water levels, weather fore-casting, ice surveys etc. Information gathered during the critical time period was discussed within a small management group. This group suggests when and how to decrease the flow and what extra measures should be taken. Before discussing practical field experience, these extra measures as well as the data acquisition methods will be briefly presented.

The most important parameter for the prediction of the time when ice formation starts, is the water temperature. Water temperatures are measured every morning at most power stations along the river with mercury thermometers, accurate to whitin $\pm$ 0,01 ^{o}C. In addition a quartz thermometer has been installed in one of the inlet sumps at Vittjärv power station. Data from this instrument is transmitted to the operation center for Lule River, situated in Vuollerim.

Upstream and downstream water levels are measured continuously a both Vittjärv and Boden power stations. Just upstream of the Trångfors bridge an extra water level gauge has been installed. Data from this gauge is also transmitted to Vuollerim. The purpose of these measurements is to detect the beginning and evolution of ice jamming downstream of Trångfors.

Observations of the evolution of shore ice and later on the formation of fragmented ice covers are made by the local hydrologic departement, responsible for field surveys in the area. During the critical time period this department produces maps showing the extension of surface ice along the actual river reach.

Before and during freeze-up, long term weather fore-casts (5 days) are ordered daily. These weather fore-casts, together with information about water temperatures and the actual surface ice situation, form the basis for a discussion within the management group responsible for the descission to reduce the flow-rate. This group consists of representatives from the local and the central operational departments, the local hydrologic survey department and from the Laboratory in Älvkarleby. Up-to-date information is transmitted to the members of the group by means of Telefax. Discussions can be held daily through telephone meetings, which has proved very useful.

The narrow section at the Trångfors bridge (Trångfors means "Narrow Rapids" in Swedish)is one of the key points to the experienced difficulties. Even at low discharges the progression of the ice cover, starting from Boden power station, is halted downstream of Trångfors. In order to secure rapid ice cover formation upstream of Trångfors an ice boom has earlier been tested upstream of the bridge. The boom proved effective in initiating an ice cover upstream of its location. Downstream thereof, however, a long reach of the river maintained open water until late in the winter. Ice production on this reach caused underhanging dams further downstream and the boom was therefor removed after a one year test (Jensen, 1981).

Trying to avoid the drawbacks with the old ice boom a new concept has been tested. The new boom is located at the previous location. See figure 1. The new idea is to keep a 100 m long section in the central part of the boom open at the beginning of freeze-up, permitting drifting ice to pass so as to contribute to the build-up of an ice cover from downstream. When the ice cover has reached close to the bridge the opening in the boom should be closed and an ice cover could start progressing from the boom leaving just a short reach with open water downstream of the bridge. At the left bank the boom wire is equipped with a force meter permitting continuous registration of the load.

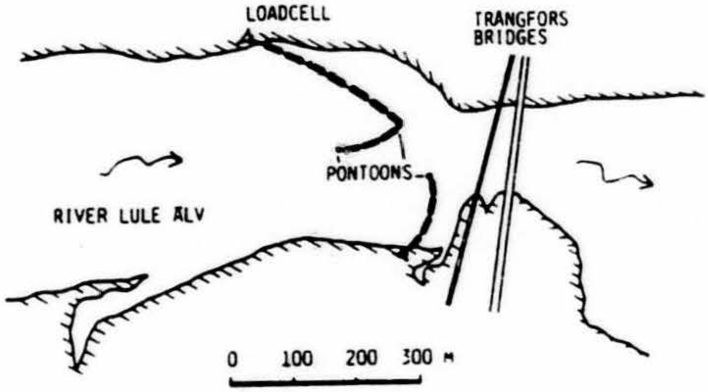

Figure 4. Ice boom at Trångfors with a central gap that can be closed.

The time period with low discharge at freeze-up must for economical reasons be made as short as possible. It is therefor important that cold weather, promoting rapid ice formation, prevails once the flow has been reduced. If natural production of drifting ice is low, ice-breaking from areas where shore ice has formed could attribute to the growth of the fragmented ice cover. This technique as well as ice-sawing has been tested. Experience from these works will be discussed in the following section.

FIELD EXPERIENCE

The described measures for achieving more complete ice cover formation were first tested during the winter 1982/83 (Billfalk, 1983). Early that winter the discharge at Vittjärv was reduced to 300 m^3/s when conditions for ice formation seemed favourable. Ice covers then rapidly developed from Boden power station to section 32.5 (km) and also from the ice boom at Trångfors to Mannbergsholmen, in spite of the opening in the boom (see Fig 2). Bridging obviously occurred at about section 35 at the low discharge and the boom opening never had to be closed that winter. The load on the boom did hardly increase during freeze-up compared to open water conditions.

In order to speed up the ice cover formation from section 32.5 towards Trångfors, breaking of shore ice from the wide sections between section 34.0 and 34.4 was started. The boat used for that purpose was a steel boat about 5 m long, which previously had been used in connection with timber floating. By running the boat towards the shore ice, long cracks could be created, thereby loosening floes sometimes on the order of 1000 m^2. If cracks did not appear the boat could be run back and forth creating a track whereby a big floe could be loosened, provided the ice thickness was less than about 0.1 m. Although the boat was somewhat small for the job, about 70.000 m^2 surface ice could be broken in less than 2 days. The ice front was thereby artificially moved about 500 m upstream.

After initial ice cover formation the discharge was kept at 300 m^3/s for about 1 week. The flow was then gradually increased to about 600 m^3/s. The avarage flow during the rest of the winter was on the order of 450 m^3/s.

The autumn in 1983 was extremely rainy and all reservoirs were almost completely full at the beginning of the winter. It was therefor important not to reduce the flow until it was absolutely necessary. Due to a very sudden cold spell, some trouble with frazil formation, which temporarily clogged the intakes at Vittjärv, occurred before the flow was reduced to 300 m^3/s this winter. At the beginning of freeze-up drifting ice passed the opening in the ice boom and the ice cover progressed to section 33.0 in a couple of days. However, even this year ice cover formation occurred fairly early upstream of the ice boom in spite of the open gap. The load on the ice boom wire this year raised to about 70 kN during the ice formation process.

Due to the high degree of reservoir filling the discharge was kept at 300 m^3/s not more than 3 days. The flow was then gradually increased to about 550 m^3/s in 4 days.

Even 1983 ice-breaking was used to reduce the open water area downstream of Trångfors. Due to cold weather this work had to be stopped after a few days. The ice front had at that time reached section 33.5, leaving about 1 kilometer of open water downstream of Trångfors. Downstream of Vittjärv power station it was open water down to Mannbergsholmen. Due to cold weather ice production on the open reaches was high and hanging dams started to develop downstream of these open reaches. In order to further reduce the open water area a specially designed ice saw was used (ice breaking with the toat was no longer possible). The ice saw is mounted on a sled and is driven by a 30 HP engine. By this machine the ice front downstream of Trångfors was fed with large floes of shore ice. The front thereby moved to section 34.0 and the remaining 500-600 m open reach was considered acceptable.

One experience of the two years of "controlled" ice cover formation is that the discharge might be kept somewhat higher than 300 m³/s at freeze-up. Bridging might thereby be avoided upstream of the ice boom and the ice cover front may reach closer to the Trångfors bridge without ice breaking or sawing. Once the ice front has reached there the gap in the boom should be closed.

The open water area at Trångfors after initial ice cover formation and complementary ice breaking and ice sawing is shown on Figure 5.

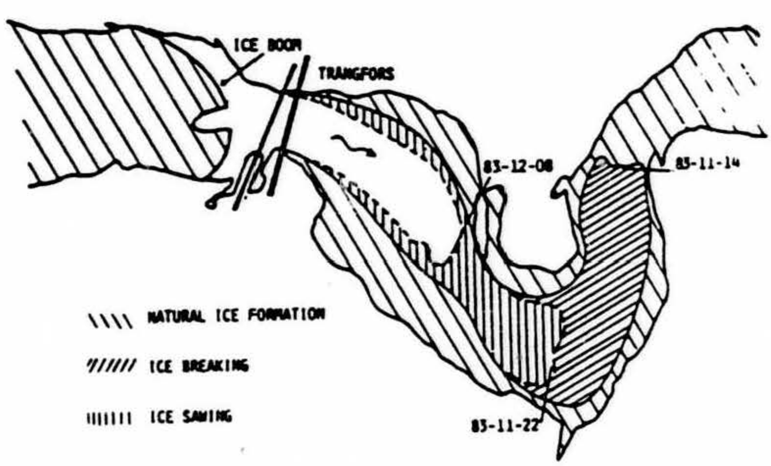

Figure 5. Open water area at Trångfors after initial ice cover formation as well as after breaking and sawing of shore ice.

CONCLUSIONS

Two years of experience of the method with reduced flow and supervision of the early ice cover formation are now available for the river reach between Vittjärv and Boden power stations. This short time period does of course not permit any general conclusions. The following preliminary conclusions have been drawn, however:

- Reduction of the flow at freeze-up permits rapid ice cover formation and the development of significant hanging dams is avoided. Considering head-losses caused by ice jamming the two latest years were as good as the best year experienced before; see Figure 3 (note that the profiles refer to different discharge rates).
- Having a management group for descissons of how to handle various problems that arize at freeze-up is of major importance. More or less daily contacts within this group during the critical time period have shown to be very useful.
- People involved in the local operation of the river have shown great interest for the tested procedure. These people now have been able to get a theoretical background to their practical experience. This "educational effect" will probably be very favourable in the future.
- The specially designed ice boom has so far been of minor use. However, the boom will probably be of vital importance when trying to achieve rapid ice cover formation at higher discharges than 300 m^3/s.
- Both breaking and sawing of shore ice have proved to be useful methods for building up fragmented ice covers. A boat may be used for ice breaking at the early freeze-up. When the shore ice has grown thicker only ice sawing is possible. It must be pointed out, though, that both methods are quite time consuming and that the applicability depends on local conditions.

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13.3

AN ESTIMATE OF ICE-DRIFTED SEDIMENTS BASED ON THE MUD CONTENT OF THE ICE COVER AT MONTMAGNY, MIDDLE ST. LAWRENCE ESTUARY

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ABSTRACT

Dionne, J.-C., 1984. An estimate of ice-drifted sediments based on the mud content of the ice cover at Montmagny, Middle St. Lawrence Estuary. Mar. Geol., 57: 149—166.

Recent measurements made at Montmagny, a locality on the south shore of the Middle St. Lawrence Estuary, 70 km northeast of Quebec City (47°N), give an idea of the volume of fine-grained sediments incorporated in the ice cover and allow an estimation of the annual load drifted by ice. At this locality, a mean thickness of 10 cm of mud was encountered in the ice cover over an area of approximately 20 km^2. Thus, the total load of fines may be as much as 4×10^6 tonnes (t). It is estimated that upon melting at break-up, about 15% of this load returns to the Montmagny tidal flat while the remaining volume is carried to the offshore zone. Considering breakup characteristics, it is estimated that about $1.5—2 \times 10^6$ t of fines return to the turbidity zone while the remaining load is ice-drifted outside that zone. Since the shore area of the Middle St. Lawrence Estuary covered by ice during the winter is approximately 60 km^2, it is calculated that a load of $5—6 \times 10^6$ t of sediment incorporated in the ice cover could escape from the turbidity zone annually. To this load should be added another 4×10^6 t of suspended matter which come from the freezing in situ of the turbid water in the offshore zone. An annual output by ice drifting of 10×10^6 t of sediment is thus likely and is in great contrast to the output during the ice-free season of approximately 1×10^6 t. In the Middle St. Lawrence Estuary, the annual output almost equals the input. Consequently the sediment budget is virtually in a state of equilibrium, which helps to explain why there is very little permanent mud deposition in the shore and offshore zones today. It is concluded that ice processes largely control the sedimentary budget of the turbidity zone of the Middle St. Lawrence Estuary, a particular environment within a mid-latitude inner continental shelf which is partly dominated by ice.

INTRODUCTION

Although the St. Lawrence is one of the major estuaries in the world, relatively little is known about the sedimentology of this large water body (Nota and Loring, 1964; Loring and Nota, 1973; Khalil and Arnac, 1975; Cremer, 1979). A better knowledge of the sedimentary budget and of the processes in action in the Middle St. Lawrence Estuary, i.e. the area between Quebec City and the Saguenay River (an area 180 km long and 2—24 km wide; Figs.1 and 2) is needed both to understand correctly the complexity of this dynamic environment and to provide a useful tool for planning the development and

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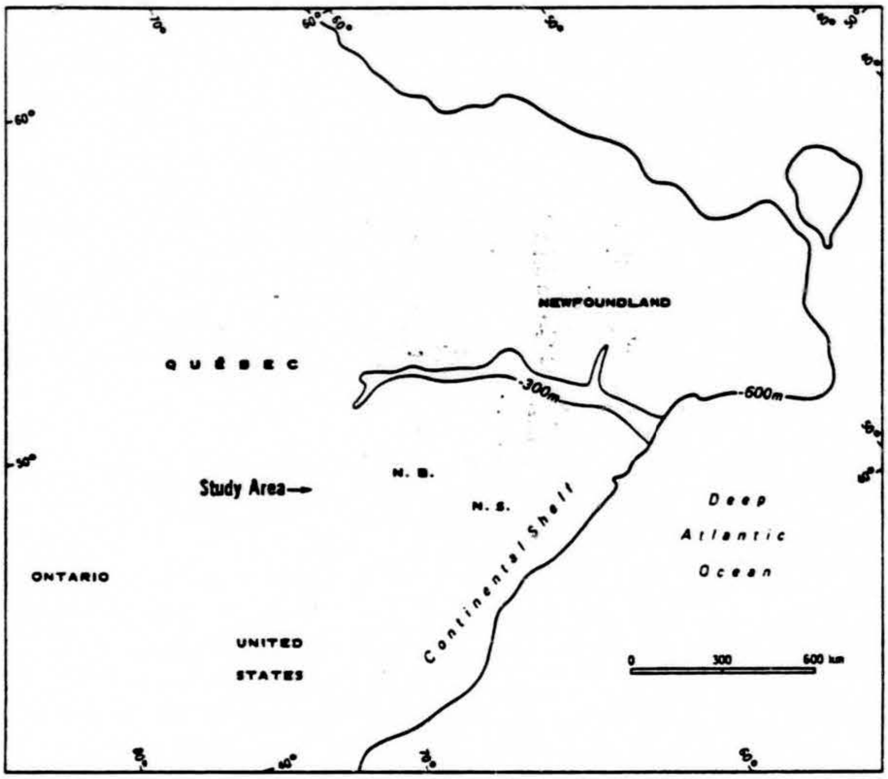


Fig.1. Location map of the study area showing the relationships with the eastern Canadian continental shelf and Atlantic Ocean.

the preservation of the shore zones presently subjected to erosional processes (Dionne, 1979) and to pollution (Sérodes, 1980).

Recent studies related to sedimentation in the St. Lawrence Estuary deal both with the offshore zones (D'Anglejan et al., 1973, 1974, 1981; Brisebois, 1975; CENTREAU, 1975; Soucy et al., 1976; D'Anglejan et Brisebois, 1974, 1978; Kranck, 1979; Silverberg and Sundby, 1979; D'Anglejan, 1981a; Couillard, 1982), and the shore zones (Sérodes, 1980; Allard, 1981; D'Anglejan, 1981b; Drapeau and Morin, 1981; Troude et al., 1981; Dubé, 1982; Sérodes et al., 1982). Even though particular attention has been given to ice action in the tidal zones (Dionne, 1968a, b, c, 1969a, b, 1971a, b, 1972a, b, 1973, 1974a, b, 1980; Allard and Champagne, 1980), until recently only gross estimates have been made of the volume of sediment incorporated annually i. the ice (Dionne, 1981a, b). However, Nota and Loring (1964, p.233) recognized sometime ago that ice should be considered as a prominent factor of erosion, transportation and deposition in the St. Lawrence Estuary and Gulf.

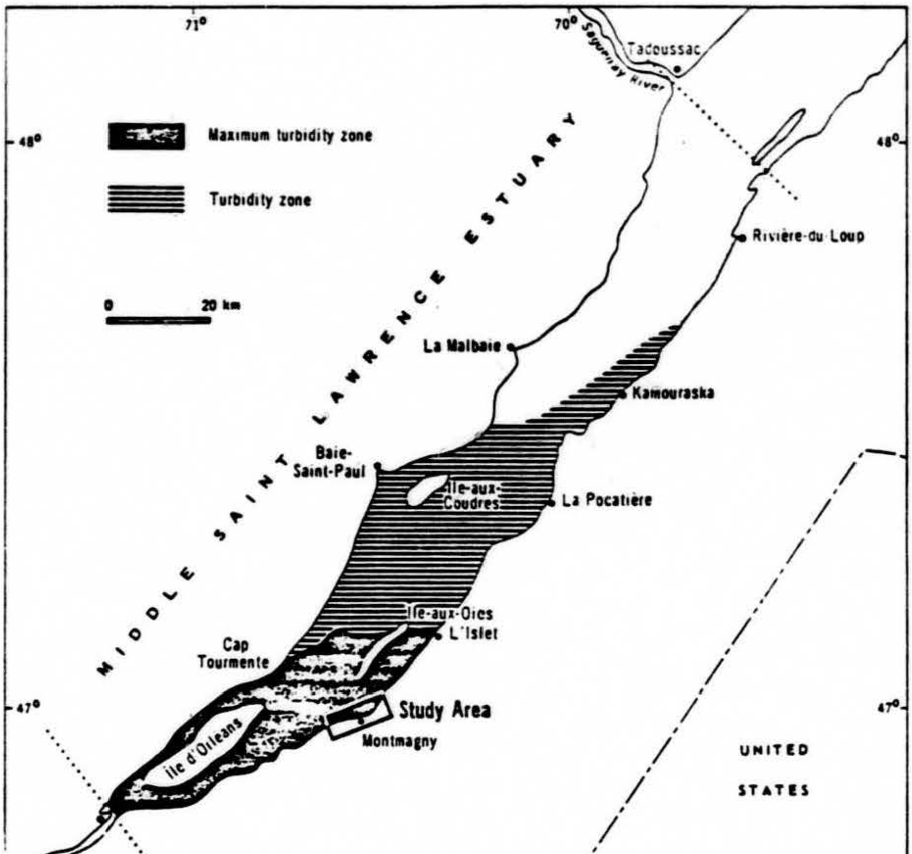


Fig.2. Map of the Middle St. Lawrence Estuary showing three major units: (a) maximum turbidity zone; (b) medium turbidity zone; and (c) turbidity-free or very low turbidity zone, according to D'Anglejan et al. (1981). The study area is located within the maximum turbidity zone.

It is the purpose of this paper to report preliminary data on the volume of fine sediment incorporated annually in the ice cover, particularly at Montmagny, to discuss briefly the significance of ice rafting in the sedimentary budget of the Middle St. Lawrence Estuary, and to point out its importance to the evolution of some high-latitude continental shelves.

CHARACTERISTICS OF THE MONTMAGNY TIDAL FLAT

The Montmagny tidal flat is located on the south shore of the Middle St. Lawrence Estuary, approximately 70 km northeast of Quebec City (lat. 47° N). From a sedimentological point of view, this area can be considered as a particular environment within the North Atlantic inner continental shelf (Fig.1). The tidal flat extends from Point St. Thomas to the west, to Cape St. Ignace in the east, a distance of approximately 15 km. The mean width of the flat is 1500 m, but locally it extends up to 3 km at lower low tide for an average area of 20 km^2 (Fig.3). It is set in a large open embayment facing the Montmagny Archipelago. The depression which is cut into Cambro-Ordovician folded slate and sandstone formations of the Appalachian Province, is filled with fine-grained Quaternary deposits. Pleistocene marine clays (Goldthwait Sea) several meters in thickness underlie Holocene and Recent stratified silts and fine sands a few centimeters to a few decimeters in thickness, and locally up to one meter or more, along the major channels of the tidal flat.

The Montmagny tidal flat is composed of two major units: a relatively wide marsh up to 500 m wide set at the higher level, and a broad muddy and sandy tidal flat extending from the marsh down to the lowest low tide level. Mean tides range from 4 to 5 m and large spring tides are up to 6 m. The area is considered as a macrotidal environment characterized by a broad tidal platform which slopes gently seaward with a gradient ranging from 1 to 5 m km^{-1}. This tidal flat is entirely ice-covered for several months each winter. Freeze-up usually occurs in December and break-up in April. The ice cover is commonly 60—100 cm thick, but locally thicknesses up to 125—150 cm have been measured. The ice cover extends seaward as far as the −5 m isobath for about 2—3 months. Throughout the winter in the offshore zone, floes of various size move upstream and downstream according to wind direction and tidal currents.

The Montmagny tidal flat is located in the upper section of the Middle St. Lawrence Estuary, an area comprising a high turbidity zone extending from Quebec City to Cap aux Oies on the north shore and to La Pocatière on the south shore (Fig.2). In this turbidity zone, the suspended matter values vary considerably from place to place. Generally, turbidity decreases downstream, shoreward and from the bottom to the surface. According to D'Anglejan et al. (1973), Silverberg and Sundby (1979) and D'Anglejan (1981a), the suspended matter values in the turbidity zone vary from 10 to 450 mg l^{-1} during the summer. No data are available on the turbidity during the ice season. The suspended particulate matter is mainly composed of silt and clay, with a varying proportion of organic debris (Kranck, 1979; Pocklington and Leonard, 1979). Illite and chlorite are the two main components (up to 94%) of the clay minerals of the suspended matter (D'Anglejan et al., 1973).

MODERN DAY SEDIMENTATION

Like most other flats of the Middle St. Lawrence Estuary, deposition occurs year round on the Montmagny tidal flat. However, two main periods of mud and fine sand deposition do exist, one during the summer and the other during the winter. It is well known that deposition today is not permanent in the turbidity maximum zone. On the contrary, it is cyclic and

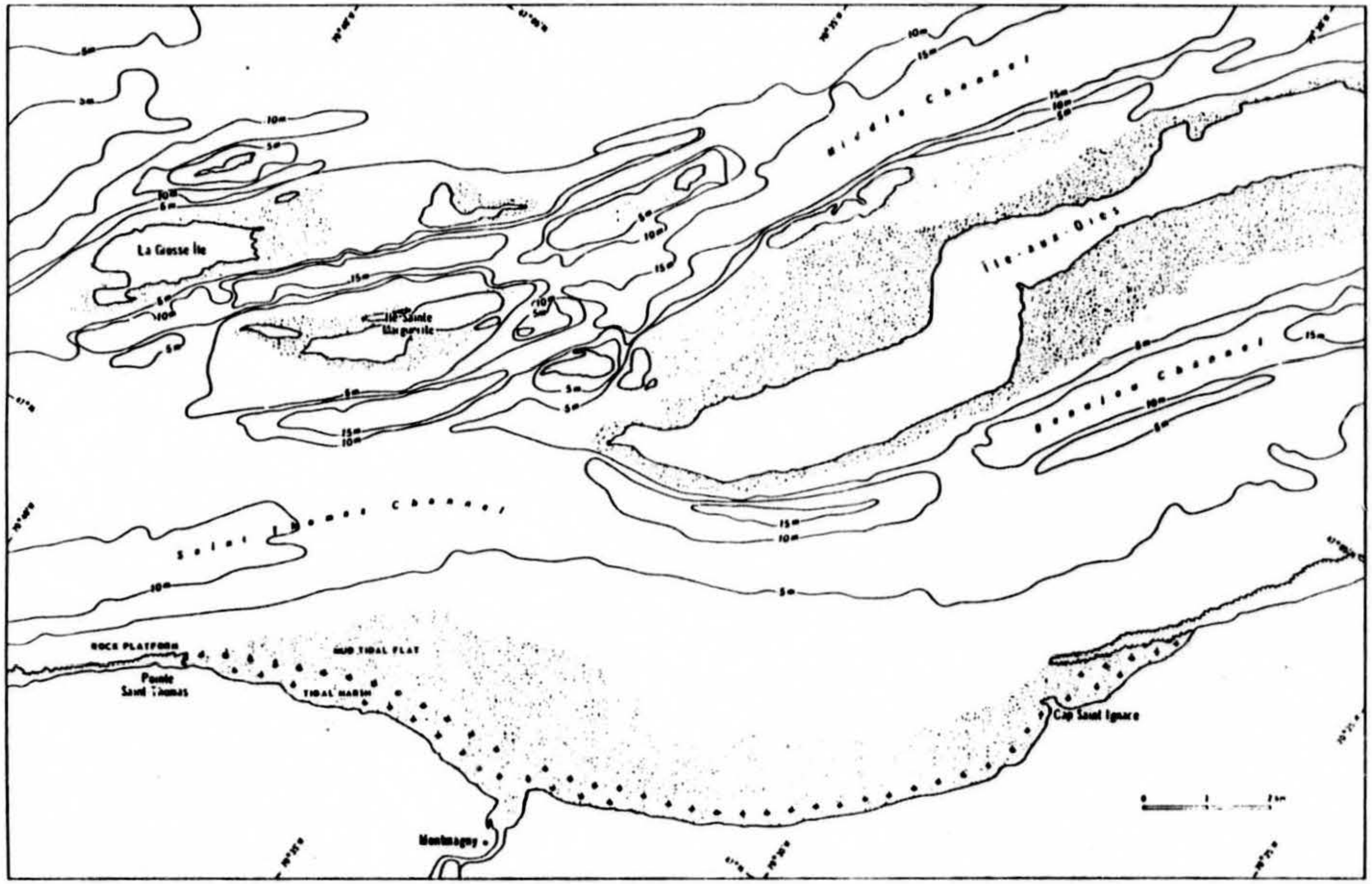


Fig.3. Map of the Montmagny tidal flat and adjacent offshore areas.

dynamic, and subject to several periods of erosion during the year (Sérodes, 1980; Troude et al., 1981; Dubé, 1982).

During the summer, mud deposition is largely concentrated in the lower marsh zone, the vegetation cover favoring the deposition of suspended matter from July to September. Although a few scattered patches of mud up to 45—50 cm thick do occur in the marsh, most commonly the average thickness of the mud layer deposited in summer is 20 cm and rarely exceeds 25—30 cm. Deposition on the bare tidal flat is less than in the marsh: 5—15 cm only. Generally at the end of September or the beginning of October, waves and tidal currents extensively rework the freshly deposited mud and return it to the offshore turbidity zone (Sérodes, 1980). Consequently, it is difficult to determine how much sediment deposited during the ice-free season is left over each year. Preliminary measurements at Cape Tourmente on the north shore of the St. Lawrence (the site which has the highest rate of summer deposition), indicate a mean increase of 5—8 mm per year over the last 30 years (Troude et al., 1981). These values compare well with the mean rate of sedimentation at Lee Bay, North Sea (Reineck, 1980). However, if one considers that at Montmagny, the Holocene and Recent stratified silts and fine sands unit overlying the Pleistocene marine clay is particularly thin (commonly only a few decimeters only), the annual permanent deposition in the tidal flat is very small today.

The other period of deposition is the winter. Although the tidal flat is entirely ice-covered during that period, sedimentary processes are still active under the ice cover, particularly in the bare mud flat. As the ice cover is not bound to the bottom, the fluctuating level of the water related to the tidal cycles allows a daily rise and fall of the ice cover. Turbid water introduced under the ice cover at high tide allows mud and fine sand to sediment in this environment. Consequently, at the end of the winter, a soft and liquid mud layer, 10—25 cm thick, covers large areas of the bare tidal flat (Dionne, 1980, 1981a, b). The situation differs considerably in the tidal marsh, because in that zone the ice cover which is usually adfrozen to the bottom does not allow penetration of turbid water, so that little or no deposition of mud occurs during the winter. Of the winter deposition, little remains over long periods throughout the tidal flat. Commonly after a few storms, most of the liquid and fresh mud is swept away.

INCORPORATION OF SEDIMENT INTO THE ICE COVER

The deposition of fine-grained sediments under the ice cover is not the only noteworthy aspect of Montmagny tidal flat sedimentation during the winter. On the contrary, a large quantity of sediment is caught up within the ice cover in various ways. Three major processes of incorporating sediment into the ice are commonly observed in the Middle St. Lawrence Estuary:

(a) Freezing at the base of the ice cover, at low tide, when the ice rests directly on the bottom. In this way, thin laminae of mud are progressively incorporated into the ice sheet to form a sequence 10—25 cm thick. This sequence can be observed easily, at break-up, directly in the ice cover itself and in the several ice floes which may have been left on the tidal flat at low tide (Figs.4 and 5).

(b) Incorporation of fine-grained sediments can also be deposited on the surface of the ice cover when it is occasionally submerged during the highest spring tides (Fig.6). Sediments are also introduced through the numerous tidal cracks and other openings in the ice cover. At high tide, hydrostatic pressure under the ice cover is such that turbid water flushes through the cracks leaving large volumes of mud at the surface of the ice sheet (Fig.7).

(c) Freezing in situ of turbid water (Fig.8). This is a common process in the Middle St. Lawrence Estuary and occurs throughout the cold season both in the shore and offshore zones. Preliminary observations indicate that a few million tonnes of suspended matter may drift seaward in this way annually.

These three major processes and possibly also other minor mechanisms incorporate a large volume of fine-grained sediment into the ice cover annually. The storage of sediment in the ice probably reduces significantly the turbidity values of the Middle St. Lawrence Estuary during the winter.

OBSERVATIONS ON SEDIMENT CONTENT IN 1981

The favorable conditions that prevailed in 1981 permitted the measurement and an estimate of the sediment content of the ice cover at Montmagny

Fig.4. Fine-grained sediments incorporated into the ice cover at Montmagny; sediments are frozen to the base and also interstratified with ice (4.7.73).

Fig.5. A view of an ice floe at Montmagny showing a sequence about 40 cm in thickness of thin layers of mud interstratified with ice and squeezed between two layers of clean ice (4.8.69).

(Fig.9). During that winter, the ice sheet over the bare mud flat was only 60 to 75 cm thick. However, in March most floes examined showed an average 10 cm-thick sequence of layered fine sediments incorporated into the ice. In addition, natural windows in the ice cover, explained by the removal of ice fragments by hydrostatic pressure, revealed the presence of several thin laminae of mud having a mean thickness of 10 cm within the ice cover itself (Figs.10 and 11).

Assuming that the sediment content of the ice cover at Montmagny in 1981, as determined from several measurements, was on average 10 cm thick, the 20 km^2 of the area could contain approximately 4×10^6 t of sediment (wet weight).

This quantity of fine sediment in the ice cover may appear surprisingly high to those who are not familiar with cold region tidal flats. However, it compares well with recent observations made in the Bay of Fundy (Gordon and Desplanque, 1981), and also with some arctic environments (Campbell and Collin, 1958; Barnes et al., 1982). It is not known yet if 1981 was an average or an exceptional year for mud content of the ice cover at Montmagny. Should it be an average year, the sedimentological significance of that process would have major consequences; it implies that a large proportion of this load can escape the turbidity zone by ice drifting. The numerous field observations made at breakup, each year since 1967, at many localities

Fig.6. The ice sheet at L'Islet is covered by a layer of fresh mud, a few cm thick, due to submergence at high spring tide (4.7.74).

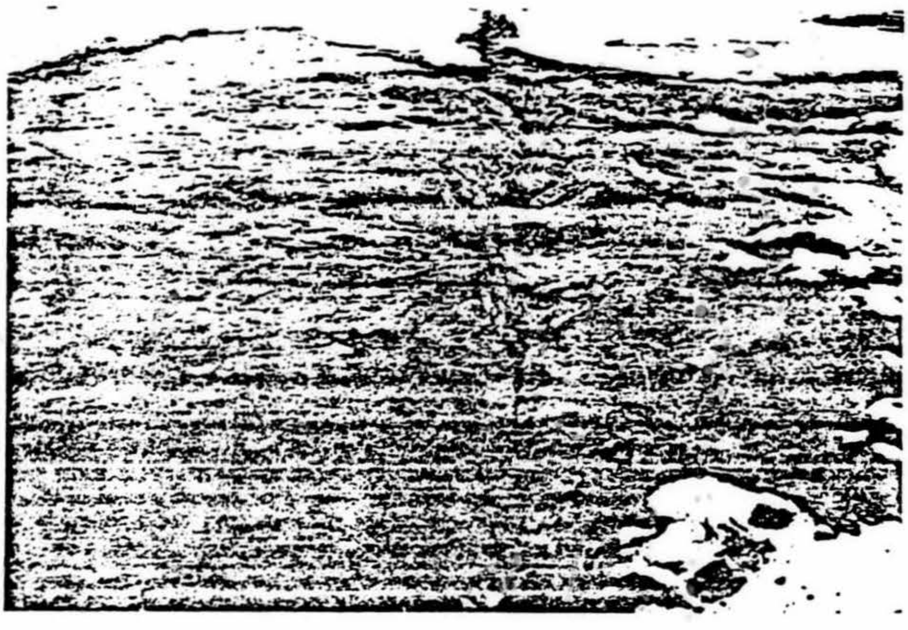


Fig.7. A tidal crack in the ice cover at Montmagny through which mud is introduced at the surface (4.15.72).

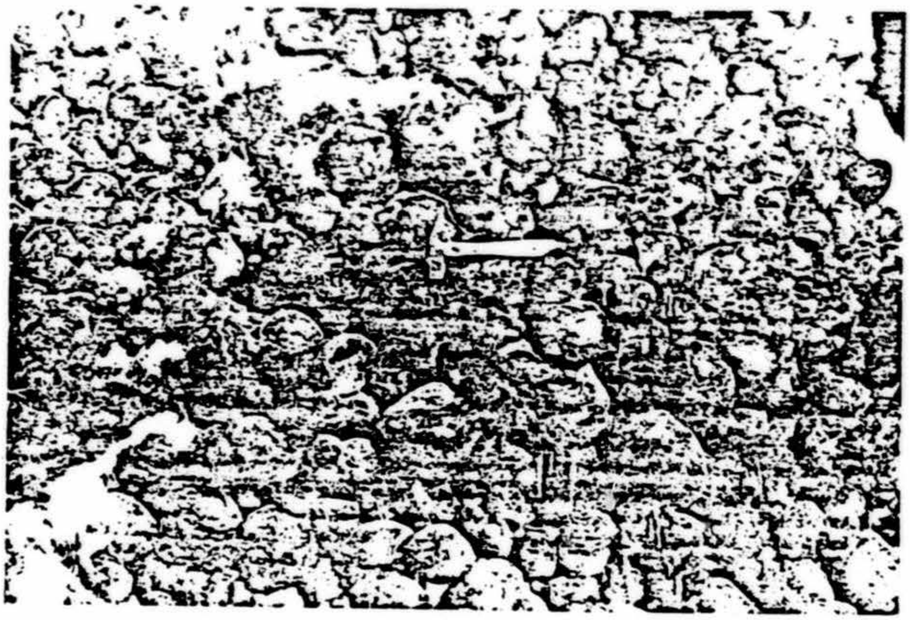
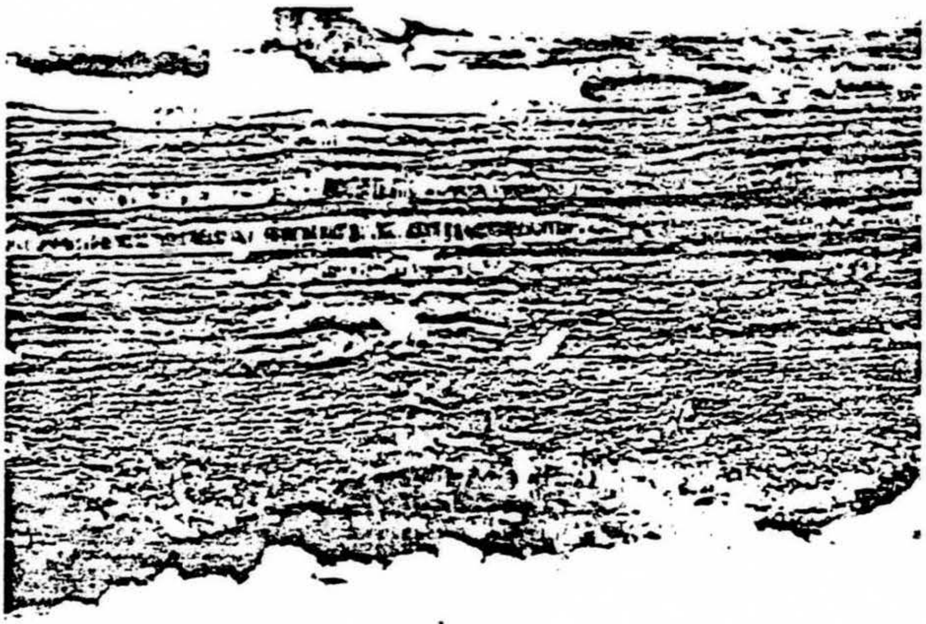


Fig.8. A close-up view of an ice floe made up of dirty ice cobbles, at Montmagny (12.4.71).

Fig.9. A general view of the ice cover at Montmagny during the 1981 winter; note that the surface is largely covered by mud (2.22.81).



Figs.10 and 11. Mud content of the ice cover at Montmagny in 1981; numerous laminae of mud and ice are interstratified; the ice cover is about 60 cm thick (3.8.81) in the Middle St. Lawrence Estuary, suggest that 1981 was not an exceptional year as far as sediment concentration within the ice cover is concerned. Consequently, these preliminary data lead to the following discussion.

DISCUSSION

A major question to be answered is, "what happens to the sediment incorporated into the ice cover?" Do the 4×10^6 t calculated for the Montmagny tidal flat drift away at break-up? If so, where exactly do they go?

As far as is known from observations made during the last 15 years, only a portion of the fine-grained sediment incorporated into the ice cover drifts outside the area of the high turbidity zone of the Middle St. Lawrence Estuary. It was estimated (Dionne, 1981d) that upon melting and washing by waves about 15% of the ice-bound sediment returns directly to the tidal flat during breakup, about 45% returns to the turbidity zone, while the remaining 40% drifts seaward. In other words, $1-1.5 \times 10^6$ t of fine-grained sediment from the ice cover of the Montmagny area could escape the high turbidity zone annually.

The Montmagny tidal flat is only one of the several tidal flats within the Middle St. Lawrence Estuary affected by the process of ice drifting. There are other large ice covers at La Pocatière, l'Islet, Ile-aux-Oies, Petite-Rivière-St-François, Baie-St-Paul, Cape Tourmente and along the North Channel near Ile d'Orléans. A gross estimate gives a shore zone area of approximately 60 km^2. Consequently, several million tonnes of sediment will also be caught up by this large ice cover, from which about 40% will drift seaward at break-up. Thus, an average sediment load of $5-6 \times 10^6$ t of fine-grained sediment is available from the ice cover of shore zones in the Middle St. Lawrence Estuary. To this load should be added the few million tonnes of sediment which come from the freezing in situ of the turbid waters in the offshore zone. This load possibly adds up to 4×10^6 t annually (wet weight). In summary, it is likely that an average of 10×10^6 t of fine sediment would drift outside the turbidity zone annually in the way which has been described.

The sediment budget

The large volume of sediment involved in ice processes is of great importance for understanding the sediment budget of the Middle St. Lawrence Estuary. The problem can be briefly summarized as follows. According to most authors the annual input for the high turbidity zone greatly exceeds the output. Consequently, there should be a positive balance reflected by deposition of fines. Surprisingly, there is little permanent sedimentation today in the Middle St. Lawrence Estuary both in the offshore and shore zones. D'Anglejan and Brisebois (1978) have clearly shown that very little deposition of fines occurs presently in the basins and channels of the middle estuary with bottom erosion occurring almost everywhere (Fig.12). This statement is also valid for the shore zone; on a long-term basis, almost everywhere

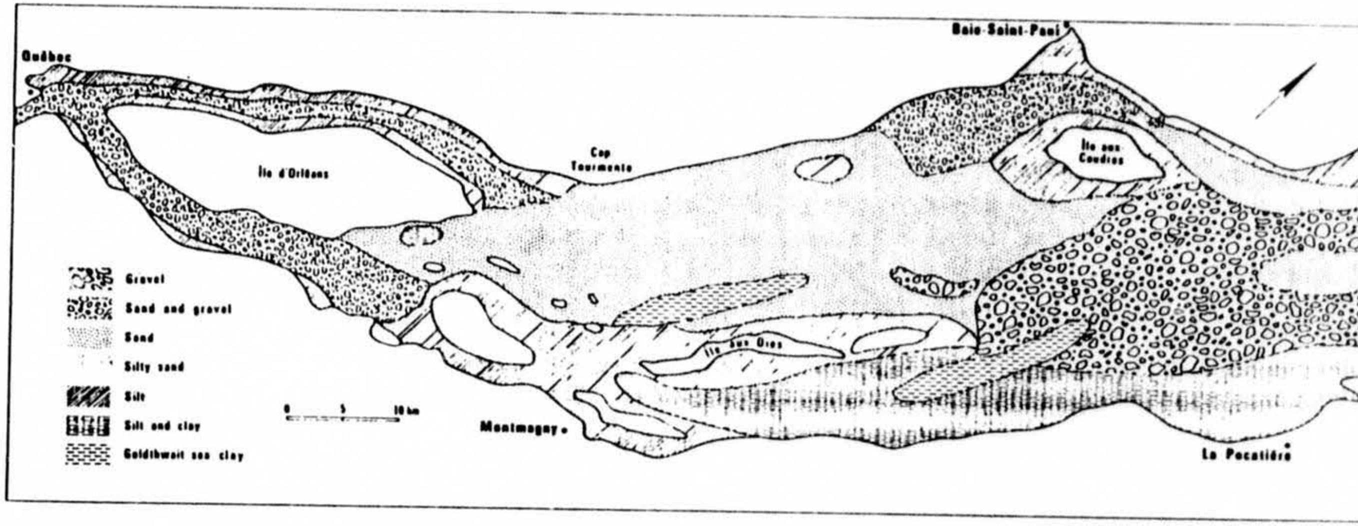


Fig.12. Distribution of surficial sediments in the upper section of the Middle St. Lawrence Estuary, according to Brisebois (1975), D'Anglejan and Brisebois (1978), and Cremer (1979).

very little sedimentation is observed on both shores of the St. Lawrence Estuary (Dionne, 1979; Allard, 1981; D'Anglejan et al., 1981; Dubé, 1982), although a relatively important deposition of a temporary nature (summer and winter) occurs at some localities: Cape Tourmente, Ile aux Oies and Montmagny for example (Dionne, 1980, 1981b, Sérodes, 1980; Troude et al., 1981). The deposition occurring in the navigation channel is mainly related to bedload and current action (Boucher, 1961). Consequently, it does not reduce significantly the volume of the suspended load.

Data are still inadequate to determine precisely the sedimentary budget of the middle estuary. The output of suspended matter has been evaluated from measurements made in summer only. D'Anglejan et al. (1973, 1974) calculated that about 1×10^6 t of fines escape the turbidity zone during the ice-free season. The output is largely controlled by the complex water circulation resulting from hydrodynamic processes, mainly tidal action. These authors underlined the possible action of ice but did not discuss it nor did they suggest any estimates of the volume of fines possibly involved in ice drifting. Although B. D'Anglejan (pers. commun., 1982) agreed that this preliminary estimate is much too low, no other figure for the output of the suspended load from the turbidity zone has been suggested yet.

The input is also poorly documented. Four estimates are commonly referred to: (a) 5×10^6 t yr^{-1} (Frenette and Larinier, 1973; Loring and Nota, 1973; CENTREAU, 1975); (b) 8–10 $\times 10^6$ t, from which 70% is introduced during April and May (Sérodes, 1980); (c) 11×10^6 t (Cataliotti-Valdina and Long, 1982); and (d) 20×10^6 t (Cremer, 1979). It is difficult to determine which one of these estimates is the most realistic.

Considering the estimated output related to ice drifting, an input of only 5×10^6 t is much too low. This would give a sediment balance of only 4×10^6 t of fines for deposition in the various zones of the estuary and for ice drifting. In this case, severe erosion would certainly result. Although erosion does occur on the bottom and along shorelines, it is not considered nearly as important as that which would result from a very negative sediment budget (i.e. approximately 5–6 $\times 10^6$ t yr^{-1}). The largest figure for the input load (at Quebec City) suggests 20×10^6 t yr^{-1} (Cremer, 1979). If this figure is correct, it would mean that the sediment budget is significantly positive, since the summer and the winter output together are possibly less than 12×10^6 t. In this case, relatively important long-term deposition should occur at least at some localities. However, there is no evidence of this. Another possibility is that output explained by ice-drifting is indeed more important than has been suggested.

The 8–10 and the 11×10^6 t figures for the annual input suggested respectively by Sérodes (1980), and Cataliotti-Valdina and Long (1982) fit better with the preliminary values obtained when ice drifting processes are taken into account. In this case, the annual output almost equals the input. Consequently the sediment budget is in near equilibrium, although it may be slightly positive or negative from year to year. This would explain why there is little or no fine sedimentation over long-term periods in the offshore and shore zones of the Middle St. Lawrence Estuary.

According to Sérodes (1980), most of the 3×10^6 t of fines deposited on tidal flats during the summer return to the turbidity zone in the autumn. Consequently most tidal flats are not really prograding today; on the contrary, many are suffering erosion.

Whatever the suspended matter values for the annual input are, if the ice-free season output does not exceed 1×10^6 t (D'Anglejan et al., 1973), there should be a positive sediment budget in the Middle St. Lawrence Estuary. Since this positive balance has not been observed in the nearshore and offshore zones, it is suggested that ice drifting is the process by which several million tonnes of fine-grained sediments are evacuated annually from the turbidity zone. Thus, ice processes are playing a significant role in the evolution of some ice-dominated shelf environments. Considering that the concentration of particulate matter in the Gulf of St. Lawrence is low (D'Anglejan, 1969; Couillard, 1982), only a small percentage of the ice-drifting sediments from the Middle St. Lawrence Estuary reaches the deep ocean (Sundby, 1974).

CONCLUSIONS

The role of ice in evacuating fine-grained sediments from the high turbidity zone of the St. Lawrence Estuary would appear to be of great significance. It offers a valid explanation for the very limited deposition which characterizes most offshore and shores zones. Because of ice action, the sediment budget in the Middle St. Lawrence Estuary is presently more or less in equilibrium (or perhaps with a slightly negative balance) over long term periods. Judging from the thickness of the recent deposits overlying the postglacial marine clays, this situation has existed for at least several centuries. Consequently, some shore zones are prograding very slowly, others are in equilibrium and some are degrading. Serious damage to the environment may result from such local degradations if adequate solutions are not proposed in the near future. High-latitude inner continental shelves are commonly ice-dominated environments. For various reasons, ice processes are often poorly documented although they are of great importance in areas such as the St. Lawrence Estuary.

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HABITAT SUITABILITY INFORMATION: RAINBOW TROUT

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PREFACE

The Habitat Suitability Index (HSI) models presented in this publication aid in identifying important habitat variables. Facts, ideas, and concepts obtained from the research literature and expert reviews are synthesized and presented in a format that can be used for impact assessment. The models are hypotheses of species-habitat relationships, and model users should recognize that the degree of veracity of the HSI model, SI graphs, and assumptions will vary according to geographical area and the extent of the data base for individual variables. After clear study objectives have been set, the HSI model building techniques presented in U.S. Fish and Wildlife Service (1981)[1] and the general guidelines for modifying HSI models and estimating model variables presented in Terrell et al. (1982)[2] may be useful for simplifying and applying the models to specific impact assessment problems. Simplified models should be tested with independent data sets if possible.

A brief discussion of the appropriateness of using selected Suitability Index (SI) curves from HSI models as a component of the Instream Flow Incremental Methodology (IFIM) is provided. Additional SI curves, developed specifically for analysis of rainbow trout habitat with IFIM, also are presented.

The U.S. Fish and Wildlife Service encourages model users to provide comments, suggestions, and test results that may help us increase the utility and effectiveness of this habitat-based approach to impact assessment. Please send comments to:

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[1]U.S. Fish and Wildlife Service. 1981. Standards for the development of habitat suitability index models. 103 ESM. U.S. Fish Wildl. Serv., Div. Ecol. Serv. n.p.

[2]Terrell, J. W., T. E. McMahon, P. D. Inskip, R. F. Raleigh, and K. L. Williamson. 1982. Habitat suitability index models: Appendix A. Guidelines for riverine and lacustrine applications of fish HSI models with the Habitat Evaluation Procedures. U.S. Fish Wildl. Serv. FWS/OBS-82/10.A. 54 pp.

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RAINBOW TROUT (Salmo gairdneri)

HABITAT USE INFORMATION

General

Because of variations in their life history pattern and the habitat in which they spend the majority of their adult lives, rainbow trout (Salmo gairdneri) can be subdivided into three basic ecological forms: (1) anadromous steelhead trout; (2) resident stream rainbow trout; and (3) lake or reservoir dwelling rainbow trout. It is important to recognize that there is a genetic or hereditary basis for each ecological form. For example, a "lake or reservoir" rainbow may react very differently to environmental stimuli associated with survival, feeding, and growth if it belongs to a population that has been evolving and adapting to the particular lake for hundreds or thousands of years, when compared to hatchery rainbow trout that have just been released in the lake.

Nonanadromous rainbow trout are native to the Pacific Coast drainages inland as far as the Rockies and from the Rio del Presidio River in Mexico to the Kuskokwim River in Southwestern Alaska (Behnke 1979). They are also native to the Peace River drainage of British Columbia and the headwaters of the Athabaska River (of the McKenzie River basin) in Alberta (MacCrimmon 1971). Their present range extends from the Arctic Circle to 55° S latitude. They are perhaps the most widely introduced fish species; the only continent lacking rainbow trout is Antarctica (McAfee 1966; MacCrimmon 1971). Rainbow trout occur from 0 to 4,500 m above sea level (MacCrimmon 1971).

Anadromous steelhead trout are distributed along the Pacific coast from the Santa Ynez Mountains, California, to the Alaska Peninsula (Jordan and Evermann 1902; Withler 1966). Large rainbow trout on and north of the Alaska Peninsula appear to be nonanadromous.

Age, Growth, and Food

Female rainbow trout typically become sexually mature during their third year; males become sexually mature during their second or third year (Holton 1953; Lagler 1956; McAfee 1966). Life expectancy averages 3 to 5 years in most southern lake populations, but life expectancy of steelhead and northern lake populations appears to be 4 to 8 years. Maximum size also varies with population, area, and habitat. Steelhead may grow to 122 cm long and weigh 16 kg. The average angler's catch is 3.6 to 4 kg. Great Lakes rainbow grow to 244 cm, but seldom exceed 9 kg (Scott and Crossman 1973). Size in wild rainbow trout appears to be a function of longevity, delayed age at maturity, and length of ocean residence for steelhead.

Adult and juvenile rainbow trout are basically opportunistic feeders and consume a wide variety of foods. Availability of different foods depends on many factors, including water type, season, and size of the trout (McAfee 1966). The diet of rainbow trout consists mainly of aquatic insects (Allen 1969; Carlander 1969; Baxter and Simon 1970; Scott and Crossman 1973), although foods, such as zooplankton (McAfee 1966), terrestrial insects, and fish (Carlander 1969), are locally or seasonally important. The relative importance of aquatic and terrestrial insects to resident stream rainbow trout varies greatly among different environments, seasonally and dielly, and with the age of the trout (Bisson 1978). Forty to fifty percent or more of the summer food of trout in headwater streams may be composed of terrestrial insects (Hunt 1971). Adult stream rainbow trout occasionally consume significant quantities of vegetation, mostly algae (McAfee 1966). Stream trout have no mechanism to break down cell walls in vegetation and cannot obtain nutrients from it, therefore, vegetation is thought to be consumed because of the invertebrates attached to it (Behnke pers. comm.). Bottom fauna may comprise 83 to 94% of the winter diet of adult and juvenile lake rainbow trout (Crossman and Larkin 1959). Lake trout usually reach 30 cm in length before they actively prey on other fish species (Crossman 1959; Crossman and Larkin 1959; Johannes and Larkin 1961).

Reproduction

Rainbow trout spawn almost exclusively in streams. Some rainbow and rainbow-cutthroat trout hybrids have successfully reproduced in lakes without tributary streams (Behnke, pers. comm.). Spawning in certain river systems may occur in intermittent tributary streams (Everest 1973; Price and Geary 1979). In one case, up to 47% of the stream rainbow trout population spawned in intermittent tributaries that dried up in midsummer and fall (Erman and Leidy 1975; Erman and Hawthorne 1976). Spawning normally occurs from January to July, depending on location. Hatchery selection has resulted in fall spawning strains, and spawning of hatchery fish may occur in almost any month of the year, depending on the strain (Behnke 1979). A few populations outside of the native range have modified their spawning times to avoid adverse environmental conditions (Van Velson 1974; Kaya 1977). Viable eggs have resulted from December and January spawning at water temperatures of 0.3 to 2.0° C in a tributary of Lake Huron (Dodge and MacCrimmon 1970). However, eggs exposed to long periods of 0 to 4° C temperatures suffered high mortality and abnormalities.

The female generally selects a redd site in gravel substrate at the head of a riffle or downstream edge of a pool (Greeley 1932; Orcutt et al. 1968). The redd pit, constructed primarily by the female, is typically longer than the female and deeper than her greatest body depth (Greeley 1932). Average depth of egg deposition is 15 cm (Hooper 1973).

Rainbow trout residing in lakes and reservoirs have a similar life history pattern to the steelhead trout, but generally lack a physiological smolt stage. Juveniles migrate from natal streams to a freshwater lake rearing area, instead of to the ocean. Lake rainbow trout most commonly spend two summers in a stream and two summers in a lake before maturing (Greeley 1933). Spawning takes place during the growing season in an inlet or an outlet stream, with more than 90% of the trout returning to the stream of natal origin (Greeley 1933; Lindsey et al. 1959). Lakes with no inlet or outlet streams generally do not possess a reproducing population of rainbow trout. Whether spawning adults enter through an inlet or an outlet, they and their progeny will return to the lake (Lindsey et al. 1959). These movements from natal sites to lake rearing areas appear to be directed by genetic/environmental interactions (Raleigh 1971).

Spawning usually begins one month earlier in the outlet than in the inlet (Lindsey et al. 1959; Hartman et al. 1962); the difference in time is apparently related to temperature differences (Lindsey et al. 1959). In Bothwell Creek, a tributary of Lake Huron, 65% of the spawning run were repeat spawners (Dodge and MacCrimmon 1970). The typical survival rate of repeat spawners is 10-30%, with extremes from 1% to more than 65%.

Average fecundity of rainbow trout is related to length, but is highly variable, ranging from 500 to 3,161 eggs per stream resident female (Carlander 1969). Fecundity of lake resident females ranges from 935 to 4,578 eggs per female, with an average of 2,028 eggs per female (Mann 1969).

Anadromy

Anadromous steelhead spawn in freshwater streams. Steelhead smolt and migrate in late spring (Wagner 1968; Chrisp and Bjornn 1978). Photoperiod appears to be the dominant triggering mechanism for parr-smolt transformation, with temperature affecting the rate of transformation (Wagner 1974). Smolts that have not migrated by approximately the summer solstice revert to parr and attempt to migrate the following season (Zaugg and Wagner 1973). Juveniles reside in freshwater for 1 to 4 years before migrating to the sea as smolts. They mature after 1 to 4 years of ocean residence and return to freshwater rivers to spawn (Chapman 1958; Withler 1966). A large number of the steelhead adults die after spawning, but some (3 to 53%) return to the ocean and spawn again (Bjornn 1960; Withler 1966; Fulton 1970). Steelhead spawners tend to be larger and older in the northern portion of their range (Withler 1966).

There are both winter and summer-run steelhead. Summer-run adults enter freshwater rivers in the spring and early summer. Winter-run steelhead enter freshwater rivers in the fall and winter. As many as 98.8% of the trout return to their natal stream (McAfee 1966). Both groups typically spawn in the spring and early summer months, March through early July (Withler 1966; Orcutt et al. 1968), although spawning at other times of the year has been reported. Summer-run and winter-run steelhead are distinguished by differences in behavior prior to spawning and, to a limited extent, by appearance (Withler 1966).

When fish migrate from freshwater to saltwater, they are moving from a hypotonic medium to a hypertonic medium. Gill Na-K ATPase activity appears to be related to saltwater tolerance and smolting (Conte and Wagner 1965; Zaugg and Wagner 1973; Adams et al. 1975). Water temperature affects Na-K-related ATPase activity. Juvenile steelhead kept in water warmer than 13° C from March to June experienced reduced levels of smoltification and very low levels of ATPase activity (Zaugg and McLain 1972; Zaugg and Wagner 1973; Wagner 1974). Water temperatures of 10.5 to 13° C resulted in a moderate ATPase response, and temperatures of 6.5 to 10° C resulted in the highest activity levels for the longest period of time (Adams et al. 1975). The effect of temperature on ATPase activity is reversible within a season (Zaugg and McLain 1972).

Coefficient of condition is another indicator of parr-smolt transformation. Juvenile steelhead not undergoing a smolt transformation do not lose weight; whereas, steelhead undergoing transformation lose enough weight to result in a greatly reduced coefficient of condition (Adams et al. 1973; Wagner 1974).

A fork length (i.e., anterior most extremity to the notch in the tail fin of fork-tailed fish or to the center of the tail fin when the tail is not forked) of 160 mm is the average length juvenile parr must reach before they undergo the physiological and morphological changes of smolting (Fessler and Wagner 1969; Chrisp and Bjornn 1978). Hatchery-reared steelhead typically reach critical size in one growing season, but native stream steelhead usually require two or more growing seasons (Chrisp and Bjornn 1978). Migrating smolts at the lower end of the minimum length requirement stay in the ocean longer than smolts that are larger in size when they migrate (Chapman 1958).

The freshwater habitat requirements of adult and juvenile steelhead are assumed to be essentially the same as those for other rainbow trout. Exceptions for steelhead are: (1) low temperature (< 13° C) requirements during the spring months for smoltification of juveniles; and (2) the presence of moderate temperatures (preferably ≤ 20° C) and freshets (periodic high flows) during the upstream migration of adults.

Specific Habitat Requirements

Optimal rainbow trout riverine habitat is characterized by clear, cold water; a silt-free rocky substrate in riffle-run areas; an approximately 1:1 pool-to-riffle ratio, with areas of slow, deep water; well-vegetated stream banks; abundant instream cover; and relatively stable water flow, temperature regimes, and stream banks (Raleigh and Duff 1980).

Optimal lacustrine habitat is characterized by clear, cold, deep lakes that are typically oligotrophic, but may vary in size and chemical quality, particularly in reservoir habitats. Rainbow trout are primarily stream spawners and generally require tributary streams with gravel substrate in riffle areas for reproduction to occur.

Trout production is typically greatest in streams with a pool-to-riffle ratio of approximately 1:1 (Fortune and Thompson 1969; Thompson and Fortune 1970). Pools are inhabited throughout the year by adult and juvenile stream rainbow trout. Pools are important to trout as a refuge from adverse conditions during the winter. Because pools differ in their ability to provide resting areas and cover, this model subdivides pools into three classes. Lewis (1969) found that streams with deep, low velocity pools containing extensive cover had the most stable trout populations.

Available trout literature does not often clearly distinguish between feeding stations, escape cover, and winter cover requirements. Prime requisites for optimal feeding stations appear to be low water velocity and access to a plentiful food supply; i.e., energy accretion at a low energy cost. Water depth is not clearly defined as a selection factor, and overhead cover is preferred but not essential. Escape cover, however, must be nearby. The feeding stations of dominant adult trout include overhead cover when available. The feeding stations of subdominant adults and juveniles, however, do not always include overhead cover. Antagonistic behavior occurs at feeding stations and hierarchies are established, but escape cover is often shared.

Cover is recognized as one of the essential components of trout streams. Boussu (1954) was able to increase the number and weight of trout in stream sections by adding artificial brush cover and to decrease numbers and weight of trout by removing brush cover and undercut banks. Lewis (1969) reported that the amount of cover was important in determining the number of trout in sections of a Montana stream. Stewart (1970) found that mean depth and underwater, overhanging bank cover were the most important variables in determining the density of brook and rainbow trout longer than 18 cm in a northcentral Colorado stream. Cover for adult trout consists of areas of obscured stream bottom in water ≥ 15 cm deep with a velocity of ≤ 15 cm/sec (Wesche 1980). Wesche (1980) reported that, in larger streams, the abundance of trout ≥ 15 cm in length increased with water depth; most trout were at depths of at least 15 cm. Cover is provided by overhanging vegetation; submerged vegetation; undercut banks; instream objects, such as debris piles, logs, and large rocks; pool depth; and surface turbulence (Giger 1973). A cover area of ≥ 25% of the total stream area provides adequate cover for adult trout; a cover area of ≥ 15% is adequate for juveniles. The main uses of summer cover are probably predator avoidance and resting.

In some streams, the major factor limiting salmonid densities may be the amount of adequate overwintering habitat, rather than the amount of summer rearing habitat (Bustard and Narver 1975a). Winter hiding behavior in salmonids is triggered by low temperatures (Chapman and Bjornn 1969; Everest 1969; Bustard and Narver 1975a,b). Cutthroat trout were found under boulders, log jams, upturned roots, and debris when temperatures neared 4 to 8° C, depending on the water velocity (Bustard and Narver 1975a). Everest (1969) found juvenile rainbow trout 15 to 30 cm deep in the substrate, which was often covered by 5 to 10 cm of anchor ice. Lewis (1969) reported that, during winter, adult rainbow trout tended to move into deeper water (first class pools). Bjornn (1971) indicated that downstream movement during or preceding winter did not occur if sufficient winter cover was available locally. Trout move to winter cover to avoid physical damage from ice scouring (Hartman 1965; Chapman and Bjornn 1969) and to conserve energy (Chapman and Bjornn 1969; Everest 1969).

Headwater trout streams are relatively unproductive. Most energy inputs to the stream are in the form of allochthonous materials, such as terrestrial vegetation and terrestrial insects (Idyll 1942; Chapman 1966; Hunt 1971). Aquatic invertebrates are most abundant and diverse in riffle areas with rubble substrate and on submerged aquatic vegetation (Hynes 1970). However, optimal substrate for maintenance of a diverse invertebrate population consists of a mosaic of mud, gravel, rubble, and boulders, with rubble dominant. A pool-to-riffle ratio of about 1:1 (approximately a 40 to 60% pool area) appears to provide an optimal mix of food producing and rearing areas for trout (Needham 1940). In riffle areas, the presence of fines (> 10%) reduces the production of invertebrate fauna (based on Cordone and Kelly 1961; Crouse et al. 1981).

Canopy cover is important in maintaining shade for stream temperature control and in providing allochthonous materials to the stream. Too much shade, however, can restrict primary productivity in a stream. Stream temperatures can be increased or decreased by controlling the amount of shade. About 50 to 75% midday shade appears optimal for most small trout streams (adapted from Oregon/Washington Interagency Wildlife Conference 1979). Shading becomes less important as stream gradient and size increase. In addition, a well vegetated riparian area helps control watershed erosion. In most cases, a buffer strip about 30 m wide, 80% of which is either well vegetated or has stable rocky stream banks, provides adequate erosion control and maintains undercut stream banks characteristic of good trout habitat. The presence of fines in riffle-run areas can adversely affect embryo survival, food production, and cover for juveniles.

There is a definite relationship between the annual flow regime and the quality of trout habitat. The most critical period is typically during base flow (lowest flows of late summer to winter). A base flow ≥ 50% of the average annual daily flow is considered excellent for maintaining quality trout habitat, a base flow of 25 to 50% is considered fair, and a base flow of < 25% is considered poor (adapted from Binns and Eiserman 1979; Wesche 1980).

<u>Adult</u>. Dissolved oxygen requirements vary with species, age, prior acclimation temperature, water velocity, activity level, and concentration of substances in the water (McKee and Wolf 1963). As temperature increases, the dissolved oxygen saturation level in the water decreases, while the dissolved oxygen requirement for the fish increases. As a result, an increase in temperature resulting in a decrease in dissolved oxygen can be detrimental to the fish. Optimal oxygen levels for rainbow trout are not well documented, but appear to be ≥ 7 mg/l at temperatures ≤ 15° C and ≥ 9 mg/l at temperatures > 15° C. Doudoroff and Shumway (1970) demonstrated that swimming speed and growth rates for salmonids declined with decreasing dissolved oxygen levels. In the summer (≥ 10° C), cutthroat trout generally avoid water with dissolved oxygen levels of less than 5 mg/l (Trojnar 1972; Sekulich 1974).

The incipient lethal level of dissolved oxygen for adult and juvenile rainbow trout is approximately 3 mg/l or less, depending on environmental conditions, especially temperature (Gutsell 1929; Burdick et al. 1954; Alabaster et al. 1957; Downing and Merken 1957; Doudoroff and Warren 1962). Although fish can survive at concentrations just above this level, they must make various physiological adaptations to low levels of dissolved oxygen that may jeopardize their health (Randall and Smith 1967; Kutty 1968; Hughes and Saunders 1970; Cameron 1971; Holeton 1971). For example, low levels of dissolved oxygen can result in reduced fecundity and even prevent spawning. Large fluctuations in dissolved oxygen may cause a reduction in food consumption and impaired growth (Doudoroff and Shumway 1970).

The upper and lower incipient lethal temperatures for adult rainbow are 25° and 0° C, respectively (Black 1953; Lagler 1956; McAfee 1966; Bidgood and Berst 1969; Hokanson et al. 1977). Zero growth rate occurred at 23° C for rainbow trout in the laboratory (Hokanson et al. 1977). Changes in the natural growth rate of rainbow trout are detrimental to their development and survival. Therefore, 25° C should be considered the upper limit suitable for rainbow trout and then only for short periods of time. Adult lake rainbow trout select waters with temperatures between 7 to 18° C (Fast 1973; May 1973) and avoid permanent residence where temperatures are above 18° C (May 1973). Adult stream rainbow trout select temperatures between 12.0 and 19.3° C (Garside and Tait 1958; Bell 1973; Cherry et al. 1977; McCauley et al. 1977). Dickson and Kramer (1971) reported that the greatest scope of rainbow trout activity occurred at 15 and 20° C when tested at 5° C temperature intervals. Stream rainbow trout select temperatures between 12 and 19° C; lake resident trout avoid temperatures > 18° C. Therefore, the optimal temperature range for rainbow trout is assumed to be 12 to 18° C.

The depth distribution of adult lake rainbow trout is usually a function of dissolved oxygen, temperature, and food. Adult lake rainbow trout remain at depths ≤ the 18° C isotherm and at dissolved oxygen levels > 3 mg/l (May 1973; Hess 1974).

Focal point velocities for adult cutthroat trout at territorial stations in Idaho streams were primarily between 10 and 14 cm/sec, with a maximum of 22 cm/sec (Griffith 1972). The focal point velocities for adult rainbow trout are assumed to be similar.

Precise pH tolerance and optimal ranges are not well documented for rainbow trout. Most trout populations can probably tolerate a pH range of 5.5 to 9.0, with an optimal range of 6.5 to 8.0 (Hartman and Gill 1968; Behnke and Zarn 1976).

Withler (1966) suggested that the correlation between the winter steelhead run and increased water volume indicates that a freshet condition is required to initiate upstream movement of spawners. Everest (1973) stated that speed of migration of summer-run steelhead in the Rogue River was inversely related to temperature and directly related to streamflows. Hanel (1971) observed that steelhead migration into the Iron Gate fish hatchery ceased when the water temperature dropped to 4° C and did not resume for several weeks until the temperature increased. This suggests that water temperatures should be > 4° C but ≤ 18° C, and streamflow conditions should be above normal seasonal flows during upstream migrations of steelhead adults.

Embryo. Incubation time varies inversely with temperature. Eggs usually hatch within 28 to 40 days (Cope 1957), but may take as long as 49 days (Scott and Crossman 1973). The optimal temperature for embryo incubation is about 7 to 12° C. Calhoun (1966) reported increased mortalities of rainbow embryos at temperatures < 7° C and normal development at temperatures ≥ 7 but ≤ 12° C. The optimal water velocity above rainbow trout redds is between 30 and 70 cm/sec. Velocities less than 10 cm/sec or greater than 90 cm/sec are unsuitable (Delisle and Eliason 1961; Thompson 1972; Hooper 1973).

The combined effects of temperature, dissolved oxygen, water velocity, and gravel permeability are important for successful incubation (Coble 1961). In a 30% sand and 70% gravel mixture, only 28% of implanted steelhead embryos hatched; of the 28% that hatched, only 74% emerged (Bjornn 1969; Phillips et al. 1975). Optimal spawning gravel conditions are assumed to include ≤ 5% fines; ≥ 30% fines are assumed to result in low survival of embryos and emerging yolk-sac fry. Suitable incubation substrate is gravel that is 0.3 to 10.0 cm in diameter (Delisle and Eliason 1961; Orcutt et al. 1968; Hooper 1973; Duff 1980). Optimal substrate size depends on the size of the spawners, but is assumed to average 1.5 to 6.0 cm in diameter for rainbows < 50 cm long and 1.5 to 10.0 cm in diameter for spawners ≥ 50 cm long (Orcutt et al. 1968). Doudoroff and Shumway (1970) reported that salmonids that incubated at low dissolved oxygen levels were weak and small with slower development and more abnormalities. Dissolved oxygen requirements for rainbow trout embryos are not well documented, but are assumed to be similar to the requirements for adults.

Fry. Rainbow trout remain in the gravel for about 2 weeks after hatching (Scott and Crossman 1973) and emerge 45 to 75 days after egg fertilization, depending on water temperature (Calhoun 1944; Lea 1968). When moving from natal gravels to rearing areas, rainbow trout fry exhibit what appears to be three distinct genetically controlled movement patterns: (1) movement downstream to a larger river, lake, or to the ocean; (2) movement upstream from an outlet river to a lake; or (3) local dispersion within a common spawning and rearing area to areas of low velocity and cover (Raleigh and Chapman 1971). Fry of lake resident fish may either move into the lake from natal streams during the first growing season or overwinter in the spawning stream and move into the lake during subsequent growing seasons.

Fry residing in streams prefer shallower water and slower velocities than do other life stages of stream trout (Miller 1957; Horner and Bjornn 1976). Fry utilize velocities less than 30 cm/sec, but velocities less than 8 cm/sec are preferred (Griffith 1972; Horner and Bjornn 1976). Fry survival decreases with increased velocity after the optimal velocity has been reached (Bulkley and Benson 1962; Drummond and McKinney 1965). A pool area of 40% to 60% of the total stream area is assumed to provide optimal fry habitat. Cover in the form of aquatic vegetation, debris piles, and the interstices between rocks is critical. Griffith (1972) states that younger trout live in shallower water and stay closer to escape cover than do older trout. Few fry are found more than 1 m from cover. As the young trout grow, they move to deeper, faster water. Everest (1969) suggested that one reason for this movement was the need for cover, which is provided by increased water depth, surface turbulence, and substrate that consists of large material.

Stream resident trout fry usually overwinter in shallow areas of low velocity near the stream margin, with rubble being the principal cover (Bustard and Narver 1975a). Optimal size of substrate used as winter cover by rainbow fry and small juveniles ranges from 10 to 40 cm in diameter (Hartman 1965; Everest 1969). An area of substrate of this size class that is ≥ 10% of the total habitat probably provides adequate cover for rainbow fry and small juveniles. The use of small diameter rocks (gravel) for winter cover may result in increased mortality due to greater shifting of the substrate (Bustard and Narver 1975a). The presence of fines (≥ 10%) in the riffle-run areas reduces the value of the area as cover for fry and small juveniles. Mantelman (1958) reported a preferred temperature range of 13 to 19° C for fry. Because fry occupy habitats contiguous with adults, their temperature and oxygen requirements are assumed to be similar to those of adults.

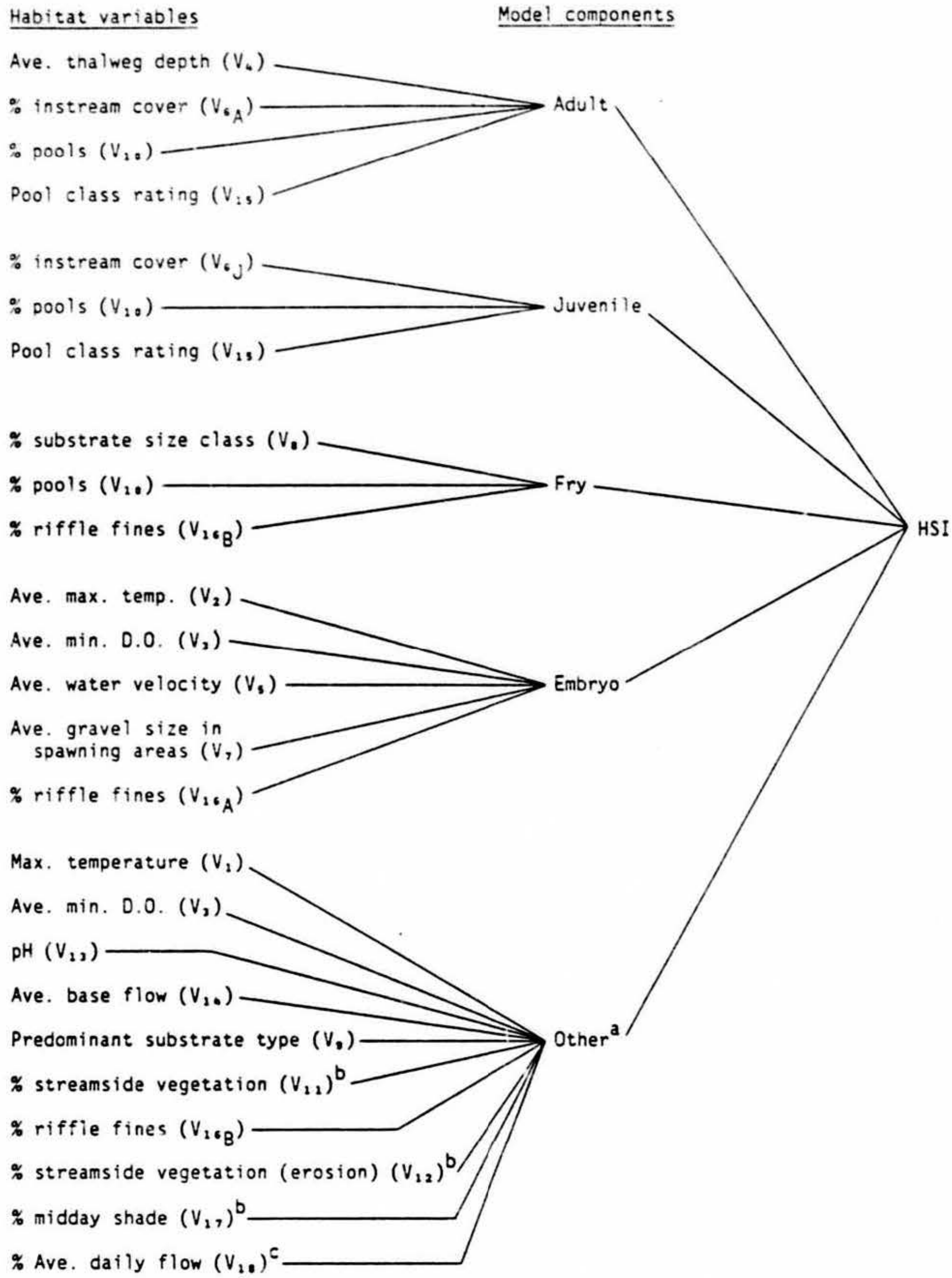
Juvenile. Griffith (1972) reported focal point velocities for juvenile cutthroat in Idaho of between 10 and 12 cm/sec, with a maximum velocity of 22 cm/sec. Metabolic rates are highest between 11 and 21° C, with an apparent optimal temperature of between 15 and 20° C (Dickson and Kramer 1971). In steelhead streams, temperatures should be < 13 but > 4° C (optimal 7 to 10° C) from March until June for normal smoltification to occur (Wagner 1974; Adams et al. 1975).

Common types of cover for juvenile trout are upturned roots, logs, debris piles, overhanging banks, riffles, and small boulders (Bustard and Narver 1975a). Young salmonids occupy different habitats in winter than in summer, with log jams and rubble important as winter cover. Wesche (1980) observed that larger cutthroat trout (> 15 cm long) and juveniles (≤ 15 cm) tended to use instream substrate cover more often than they used streamside cover (undercut banks and overhanging vegetation). However, juvenile brown trout preferred streamside cover. An area of cover ≥ 15% of the total habitat area appears to provide adequate cover for juvenile trout.

Because juvenile rainbow trout occupy habitats contiguous with adults, their temperature and oxygen requirements are assumed to be similar.

HABITAT SUITABILITY INDEX (HSI) MODELS

Figure 1 illustrates the assumed relationships among model variables, components, and the HSI for the rainbow trout model.



[a]Variables that affect all life stages.

[b]Optional variables.

[c]Steelhead variable.

Figure 1. Diagram illustrating the relationship among model variables, components, and HSI.

Model Applicability

Geographic area. The following models are applicable over the entire North American range of the rainbow trout.

Season. The model rates the year-round freshwater habitat of rainbow trout.

Cover types. The model is applicable to freshwater riverine or lacustrine habitats.

Minimum habitat area. Minimum habitat area is the minimum area of continuous habitat that is required for a species to live and reproduce. Because trout can move considerable distances to spawn or locate suitable summer or winter rearing habitat, no attempt has been made to define a minimum amount of habitat for the species.

Verification level. An acceptable level of performance for this rainbow trout model is for it to produce an index between 0 and 1 that the authors and other biologists familiar with rainbow trout ecology believe is positively correlated with long term carrying capacity of the habitat. Model verification consisted of testing the model outputs from sample data sets developed by the authors to simulate rainbow trout habitat of high, medium, and low quality and model review by rainbow trout experts.

Model Description

The HSI model consists of five components: Adult (C_A); Juvenile (C_J); Fry (C_F); Embryo (C_E); and Other (C_O). Each life stage component contains variables specifically related to that component. The component C_O contains variables related to water quality and food supply that affect all life stages of rainbow trout.

The model utilizes a modified limiting factor procedure. This procedure assumes that model variables and components with suitability indices in the average to good range, > 0.4 but < 1.0, can be compensated for by higher suitability indices of other related model variables and components. However, variables and components with suitabilities ≤ 0.4 cannot be compensated for and, thus, become limiting factors for the habitat suitability.

Adult component. Variable V_6, percent instream cover, is included because standing crops of adult trout are assumed to be related to the amount of cover available based on studies of brook and cutthroat trout. Percent pools (V_{10}) is included because pools provide cover and resting areas for adult trout. Variable V_{10} also quantifies the amount of pool habitat that is needed. Variable V_{15}, pool class rating, is included because pools differ in the amount and quality of escape cover, winter cover, and resting areas that they provide. Average thalweg depth (V_4) is included because average water depth affects the amount and quality of pools and instream cover available to adult trout and the migratory access to spawning and rearing areas.

Juvenile component. Variables V_6, percent instream cover; V_{10}, percent pools; and V_{15}, pool class rating are included in the juvenile component for the same reasons listed above for the adult component. Juvenile rainbow trout use these essential stream features for escape cover, winter cover, and resting areas.

Fry component. Variable V_8, percent substrate size class, is included because trout fry utilize substrate as escape cover and winter cover. Variable V_{10}, percent pools, is included because fry use the shallow, slow water areas of pools and backwaters as resting and feeding stations. Variable V_{16}, percent riffle fines, is included because the percent fines affects the ability of the fry to utilize the rubble substrate for cover.

Embryo component. It is assumed that habitat suitability for trout embryos depends primarily on average maximum water temperature, V_2; average minimum dissolved oxygen, V_3; average water velocity, V_5; gravel size in spawning areas, V_7; and percent riffle fines, V_{16}. Water velocity (V_5), gravel size (V_7), and percent fines (V_{16}) are interrelated factors that effect the transport of dissolved oxygen to the embryo and the removal of metabolic waste products from the embryo. In addition, the presence of too many fines in the redds blocks movement of the fry from the incubating gravels to the stream.

Other component. This component contains model variables for two subcomponents, water quality and food supply, that affect all life stages. The subcomponent water quality contains four variables: maximum temperature (V_1); average minimum dissolved oxygen (V_3); pH (V_{13}); and average base flow (V_{14}). The waterflow of all streams fluctuates on a seasonal cycle, and a correlation exists between the average annual daily streamflow and the annual low base flow period. Average base flow (V_{14}) is included to quantify the relationship between annual water flow fluctuations and trout habitat suitability. These four variables affect the growth and survival of all life stages except embryos, whose water quality requirements are included with the embryo component. The subcomponent food supply contains three variables: dominant substrate type (V_9); percent streamside vegetation (V_{11}); and percent riffle fines (V_{16}). Predominant substrate type (V_9) is included because the abundance of aquatic insects, an important food item for rainbow trout, is correlated with substrate type. Variable V_{16}, percent fines in riffle-run and spawning areas, is included because the presence of excessive fines in riffle-run areas reduces the production of aquatic insects. Variable V_{11} is included because allochthonous materials are an important source of nutrients in cold, unproductive trout streams.

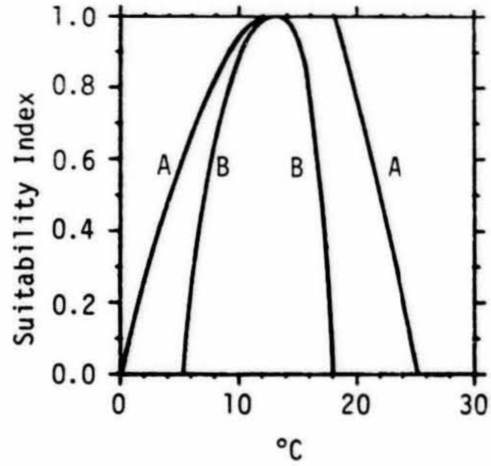
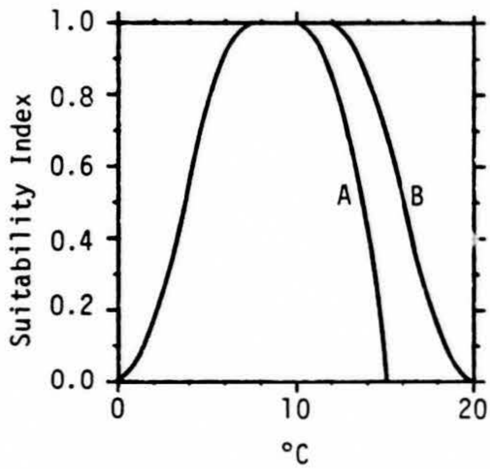
Variables V_{12}, V_{17}, and V_{18} are optional variables to be used only when needed and appropriate. Streamside vegetation, V_{12}, is an important means of controlling soil erosion, a major source of fines in streams, and for input of terrestrial insects. Variable V_{17}, percent midday shade, is included because the amount of shade can affect water temperature and photosynthesis in streams. Average daily flows, V_{18}, are associated with rapid upstream migration of steelhead adults. Variables V_{12} and V_{17} are used primarily for streams ≤ 50 m wide where temperature, photosynthesis, or erosion problems occur or when changes in the riparian vegetation are part of a potential project plan. Variable V_{18} is used only for habitat evaluation for spawning migration of steelhead trout.

Suitability Index (SI) Graphs for Model Variables

This section contains suitability index graphs for the 18 model variables. Equations and instructions for combining groups of variable SI scores into component scores and component scores into rainbow trout HSI scores are included.

The graphs were constructed by quantifying information on the effect of each habitat variable on the growth, survival, or biomass of rainbow trout. The curves were built on the assumption that increments of growth, survival, or biomass plotted on the y-axis of the graph could be directly converted into an index of suitability from 0.0 to 1.0 for the species, with 0.0 indicating unsuitable conditions and 1.0 indicating optimal conditions. Graph trend lines represent the authors' best estimate of suitability for the various levels of each variable. The graphs have been reviewed by biologists familiar with the ecology of the species, but some degree of SI variability exists. The user is encouraged to modify the shape of the graphs when existing regional information indicates that the variable suitability relationship is different from that illustrated.

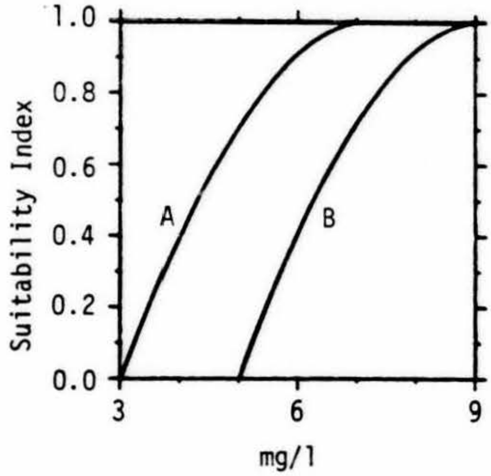
The habitat measurements and the SI graph construction are based on the premise that extreme, rather than average, values of a variable most often limit the carrying capacity of a habitat. Thus, extreme conditions, such as maximum temperatures and minimum dissolved oxygen levels, are often used in the graphs to derive the SI values for the model. The letters R and L in the habitat column identify variables used to evaluate riverine (R) or lacustrine (L) habitats.

Habitat	Variable		Suitability Graph
R,L	V_1	Average maximum water temperature (°C) during the warmest period of the year (adult, juvenile, and fry), and during upstream migrations of adult steelhead. For lacustrine habitats, use the temperature strata nearest to optimal in dissolved oxygen zones > 3 mg/l. A = resident rainbow trout B = migrating adult steelhead	
R	V_2	Average maximum water temperature (°C) during embryo development (all rainbows) and during the March to June smoltification period (steelhead juveniles). A = steelhead smolts B = embryos	

R,L V_3 Average minimum dissolved oxygen (mg/l) during the late growing season low water period and during embryo development (adult, juvenile, fry, and embryo).

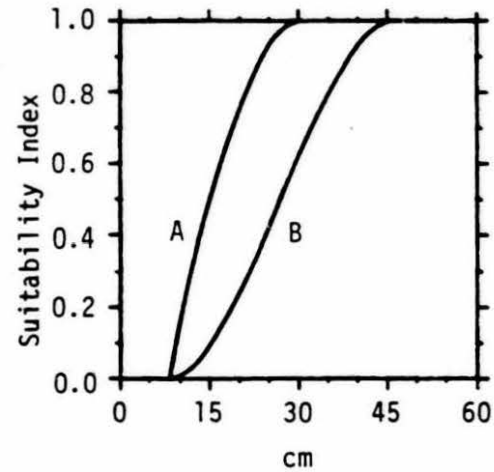
For lacustrine habitats, use the dissolved oxygen readings in temperature zones nearest to optimal where dissolved oxygen is > 3 mg/l.

A = ≤ 15° C
B = > 15° C

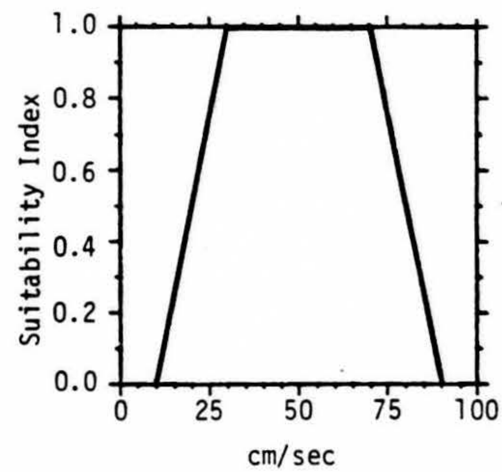


R V_4 Average thalweg depth (cm) during the late growing season low water period (adult).

A = ≤ 5 m stream width
B = > 5 m stream width

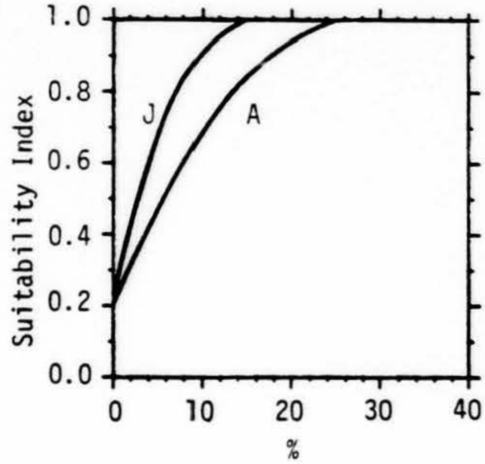


R V_5 Average velocity (cm/sec) over spawning areas during embryo development.



R V_6 Percent instream cover during the late growing season low water period at depths ≥ 15 cm and velocities < 15 cm/sec.

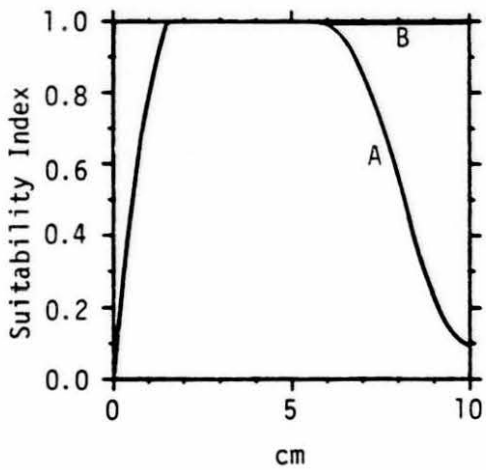
J = juveniles
A = adults



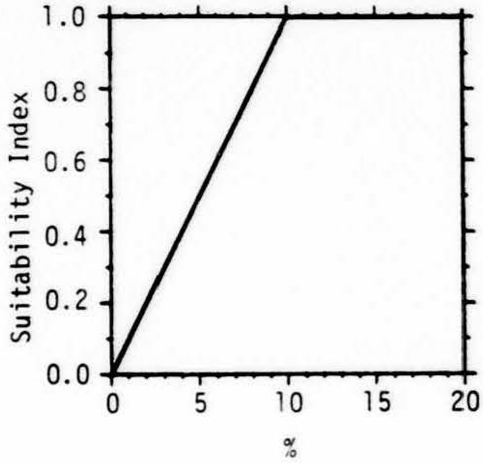
R V_7 Average size of substrate (cm) in spawning areas, preferably during the spawning period.

A = average size of spawner < 50 cm
B = average size of spawner ≥ 50 cm

To derive an average value for use with graph V_7, include areas containing the best spawning substrate sampled until all potential spawning sites are included or until the sample contains an area equal to 5% of the total rainbow habitat being evaluated.

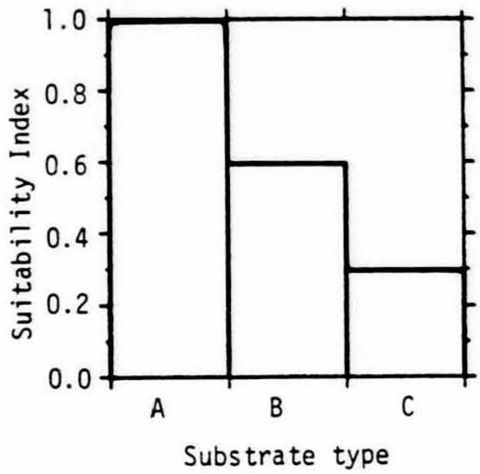


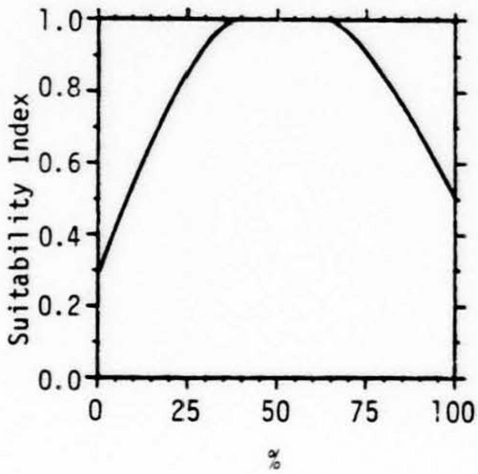
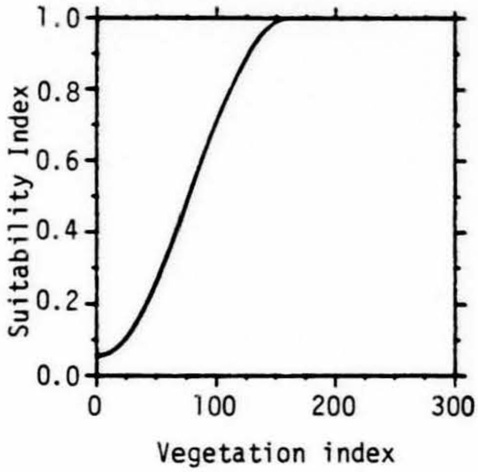
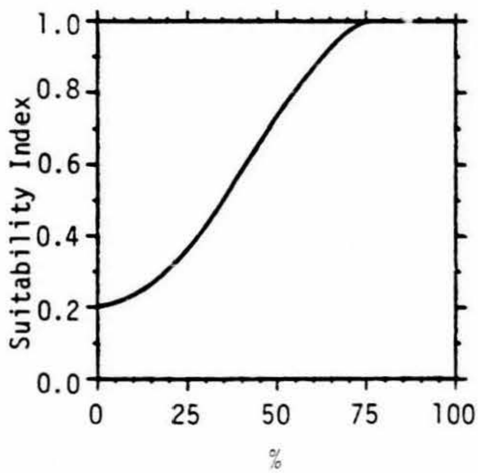
R V_8 Percent substrate size class (10 to 40 cm) used for winter and escape cover by fry and small juveniles.



R V_9 Predominant (≥ 50%) substrate type in riffle-run areas for food production.

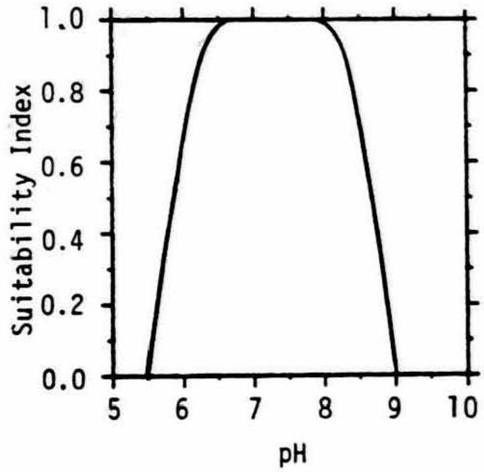
A) Rubble or small boulders (or aquatic vegetation in spring areas) predominant; limited amounts of gravel, large boulders, or bedrock.
B) Rubble, gravel, boulders, and fines occur in approximately equal amounts, or gravel is predominant. Aquatic vegetation may or may not be present.
C) Fines, bedrock, or large boulders are predominant. Rubble and gravel are insignificant (≤ 25%).



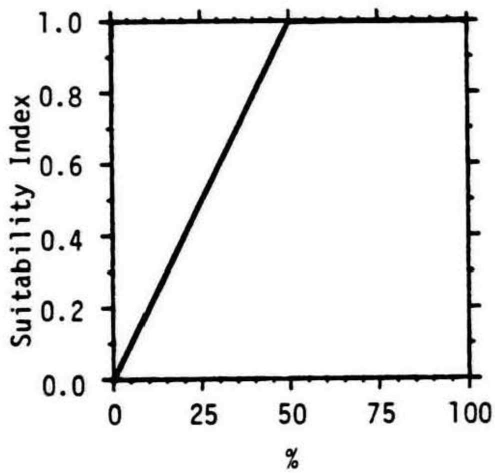
R	V_{10}	Percent pools during the late growing season low water period.	
R	V_{11}	Average percent vegetational ground cover and canopy closure (trees, shrubs, and grasses-forbs) along the streambank during the summer for allochthonous input. Vegetation Index = 2(% shrubs) + 1.5(% grasses) + (% trees). (For streams ≤ 50 m wide)	
R	V_{12} (Optional)	Average percent rooted vegetation and stable rocky ground cover along stream bank.	

R,L V_{13} Annual maximal or minimal pH. Use the measurement with the lowest SI value.

For lacustrine habitats, measure pH in the zone with the best combination of dissolved oxygen and temperature.



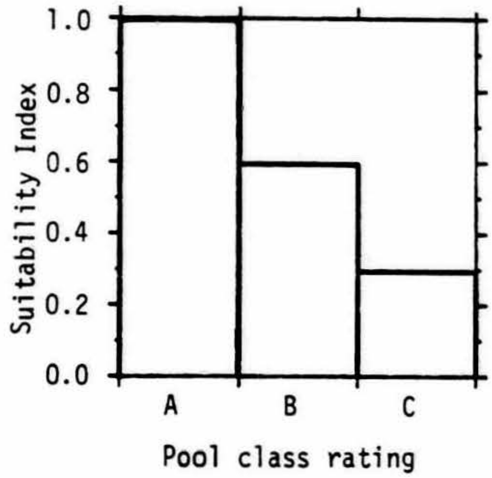
R V_{14} Average annual base flow regime during the late summer or winter low flow period as a percentage of the average annual daily flow.



R V_{15} Pool class rating during the late growing season low flow period. The rating is based on the % of the area that contains pools of the three classes described below:

A) ≥ 30% of the area is comprised of 1st-class pools.
B) ≥ 10% but < 30% of the area is 1st-class pools or ≥ 50% is 2nd-class pools.
C) < 10% of the area is 1st-class pools and < 50% is 2nd-class pools.

(See pool class descriptions below)

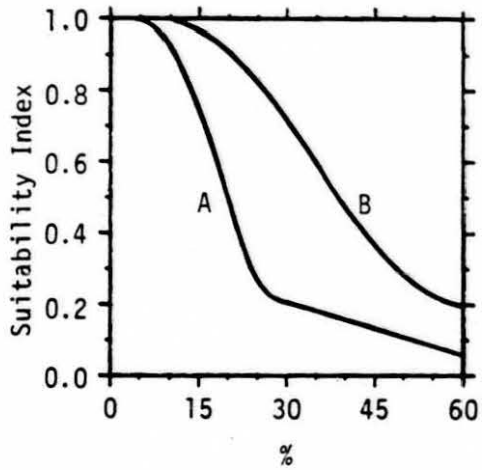


- First-class pool: Large and deep. Pool depth and size are sufficient to provide a low velocity resting area for several adult trout. More than 30% of the pool bottom is obscured due to depth, surface turbulence, or the presence of structures, such as logs, debris piles, boulders, or overhanging banks and vegetation. Or, the greatest pool depth is ≥ 1.5 m in streams ≤ 5 m wide or ≥ 2 m deep in streams > 5 m wide.

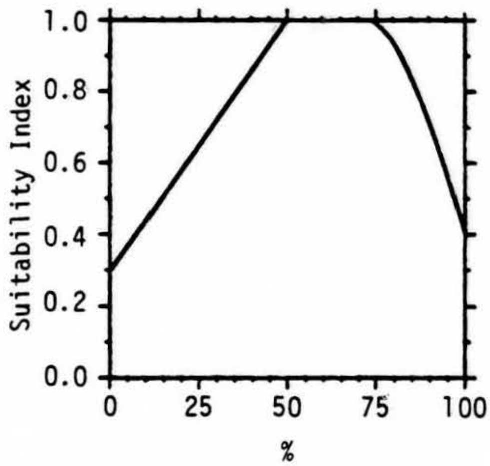
- Second-class pool: Moderate size and depth. Pool depth and size are sufficient to provide a low velocity resting area for a few adult trout. From 5 to 30% of the bottom is obscured due to surface turbulence, depth, or the presence of structures. Typical second-class pools are large eddies behind boulders and low velocity, moderately deep areas beneath overhanging banks and vegetation.

- Third-class pool: Small or shallow or both. Pool depth and size are sufficient to provide a low velocity resting area for one to a very few adult trout. Cover, if present, is in the form of shade, surface turbulence, or very limited structures. Typical third-class pools are wide, shallow pool areas of streams or small eddies behind boulders. The entire bottom area of the pool is visible.

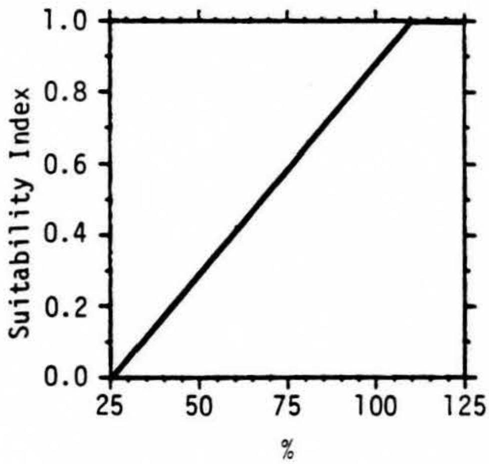
R	V_{16}	Percent fines (< 3 mm) in riffle-run and spawning areas during average summer flows. A = spawning B = riffle-run



R	V_{17} (Optional)	Percent of stream area shaded between 1000 and 1400 hrs (for streams ≤ 50 m wide). Do not use for cold (< 18° C max. temp.), unproductive streams.



R	V_{18} (Optional)	Percent average daily flow during the season of upstream migration of adult steelhead.



An HSI based on the concept of a limiting factor can be obtained for a particular life stage of rainbow trout, or all life stages combined, by selecting the lowest suitability index (SI) for the appropriate listed habitat variables as is done in Table 1.

Alternate models and mathematical methods of aggregating SI's into life stage and species HSI's are presented in the following pages.

Table 1. Matrix table for displaying suitability indices (SI's) for rainbow trout habitat variables.

	SI's				
Variables	Adult	Embryo	Fry	Juvenile	Other
V_1 Maximum temperature					X
V_2 Maximum temperature (embryo)		X			
V_3 Minimum dissolved O_2		X			X
V_2 Average thalweg depth	X				
V_5 Average velocity (spawning)		X			
V_6 % cover	X			X	
V_7 Substrate size (spawning)		X			
V_8 % substrate class			X		
V_9 Substrate type (food)					X
V_{10} % pools	X		X	X	
V_{11} % riparian vegetation					X
V_{12} % ground cover (erosion)[a]					
V_{13} Maximum-minimum pH					X
V_{14} Average annual base flow					X
V_{15} Pool class	X			X	
V_{16} % fines		X	X		X
V_{17} % shade[a]					
V_{18} % average daily flow[a]					

[a]Optional variables.

Data sources and the assumptions used to construct the suitability index graphs for rainbow trout HSI models are presented in Table 2.

Riverine Model

This model uses a life stage approach with five components: adult; juvenile; fry; embryo; and other.

Adult (C_A). C_A variables: V_4; V_5; V_{10}; V_{15}; V_{1B}; and V_{18}

Case 1: Where V_6 is > $(V_{10} \times V_{15})^{1/2}$,

$$C_A = [V_4 \times V_6 \ (V_{10} \times V_{15})^{1/2}]^{1/3}$$

Case 2: Where V_6 is ≤ $(V_{10} \times V_{15})^{1/2}$,

$$C_A = [V_4 \ (V_{10} \times V_{15})^{1/2}]^{1/2}$$

Case 3: Steelhead (C_{AS})

$$C_{AS} = (C_A \times V_{1B} \times V_{18})^{1/3}$$

If V_4 or $(V_{10} \times V_{15})^{1/2}$ is ≤ 0.4 in either equation, then C_A = the lowest factor score.

Juvenile (C_J). C_J variables: V_6; V_{10}; V_{15}; and V_{2A}

Case 1:

$$C_J = \frac{V_6 + V_{10} + V_{15}}{3}$$

Or, if any variable is ≤ 0.4, C_J = the lowest variable score.

Table 2. Literature sources and assumptions for rainbow trout suitability indices.

Variable	and source[a]	Assumptions
V_1	Black 1953 Garside and Tait 1958 Dickson and Kramer 1971 Hanel 1971 May 1973 Cherry et al. 1977	Average maximal daily water temperatures have a greater effect on trout growth and survival than minimal temperatures. The temperature that supports the greatest scope of activity is optimal. In addition, the temperature range associated with rapid migration rates for adult steelhead is optimal.
V_2	Snyder and Tanner 1960 Calhoun 1966 Zaugg and McLain 1972 Zaugg and Wagner 1973 Wagner 1974	The average maximal daily water temperature during the embryo and smoltification development periods that is related to the highest survival of embryos and normal development of smolts is optimal. Temperatures that reduce survival or development of smolts are suboptimal.
V_3	Randall and Smith 1967 Doudoroff and Shumway 1970 Trojnar 1972 Sekulich 1974	The average minimal daily dissolved oxygen level during embryo development and the late growing season that is related to the greatest growth and survival of rainbow trout and trout embryos is optimal. Dissolved oxygen concentrations that reduce survival and growth are suboptimal.
V_4	Delisle and Eliason 1961 Estimated by authors.	Average thalweg depths that provide the best combination of pools, instream cover, and instream movement of adult trout are optimal.
V_5	Delisle and Eliason 1961 Thompson 1972 Hooper 1973 Silver et al. 1963	The average velocities over spawning areas affect habitat suitability because dissolved oxygen is carried to, and waste products are carried away from, the developing embryos. Average velocities that result in the highest survival of embryos are optimal. Velocities that result in reduced survival are suboptimal.

Table 2. (continued)

Variable and source[a]		Assumptions
V_6	Boussu 1954 Elser 1968 Lewis 1969 Wesche 1980	Trout standing crops are correlated with the amount of usable cover. Usable cover is associated with water ≥ 15 cm deep and velocities ≤ 15 cm/sec. These conditions are associated more with pool than with riffle conditions. The best ratio of habitat conditions is approximately 50% pool area to 50% riffle area. Not all of the area of a pool provides usable cover. Thus, it is assumed that optimal conditions exist when usable cover comprises < 50% of the total stream area.
V_7	Orcutt et al. 1968 Bjornn 1969 Phillips et al. 1975 Duff 1980	The average size of spawning gravel that is correlated with the best water exchange rates, proper redd construction, and highest fry survival is assumed to be optimal. The percent total spawning area needed to support a good non-anadromous trout population was calculated from the following assumptions: 1. Excellent riverine trout habitat supports about 500 kg/ha. 2. Spawners comprise about 80% of the weight of the population. 500 kg x 80% = 400 kg of spawners. 3. Rainbow adults average about 0.2 kg each. $\frac{400 \text{ kg}}{0.2 \text{ kg}} = 2{,}000$ adult spawners per hectare 4. There are two adults per redd. $\frac{2{,}000}{2} = 1{,}000$ pairs

Table 2. (continued)

Variable and source[a]		Assumptions
		5. Each redd covers ≥ 0.5 m^2. $1{,}000 \times 0.5 \geq 500\ m^2/ha$ 6. There are 10,000 m^2 per hectare. $\frac{500}{10{,}000}$ = 5% of total area
V_8	Hartman 1965 Everest 1969 Bustard and Narver 1975a	The substrate size range selected for escape and winter cover by trout fry and small juveniles is assumed to be optimal.
V_9	Pennak and Van Gerpen 1947 Hynes 1970 Binns and Eiserman 1979	The predominant substrate type containing the greatest numbers of aquatic insects is assumed to be optimal for insect production.
V_{10}	Elser 1968 Fortune and Thompson 1969 Hunt 1971	The percent pools during late summer low flows that is associated with the greatest trout abundance is optimal.
V_{11}	Idyll 1942 Delisle and Eliason 1961 Chapman 1966 Hunt 1971	The average percent vegetation along the streambank is related to the amount of allochthonous materials deposited annually in the stream. Shrubs are the best source of allochthonous materials, followed by grasses and forbs, and then trees. The vegetational index is a reasonable approximation of optimal and suboptimal conditions for most trout stream habitats.
V_{12}	Oregon/Washington Interagency Wildlife Conference 1979 Raleigh and Duff 1980	The average percent rooted vegetation and rocky ground cover that provides adequate erosion control to the stream is optimal.
V_{13}	Hartman and Gill 1968 Behnke and Zarn 1976	The average annual maximal or minimal pH levels related to high survival of trout are optimal.

Table 2. (concluded)

Variable and source[a]		Assumptions
V_{14}	Duff and Cooper 1976 Binns 1979	Flow variations affect the amount and quality of pools, instream cover, and water quality. Average annual base flows associated with the highest standing crops are optimal.
V_{15}	Lewis 1969	Pool classes associated with the highest standing crops of trout are optimal.
V_{16}	Cordone and Kelly 1961 Bjornn 1969 Phillips et al. 1975 Crouse et al. 1981	The percent fines associated with the highest standing crops of food organisms, embryos, and fry in each designated area are optimal.
V_{17}	Sabean 1976, 1977 Anonymous 1979	The percent of shaded stream area during midday that is associated with optimal water temperatures and photosynthesis rates is optimal.[b]
V_{18}	Withler 1966 Everest 1973	The above average daily flows (freshets) associated with rapid upstream migrations of steelhead adults are optimal. Low flows associated with migration delays are suboptimal.

[a]The above references include data from studies on related salmonid species. This information has been selectively used to supplement, verify, or fill data gaps on the habitat requirements of rainbow trout.

[b]Shading is highly variable from site to site. Low elevations with warmer climates require abundant shading to maintain cool waters. At higher elevations with cooler climates, the absence of shading is beneficial because it results in higher photosynthetic rates and warming of water to a more optimal temperature.

Case 2: Steelhead (C_{JS})

$$C_{JS} = (C_J \times V_{2A})^{1/2}$$

Fry (C_F). C_F variables: V_8; V_{10}; and V_{16}

$$C_F = [V_{10} \, (V_8 \times V_{16})^{1/2}]^{1/2}$$

Or, if V_{10} or $(V_8 \times V_{16})^{1/2}$ is ≤ 0.4, C_F = the lowest factor score.

Embryo (C_E). C_E variables: V_2; V_3; V_5; V_7; and V_{16}

Steps in calculating C_E:

A. A potential spawning site is a ≥ 0.5 m² area of gravel, with an average diameter of 0.3 to 8.0 cm, covered by flowing water $\geq$ 15 cm deep. For steelhead, increase the spawning site area from 0.5 to 2.0 m and the gravel size to 0.3 to 10.0 cm. At each spawning site sampled, record:

1. The average water velocity over the site;
2. The average size of all gravel 0.3 to 8.0 cm;
3. The percentage of fines < 0.3 cm in the gravel; and
4. The total area.

B. Derive a spawning site suitability index (V_s) for each site by combining V_5, V_7, and V_{16} values for each site as follows.

$$V_s = (V_5 \times V_7 \times V_{16})^{1/3}$$

C. Derive a weighted average ($\bar{V}_s$) for all sites included in the sample.

Select the best V_s scores until all sites are included or until a total spawning area equal to, but not exceeding, 5% of the total trout habitat has been included, whichever comes first.

$$\bar{V}_s = \frac{\sum_{i=1}^{n} A_i \, V_{si}}{\text{total habitat area}} / 0.05 \text{ (output cannot exceed 1.0)}$$

where A_i = the area of each spawning site in m^2, but ΣA_i cannot exceed 5% of the total habitat

V_{si} = the individual SI's from the best spawning areas until all spawning sites have been included or until the SI's from an area equal to 5% of the total habitat being evaluated have been included, whichever occurs first.

Disregard area restrictions for steelhead. Because advanced juvenile and adult steelhead mature in the ocean, they can theoretically utilize a much greater spawning area than nonanadromous rainbows.

D. Derive C_E

C_E = the lowest score of V_2, V_3, or $\bar{V}_s$

Other (C_O). C_O variables: V_1; V_3; V_9; V_{11}; V_{12}; V_{13}; V_{14}; V_{16}; and V_{17}

$$C_O = \left[\frac{(V_9 \times V_{16})^{1/2} + V_{11}}{2} \times (V_1 \times V_3 \times V_{12} \times V_{13} \times V_{14} \times V_{17})^{1/N}\right]^{1/2}$$

where N = the number of variables within the brackets. Note that variables V_{12} and V_{17} are optional and, therefore, can be omitted (see page 13).

HSI determination. HSI scores can be derived for a single life stage, a combination of two or more life stages, or all life stages combined. In all cases, except for the embryo component (C_E), an HSI is obtained by combining one or more life stage component scores with the C_O component score.

1. Equal Component Value Method. The equal component value method assumes that each component exerts equal influence in determining the HSI. This method should be used to determine the HSI unless information exists that suggests that individual components should be weighted differently. Components: C_A; C_J; C_F; C_E; and C_O

$$HSI = (C_A \times C_J \times C_F \times C_E \times C_O)^{1/N}$$

where N = the number of components in the equation

Or, if any component is ≤ 0.4, the HSI = the lowest component value.

Solve the equation for the number of components included in the evaluation. There will be a minimum of two; one or more life stage components and the component (C_O), unless only the embryo life stage (C_E) is being evaluated, in which case, the HSI = C_E.

2. Unequal Component Value Method. This method also uses a life stage approach with five components: adult (C_A); juvenile (C_J); fry (C_F); embryo (C_E); and other (C_O). However, the C_O component is divided into two subcomponents, food (C_{OF}) and water quality (C_{OQ}). It is assumed that the C_{OF} subcomponent can either increase or decrease the suitability of the habitat by its effect on growth at each life stage, except embryo. The C_{OQ} subcomponent is assumed to exert an influence equal to the combined influence of all other model components in determining habitat suitability. This method also assumes that water quality is excellent; i.e., $C_{OQ} = 1$. When C_{OQ} is < 1, the HSI is decreased. In addition, when a basis for weighting the individual components exists, model component and subcomponent weights can be increased by multiplying each index value by multiples > 1. Model weighting procedures must be documented.

   Components and subcomponents: C_A; C_J; C_F; C_E; C_{OF}; and C_{OQ}

   Steps:

   A. Calculate the subcomponents C_{OF} and C_{OQ} of C_O

$$C_{OF} = \frac{(V_9 \times V_{16})^{1/2} + V_{11}}{2}$$

$$C_{OQ} = (V_1 \times V_3 \times V_{13} \times V_{14})^{1/4}$$

   Or, if any variable is ≤ 0.4, C_{OQ} = the value of the lowest variable.

B. Calculate the HSI by either the noncompensatory or the compensatory option.

Noncompensatory option. This option assumes that degraded water quality conditions cannot be compensated for by good physical habitat conditions. This assumption is most likely to be true for small streams (≤ 5 m wide) and for persistently degraded water quality conditions.

$$HSI = (C_A \times C_J \times C_F \times C_E \times C_{OF})^{1/N} \times C_{OQ}$$

Or, if any component is ≤ 0.4, the HSI = the lowest component value.

where N = the number of components and subcomponents inside the parentheses or, if the model components or subcomponents are weighted, N = the summation of weights selected

For steelhead, substitute C_{AS} and C_{JS} for C_A and C_J.

If only the embryo component is being evaluated, then HSI = $C_E \times C_{OQ}$.

Compensatory option. This method assumes that moderately degraded water quality conditions can be partially compensated for by good physical habitat conditions. This assumption is most useful for large rivers (≥ 50 m wide) and for temporary poor water quality conditions.

1) $HSI' = (C_A \times C_J \times C_F \times C_E \times C_{OF})^{1/N}$

Or, if any component is ≤ 0.4, the HSI' = the lowest component value.

where N = the number of components and subcomponents in the equation or, if the model components or subcomponents are weighted, N = the summation of the weights selected

For steelhead, substitute C_{AS} and C_{JS} for C_A and C_J.

2) If C_{OQ} is < HSI', then HSI = HSI' x [1 - (HSI' - C_{OQ})]; if C_{OQ} is ≥ HSI, then HSI = HSI'.

3) If only the embryo component is being evaluated, substitute C_E for HSI' and follow the procedure in Step 2.

Lacustrine Models

The following models are designed to evaluate rainbow trout lacustrine habitat. The lacustrine model contains two components, water quality (C_{WQ}) and reproduction (C_R).

Water Quality (C_{WQ}). C_{WQ} variables: V_1; V_3; and V_{13}

$$C_{WQ} = (V_1 \times V_3 \times V_{13})^{1/3}$$

Or, if the SI scores for V_1 or V_3 are ≤ 0.4, C_{WQ} = the lowest SI score for V_1 or V_3.

Note: Lacustrine rainbows require a tributary stream for spawning and embryo development. If the embryo life stage habitat is included in the evaluation, use the embryo component steps and equations in the riverine model above, except that the area of spawning gravel needed is only about 1% of the total surface area of the lacustrine habitat.

Embryo (C_E). C_E variables: V_2; V_3; V_5; V_7; and V_{16}

$$\bar{V}_s = \frac{\sum_{i=1}^{n} A_i V_{si}}{\text{total habitat area}} / 0.01 \text{ (output cannot be > 1.0)}$$

$$HSI = (C_{WQ} \times C_E)^{1/2}$$

If only the lacustrine habitat is evaluated, the HSI = C_{WQ}.

Interpreting Model Outputs

Model HSI scores for individual life stages, composite life stages, or for the species as a whole are a relative indicator of habitat suitability. The HSI models, in their present form, are not intended to predict standing crops of fishes throughout the United States. Standing crop limiting factors, such as interspecific competition, predation, disease, water nutrient levels, and length of growing season, are not included in the aquatic HSI models. The models contain physical habitat variables important in maintaining viable populations of rainbow trout. If the model is correctly structured, a high HSI score for a habitat would indicate near optimal regional conditions for rainbow trout for those factors included in the model, intermediate HSI scores would indicate average habitat conditions, and low HSI scores would indicate poor habitat conditions. An HSI of 0 does not necessarily mean that the species is not present; it does mean that the habitat is very poor and that the species is likely to be scarce or absent.

Rainbow trout tend to occupy riverine habitats where few other fish species are present. Thus, factors such as disease, interspecific competition, and predation may have little affect on the model. When the rainbow trout model is applied to rainbow trout streams containing few other species and similar water quality and length of growing season conditions, it should be possible to calibrate the model output to reflect the size of standing crops within some reasonable confidence limits. This possibility, however, has not been tested with the present model.

Sample data sets selected by the authors to represent high, intermediate, and low habitat suitabilities are in Table 3, along with the SI's and HSI's generated by the rainbow trout nonanadromous riverine model. The model outputs calculated from the sample data sets (Tables 4 and 5) reflect what the authors believe carrying capacity trends would be in riverine habitats with the listed characteristics; thus, the model meets the specified acceptance level.

ADDITIONAL HABITAT MODELS

Model 1

Optimal riverine rainbow trout habitat is characterized by:

1. Clear, cold water with an average maximum summer temperature of < 22° C;

2. Approximately a 1:1 pool-to-riffle ratio;

3. Well vegetated, stable stream banks;

4. Cover ≥ 25% of the stream area;

5. Relatively stable water flow regime with < 50% of the annual fluctuation from the average annual daily flow;

Table 3. Sample data sets using the nonanadromous riverine rainbow trout HSI model.

Variable		Data set 1		Data set 2		Data set 3	
		Data	SI	Data	SI	Data	SI
Max. temperature (°C)	V_1	14	1.0	15	1.0	16	1.0
Max. temperature (°C)	V_2	12	1.0	15	0.66	17	0.4
Min. dissolved O_2 (mg/l)	V_3	9	1.0	7	0.73	6	0.42
Ave. depth (cm)	V_4	25	0.9	18	0.6	18	0.6
Ave. velocity (cm/s)	V_5	30	1.0	25	0.7	20	0.57
% cover	V_6	20	A 0.95 J 1.0	10	A 0.65 J 0.92	10	A 0.65 J 0.92
Ave. gravel size (cm)	V_7	4	1.0	3	1.0	2.5	1.0
Predom. substrate size (cm)	V_8	15	1.0	7	0.7	7	0.7
Predom. substrate type	V_9	Class A	1.0	Class B	0.6	Class B	0.6
% pools	V_{10}	55	1.0	15	0.65	10	0.46
% streamside vegetation	V_{11}	225	1.0	175	1.0	200	1.0
% bank vegetation	V_{12}	95	1.0	40	0.6	35	0.5
Max. pH	V_{13}	7.1	1.0	7.2	1.0	7.2	1.0
% average base flow	V_{14}	37	0.8	30	0.6	25	0.5
Pool class rating	V_{15}	Class A	1.0	Class B	0.6	Class C	0.3
% fines (A)	V_{16}	5	1.0	20	0.5	20	0.5
% fines (B)	V_{16}	20	0.9	30	0.6	30	0.6
% midday shade	V_{17}	60	1.0	60	1.0	60	1.0

Table 4. Average value method.

Variable	Suitability index Data set 1	Data set 2	Data set 3
Component			
C_A	0.95	0.62	0.37
C_J	1.00	0.72	0.30
C_F	1.00	0.65	0.55
C_E	1.00	0.66	0.40
C_O	0.96	0.79	0.73
Species HSI	0.98	0.68	0.30

Table 5. Average value, probability method.

Variable	Suitability index Data set 1	Data set 2	Data set 3
Component			
C_A	0.95	0.62	0.37
C_J	1.00	0.72	0.30
C_F	1.00	0.65	0.55
C_E	1.00	0.66	0.40
C_{OF}	0.97	0.80	0.80
C_{OQ}	0.95	0.81	0.68
Species HSI			
Noncompensatory	0.94	0.56	0.30
Compensatory	0.95	0.70	0.30

6. Relatively stable summer temperature regime, averaging about 13° C ± 4° C; and

7. A relatively silt free rocky substrate in riffle-run areas.

$$HSI = \frac{\text{number of attributes present}}{7}$$

Model 2

A riverine trout habitat model has been developed by Binns and Eiserman (1979) and Binns (1979). Transpose the model output of pounds per acre to an index as follows:

$$HSI = \frac{\text{model output of pounds per acre}}{\text{regional maximum pounds per acre}}$$

Model 3

Optimal lacustrine rainbow trout habitat is characterized by:

1. Clear, cold water with an average summer midepilimnion temperature of < 22° C;

2. Dissolved oxygen content of epilimnion of ≥ 8 mg/l;

3. Access to riverine spawning tributaries;

4. A midepilimnion pH of 6.5-8.0; and

5. Abundance of aquatic invertebrates.

$$HSI = \frac{\text{number of attributes present}}{5}$$

However, a high elevation lake with optimal habitat will have only a fraction of the trout production of a more eutrophic lake at a lower elevation, if no other fish species are present in either lake.

Model 4

A low effort system for predicting habitat suitability of planned cool water and cold water reservoirs as habitat for individual fish species is also available (McConnell et al. 1982).

INSTREAM FLOW INCREMENTAL METHODOLOGY (IFIM)

The U.S. Fish and Wildlife Service's Instream Flow Incremental Methodology (IFIM), as outlined by Bovee 1982, is a set of ideas used to assess instream flow problems. The Physical Habitat Simulation System (PHABSIM), described by Milhous et al. 1981, is one component of IFIM that can be used by investigators interested in determining the amount of available instream habitat for a fish species as a function of streamflow. The output generated by PHABSIM can be used for several IFIM habitat display and interpretation techniques, including:

1. Optimization. Determination of monthly flows that minimize habitat reductions for species and life stages of interest;

2. Habitat Time Series. Determination of the impact of a project on habitat by imposing project operation curves over historical flow records and integrating the difference between the curves; and

3. Effective Habitat Time Series. Calculation of the habitat requirements of each life stage of a fish species at a given time by using habitat ratios (relative spatial requirements of various life stages).

Suitability Index Graphs as Used in IFIM

PHABSIM utilizes Suitability Index graphs (SI curves) that describe the instream suitability of the habitat variables most closely related to stream hydraulics and channel structure (velocity, depth, substrate, temperature, and cover) for each major life stage of a given fish species (spawning, egg incubation, fry, juvenile, and adult). The specific curves required for a PHABSIM analysis represent the hydraulic-related parameters for which a species or life stage demonstrates a strong preference (i.e., a pelagic species that only shows preferences for velocity and temperature will have very broad curves for depth, substrate, and cover). Instream Flow Information Papers 11 (Milhous et al. 1981) and 12 (Bovee 1982) should be reviewed carefully before using any curves for a PHABSIM analysis. SI curves used with the IFIM are quite similar to the SI curves developed in many HSI models. These two types of SI curves may be interchangeable after conversion to the same units of measurement (English, metric, or codes). SI curve validity is dependent on the quality and quantity of information used to generate the curve. The curves used need to accurately reflect the conditions and assumptions inherent to the model(s) used to aggregate the curve-generated SI values into a measure of habitat suitability. If the necessary curves are unavailable or if available curves are inadequate (i.e., built on different assumptions), a new set of curves should be generated.

There are several ways to develop SI curves. The method selected depends on the habitat model that will be used and the available database for the species. The transferability of the curve is not obvious and, therefore, the method by which the curve is generated and the extent of the database are very important. Care also must be taken to choose the habitat model most appropriate for the specific study or evaluation; the choice of models will determine the type of SI curves that will be used.

A system with standard terminology has been developed for classifying SI curve sets and describing the database used to construct the curves in IFIM applications. There are four categories in the classification. A category one curve has a generalized description or summary of habitat preferences as its database. This type of curve is based on information concerning the upper and lower limits of a variable for a species (e.g., juveniles are usually found at water depths of 0.3 to 1.0 m). Expert opinion can also be used to define the optimal or preferred condition within the limits of tolerance (e.g., juveniles are found at water depths of 0.3 to 1.0 m, but are most common at depths from 0.4 to 0.6 m).

Utilization curves (category two) are based on frequency analyses of fish observations in the stream environment with the habitat variables measured at each sighting [see Instream Flow Information Paper 3 (Bovee and Cochnauer 1977) and Instream Flow Information Paper 12 (Bovee 1982:173-196)]. These curves are designated as utilization curves because they depict the habitat conditions a fish has been observed to use within a specific range of available conditions. If the data are correctly collected for utilization curves, the resulting function represents the probability of occurrence of a particular environmental condition, given the presence of a fish of a particular species, $P(E|F)$. However, due to sampling problems, a utilization curve cannot be assumed to reflect fish preference or optimal conditions. Also, utilization curves may not be transferable to streams that differ substantially in size or complexity from the streams where the data were obtained.

A preference curve (category three) is a utilization curve that has been corrected for environmental bias. For example, if 50% of the fish are found in pools over 1.0 m deep, but only 10% of the stream has such pools, the fish are actively selecting that type of habitat. Preference curves approximate the function of the probability of occurrence of a fish in a given set of environmental conditions:

$$P(F|E) \approx \frac{P(E|F)}{P(E)}$$

Only a limited number of experimental data sets have been compiled into IFIM preference curves. The development of these curves should be the goal of all new curve development efforts for use in IFIM.

An additional set of curves is still largely conceptual. One type of curve under consideration is a cover-conditioned, or season-conditioned, preference curve set. Such a curve set would consist of different depth-velocity preference curves as a function or condition of the type of cover present or the time of year. No fourth category curves have been developed at this time.

The advantage of these last two sets of curves is the significant improvement in precision and confidence in the curves when applied to streams similar to the streams where the original data were obtained. The degree of increased accuracy and transferability obtainable when applying these curves to dissimilar streams is unknown. In theory, the curves should be widely transferable to any stream in which the range of environmental conditions is within the range of conditions found in the streams from which the curves were developed.

Availability of Graphs for Use in IFIM

Numerous curves are available for the IFIM analysis of rainbow trout habitat (Table 6). Investigators are asked to review the curves (Figs. 2-5) and modify them, if necessary, before using them.

Spawning. For IFIM analyses of rainbow trout spawning habitat, use curves for the time period during which spawning occurs (which is dependent on locale). Spawning curves are broad and, if more accuracy is desired, investigators are encouraged to develop their own curves which will specifically reflect habitat utilization at the selected site.

Spawning velocity. Hartman and Galbraith (1970) measured water velocities 0.66 feet above 550 redds during April through June, 1966 and 1967, in the Lardeau River, British Columbia, and found few redds constructed in areas where velocities were less than 1.0 feet per second (fps); most redds were associated with velocities ranging 1.6 to 3.0 fps (velocities from zero to greater than 4.0 fps were available). Smith (1973) determined that 95% of the redds (n = 51) observed in the Deschutes River, Oregon, were in velocities ranging from 1.6 to 3.0 fps; Hooper (1973) measured velocities over redds (n = 10) which ranged from 1.4 to 2.7 fps in the South Fork of the Feather River, California; and Orcutt et al. (1968) found that average velocities 0.4 feet above redds (n = 54 to 68) ranged from 2.3 to 2.5 fps in Idaho streams. The SI curve for spawning velocity (Fig. 2) assumes that velocities less than 1.0 or greater than 3.0 fps are unsuitable for rainbow trout spawning.

Table 6. Availability of curves for IFIM analysis of rainbow trout habitat.

	Velocity[a]	Depth[a]	Substrate[a,b]	Temperature[a]	Cover[a]
Spawning	Use SI curve, Fig. 2.	Use SI curve, Fig. 2.	Use SI curve, Fig. 2.	Use SI curve, Fig. 2.	No curve necessary.
Egg incubation	Use SI curve, Fig. 2.	Use SI curve, Fig. 2.	Use SI curve for V_{7B}.	Use SI curve for V_{2B}.	No curve necessary.
Fry	Use SI curve, Fig. 3.	Use SI curve, Fig. 3.	Use SI curve, Fig. 3.	Use SI curve, Fig. 3.	No curve available.
Juvenile	Use SI curve, Fig. 4.	Use SI curve, Fig. 4.	Use SI curve, Fig. 4.	Use SI curve, Fig. 4.	Use SI curve for V_6.
Adult	Use SI curve, Fig. 5.	Use SI curve, Fig. 5.	Use SI curve, Fig. 5.	Use SI curve, Fig. 5.	Use SI curve for V_6.

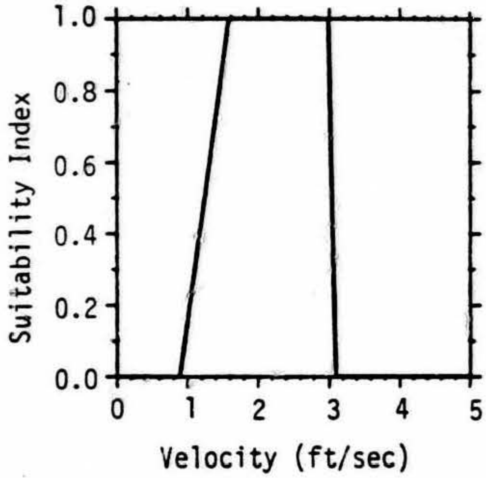
[a]When use of SI curves is prescribed, refer to the appropriate curve in the HSI or IFIM section.

[b]The following categories must be used for IFIM analyses (see Bovee 1982):

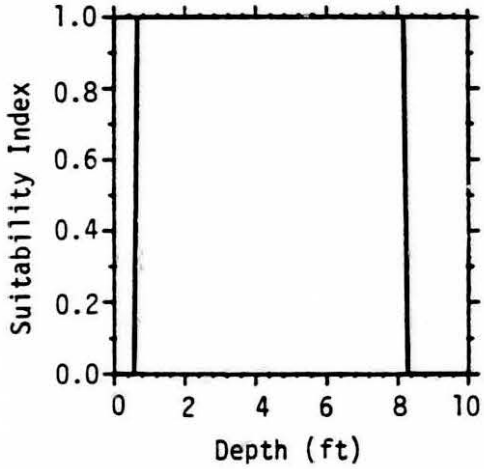
1 = plant detritus/organic material
2 = mud/soft clay
3 = silt (particle size < 0.062 mm)
4 = sand (particle size 0.062-2.000 mm)
5 = gravel (particle size 2.0-64.0 mm)
6 = cobble/rubble (particle size 64.0-250.0 mm)
7 = boulder (particle size 250.0-4000.0 mm)
8 = bedrock (solid rock)

Coordinates

x	y
0.0	0.0
0.9	0.0
1.6	1.0
3.0	1.0
3.1	0.0
100.0	0.0



x	y
0.0	0.0
0.6	0.0
0.7	1.0
8.2	1.0
8.3	0.0
100.0	0.0



x	y
0.0000	0.0
0.0005	0.0
0.0010	1.0
4.0000	1.0
4.1000	0.0
100.0000	0.0

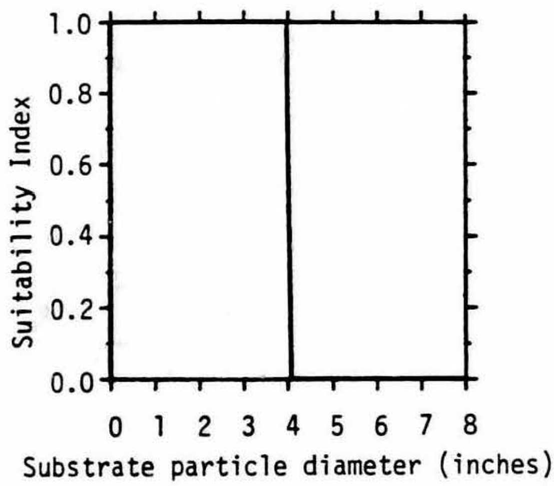


Figure 2. SI curves for rainbow trout spawning velocity, depth, substrate, and temperature.

Coordinates

x	y
0.0	0.0
35.0	0.0
36.0	1.0
60.0	1.0
61.0	0.0
100.0	0.0

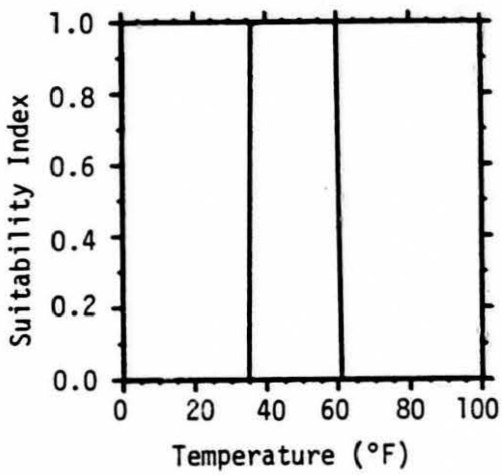


Figure 2. (concluded)

Spawning Depth. Smith (1973) found 95% of rainbow trout redds in depths greater than 0.6 feet; Hooper (1973) found depths of redds ranged from 0.7 to 1.1 feet; Orcutt et al. found redds in depths from 0.7 to greater than 5.0 feet; Hartman and Galbraith (1970) found the majority of redds (n = 550) in depths ranging from 1.6 to 8.2 feet, and the most intensive nest digging occurred in depths of 5.7 to 6.6 feet (maximum depths available ranged from 13 to 16 feet). Therefore, depths less than 0.6 and greater than 8.2 feet are assumed to be unsuitable for spawning (although 8.2 feet may not be the upper limit; Fig. 2).

Spawning substrate. Hartman and Galbraith (1970) found that gravel composed of particle sizes ranging from 0.04 to 4.0 inches in diameter were utilized for spawning, two-thirds of which were from 0.5 to 3.0 inches in diameter (particle sizes to 18 inches in diameter were available). Hooper (1973) found that preferred spawning substrate consisted of particles from 0.5 to 1.5 inches in diameter, although particles from 0.25 to 3.0 inches were utilized; Orcutt et al. (1968) found that steelhead favored gravels from 0.5 to 4.0 inches in diameter; Coble (1961) stated that salmonids dug redds in substrates ranging from silt to 3-inch diameter cobble (particles up to 4 inches in diameter were available) in Lincoln County, Oregon. Therefore, substrates consisting of particle sizes ranging from silt (< 0.002 inches) to cobble (4.0 inches) are considered suitable for spawning (though not necessarily suitable for egg incubation; Fig. 2). The particle size range of spawning substrate selected may be dependent on the size of the spawner.

Spawning cover. No information was found in the literature concerning cover requirements for rainbow trout spawning. The author assumes that cover is not important, and it may be omitted from the FISHFIL (Milhous et al. 1981).

Spawning temperature. Scott and Crossman (1973) stated that rainbow trout usually spawn at 50 to 60° F; Carlander (1969) stated that peak spawning near Finger Lakes, New York, occurred at 42 to 55° F; Hooper (1973) stated that temperatures ranging from 37 to 55° F are desirable for spawning; and Orcutt et al. (1981) found that spawning occurred at 36 to 47° F in Idaho streams. The author assumes that temperatures ranging from 36 to 60° F are suitable for spawning (Fig. 2), depending on locale.

Egg incubation. For IFIM analyses of rainbow trout egg incubation habitat, curves should be used for the time period from the beginning of spawning to 30 to 100 days beyond the end of spawning, depending on locale and water temperatures (Carlander 1969). The recommended analysis of incubation with IFIM is the effective spawning program, which computes suitability of each stream cell based on both spawning and incubation criteria over a range of flows. This program is explained in a working paper available from IFG (Milhous 1982).

Egg incubation velocity. Evidence suggests that water velocity may not be important for embryo hatching success if dissolved oxygen concentrations around embryos are greater than 2.6 ppm (Coble 1961; Silver et al. 1963; Reiser and White 1981, 1983). At low concentrations of dissolved oxygen, however, apparent velocities must be sufficient to deliver oxygen to embryos, remove metabolic waste products, and keep the substrate free from silt. Silver et al. (1963) found that rates of development and lengths of fry upon hatching were greater at higher velocities and dissolved oxygen concentrations. Therefore, given suitable spawning velocities (Fig. 2), it may be assumed that egg incubation velocities above redds which range from 1.0 to 3.0 fps (at adequate levels of dissolved oxygen and suitable substrate) will yield suitable apparent velocities among embryos. Another type of incubation velocity criteria is based on the shear velocities needed to prevent sediment of various sizes from settling out on the redds. This approach is documented in the effective spawning working paper mentioned above.

Egg incubation depth. Depth may not be an important variable for egg incubation in many cases (Reiser and White 1981, 1983) as long as eggs are kept moist during incubation and redds are submerged when fry begin to hatch and emerge. Therefore, the author assumes that the SI curve for spawning depth (Fig. 2) may also be used for egg incubation depth.

Egg incubation substrate. Although rainbow trout utilize a wide range of substrate types for spawning (Coble 1961), the author assumes that particles must be at least 0.5 inches in diameter to permit adequate percolation for successful embryonic development, and the SI curve for V_{7B} (page 16) may be used.

Egg incubation cover. The author assumes that cover is not important, and cover may, therefore, be omitted from FISHFIL. The egg incubation substrate curve should satisfy embryo cover requirements.

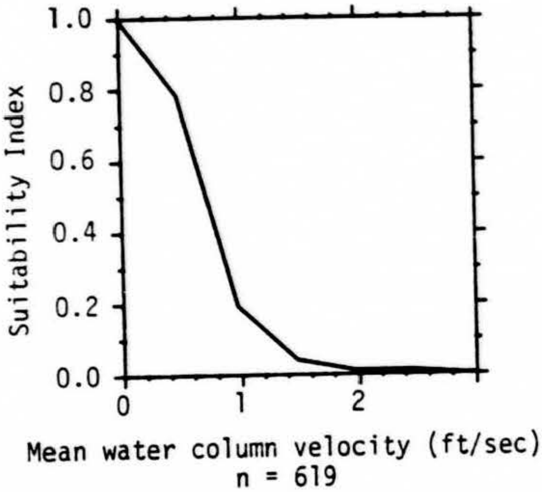
Egg incubation temperature. Kwain (1975) found the highest survival rate for rainbow trout embryos at temperatues of 45 and 50° F; low survival (15 to 40%) at 59° F; and moderate survival at 37 and 41° F. According to Hooper (1973), the desirable temperature range for egg incubation of trout is 42 to 54° F, and the extremes are 35 and 61° F. Therefore, the SI curve for V_{2B} (page 14) may be used.

Fry. Rainbow trout fry lose their yolk sacs at lengths of 1.4 to 1.6 inches, approximately 3-4 months after hatching (Carlander 1969). The author assumes that fry habitat is required from the end of the spawning period to 4 months beyond the end of the egg incubation period (from the time that fry emerge from the spawning gravel to when they become juveniles).

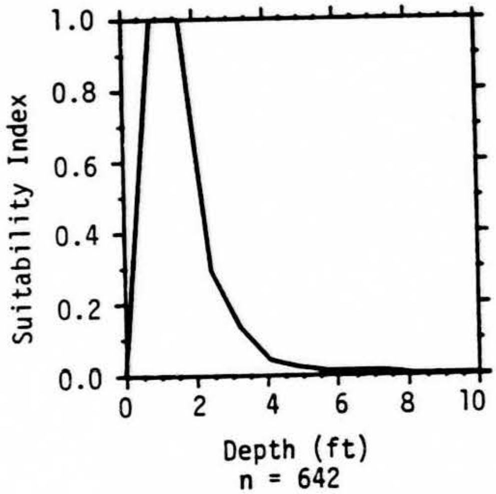
Fry velocity. Moyle et al. (1983) observed fry (≤ 2 inches in length) in Cherry and Eleanor Creeks (n=404), Putah Creek (n=134), and Deer Creek (n=81), California. Maximum velocities available were 2 fps in Putah Creek and > 3.44 fps in Cherry, Eleanor, and Deer Creeks. Weighted mean frequencies were calculated for each velocity and then normalized, with the highest frequency being set to SI = 1.0 (Fig. 3).

Coordinates

x	y
0.00	1.00
0.49	0.78
0.98	0.19
1.48	0.04
1.97	0.01
2.46	0.01
2.95	0.00
100.00	0.00



x	y
0.00	0.00
0.10	0.11
0.82	1.00
1.64	1.00
2.46	0.29
3.28	0.13
4.10	0.04
4.92	0.02
5.74	0.01
7.38	0.01
8.20	0.00
100.00	0.00



x	y
0.0	0.00
1.0	0.00
2.0	0.02
3.0	0.01
4.0	0.17
5.0	0.13
6.0	1.00
7.0	1.00
8.0	0.18
8.1	0.00
100.0	0.00

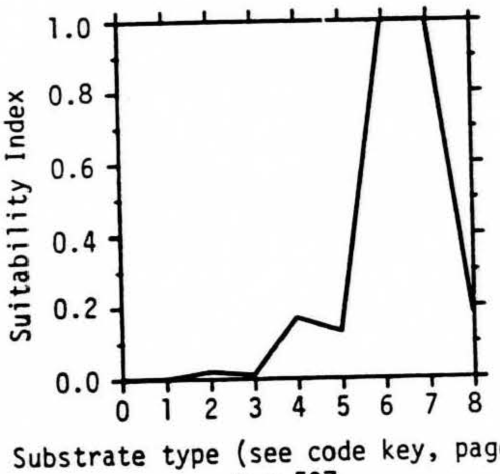


Figure 3. SI curves for rainbow trout fry velocity, depth, substrate, and temperature.

Coordinates	
x	y
0.0	0.00
32.0	0.00
37.4	0.08
50.0	0.80
56.8	1.00
66.2	1.00
69.8	0.80
77.0	0.00
100.0	0.00

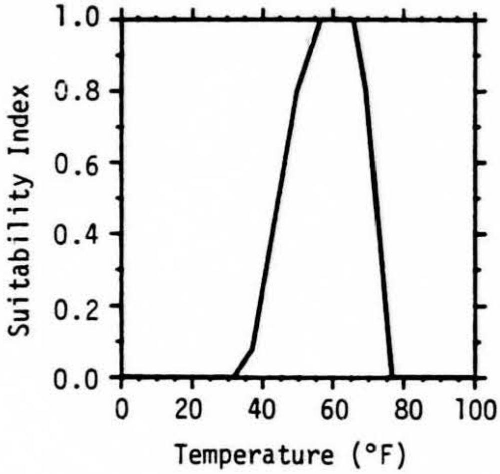


Figure 3. (concluded)

Fry depth. The SI curve for fry depth was developed in the same manner as the velocity curve, using data from Deer Creek (n=103), Cherry and Eleanor Creeks (n=404), and Putah Creek (n=135) (Moyle et al. 1983). Depths to 1.7 ft were available in Putah Creek; to 6.6 ft in Deer Creek; and to greater than 9.0 ft in Cherry and Eleanor Creeks. The final curve (Fig. 3) was modified based on the assumption that SI = 1.0 at depths ranging from 0.82 ft (as in Putah and Deer Creeks) to 1.64 ft (as in Cherry and Eleanor Creeks); and that SI = 0.0 at a depth of 0.0 ft and SI = 0.11 at a depth of 0.1 ft. For depths greater than 1.7 ft, Putah Creek was excluded, and for depths greater than 6.6 ft, Deer Creek was excluded from the analysis.

Fry substrate. The SI curve for fry substrate was generated in the same way as the curves for depth and velocity. Substrate available in Putah Creek (n=123) ranged from mud to bedrock; and in Deer Creek (n=70), Cherry Creek, and Eleanor Creek (n=404) it ranged from silt to bedrock. The curve (Fig. 3) was modified based on the assumption that SI = 1.0 for cobbles (as in Deer and Putah Creeks) and for boulders (as in Cherry and Eleanor Creeks). Bustard and Narver (1975) also found that age 0 steelhead associated with substrate consisting primarily of particles from 4 to 10 inches in diameter (cobble) in Carnation Creek, British Columbia, during the winter.

Fry cover. Cover requirements of rainbow trout fry are unknown by IFASG at this time. The author assumes that substrate is used for cover, and thus cover may be omitted from FISHFIL; or a cover curve may be developed by the investigator.

Fry temperature. Peterson et al. (1979), in lab experiments, found that temperatures preferred by rainbow trout fry (between 1.1 and 1.8 inches in length) ranged from 56.8 to 58.6° F (n=30). Kwain and McCauley (1978) found that age was a factor in temperatures preferred (selected) by rainbow trout, and fry at 1 month selected 66.2° F; at 2 months, 65.3° F; at 3 months, 64.4° F; and at 5 months, 59.7 to 63.7° F. Kwain (1975) found that the growth rate of fry at 50 F was ten times greater than at 37.4° F. Based on the information available for fry up to 1.8 inches in length and up to 4 months after hatching, the SI curve for temperature (Fig. 3) is assumed to be reasonably accurate. It will be modified as new information becomes available.

Juvenile. Juvenile rainbow trout range in length from approximately 1.8 to 7.9 inches, or from 4 months of age to sexual maturity (usually age II or III; Carlander 1969). Juveniles are probably the most difficult life stage for which to develop criteria, because of the variability in size.

Juvenile velocity. Factors which may affect velocity preferences of rainbow trout include water temperature, size and activity of the individual trout, stream flow, season, habitat availability, species interactions, and stream location (Logan 1963; Chapman and Bjornn 1969; Bjornn 1971; Everest and Chapman 1972; Bustard and Narver 1975; Moyle et al. 1983). Moyle et al. (1983) observed juvenile rainbow trout (from 2.0 to 4.7 inches in length) in Putah Creek (n=35), Deer Creek (n=108), Martis Creek (n=58), Cherry Creek, and Eleanor Creek (n=300), and in the Tuolumne River (n=45). The maximum velocity available in Putah Creek was 2.0 fps; in the Tuolumne River it was 2.5 fps; in Martis Creek it was 4.2 fps; and in Deer, Cherry, and Eleanor Creeks it was greater than 3.4 fps. Data from all streams were used to generate the velocity curve (Fig. 4). Weighted means were calculated for each velocity and then normalized, with the highest SI being set to 1.0. The final curve is very similar to the results of Bustard and Narver (1975) who collected Age 0 (n=78) and Age I+ (n=122) juvenile steelhead trout in Carnation Creek, British Columbia, during the winter. They found that juveniles preferred velocities less than 0.5 fps, and almost no individuals were found in velocities greater than 1 fps at 45° F. Gosse (1982), however, found that velocities preferred by juvenile rainbow trout (< 9 inches in length) were partially dependent upon activity (random or stationary swimming), season, and flow. Gosse found that the average mean column velocities occupied by juveniles in low to high flows during random swimming ranged from 0.40 to 0.56 fps in the winter and from 0.43 to 0.75 fps in the summer. During stationary swimming, velocities ranged from 0.66 to 1.18 fps in the winter and from 1.05 to 2.00 fps in the summer. Differences may be due in part to differences in the sizes of the juveniles observed (Chapman and Bjornn 1969).

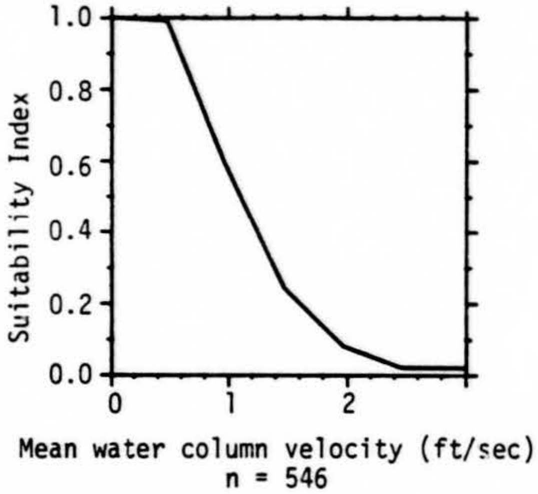
Juvenile depth. Factors which may affect depth preferences of juvenile rainbow trout are the same as those which affect velocity preferences. Observations of Moyle et al. (1983) in Putah Creek (n=36, maximum depths of 1.7 ft), Martis Creek (n=58, depths to 4 ft), Deer Creek (n=126, depths to 6.6 ft), Tuloumne River (n=44, depths to greater than 9 ft), and Cherry and Eleanor Creeks (n=301, depths to greater than 9 ft), of juveniles (2.0 to 4.7 inches in length) indicated substantial variability in depth preferences from stream to stream, with most fish located in depths of 1 to 4 ft. Bustard and Narver (1975), however, found that Age 0 steelhead in Carnation Creek, British Columbia, preferred depths to 1.5 ft, while Age I+ steelhead preferred depths greater than 3 ft. Also, Gosse (1982) found that the average water depths occupied by juvenile rainbow trout (< 9 to 10 inches in length) in the Green River below Flaming Gorge Dam, Colorado, ranged between 10 and 14 ft in the summer and between 18 and 20 ft in the winter (n=111 to 291). Therefore, it is easy to see that juvenile rainbow trout may occupy a wide variety of depths, and it is recommended that investigators develop their own depth curves, or assume that SI = 1.0 for all depths ≥ 2.0 feet (Fig. 4).

Juvenile substrate. Moyle et al. (1983) found most juvenile rainbow trout (2.0 to 4.7 inches in length) over gravel and cobble in Martis Creek (n=53), over cobble and boulders in Deer Creek (n=103), and over boulders in Cherry and Eleanor Creeks (n=301). The composite weighted substrate curve (Fig. 4) shows a preference for boulder substrate. In the Green River, Gosse (1982) found most juveniles (< 9 to 10 inches in length) over cobble and boulders during stationary swimming, but over silt and sand during random swimming. Bjornn (1971) found a correlation between the movement (out migration) of juvenile trout and the lack of large cobble substrate in Idaho streams. Therefore, it may be assumed that cobble and boulders are suitable juvenile substrate, or curves may be developed that are specific to the area of interest.

Juvenile cover. Cover requirements or preferences of juveniles are unknown. It may be assumed that substrate curve reflects cover requirements; cover curves may be developed by the investigator; cover may be omitted from FISHFIL; or, the curve for V_6 may be used to represent juvenile rainbow trout cover requirements (page 16).

Coordinates

x	y
0.00	1.00
0.49	0.99
0.98	0.59
1.48	0.24
1.97	0.08
2.46	0.02
3.44	0.02
3.50	0.00
100.00	0.00



x	y
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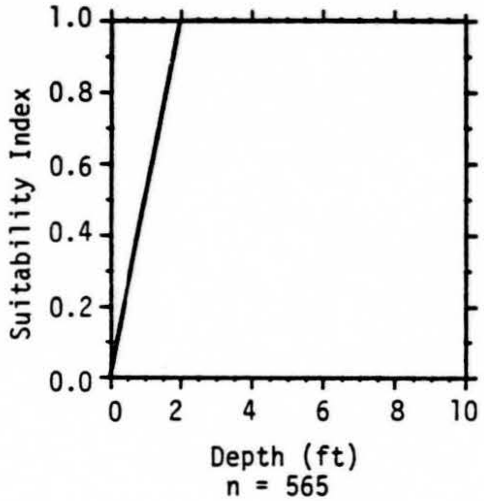
2.0	1.0
100.0	1.0



x	y
0.0	0.00
2.0	0.00
3.0	0.02
4.0	0.11
5.0	0.21
6.0	0.77
7.0	1.00
8.0	0.47
8.1	0.00
100.0	0.00

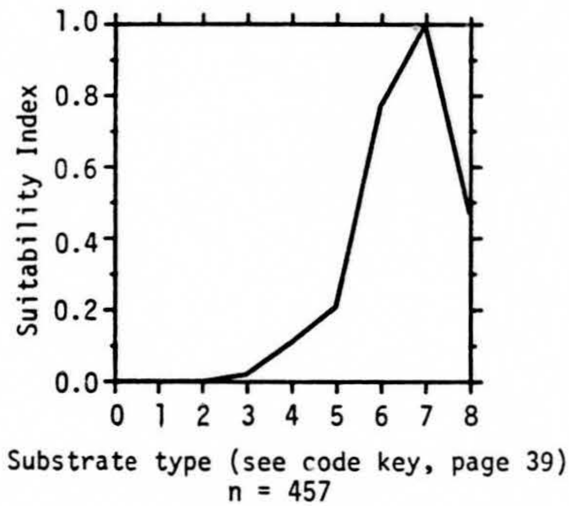


Figure 4. SI curves for rainbow trout juvenile velocity, depth, substrate, and temperature.

Coordinates	
x	y
0.0	0.0
32.0	0.0
50.0	1.0
72.0	1.0
84.0	0.0
100.0	0.0

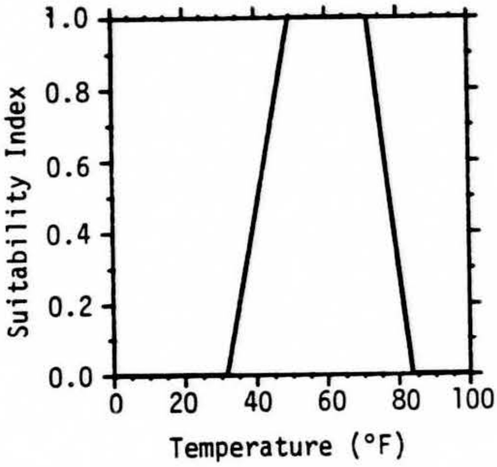


Figure 4. (concluded)

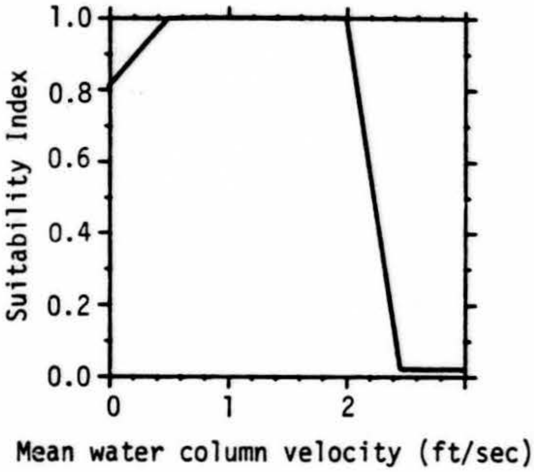
Juvenile temperature. There has been a great deal of variability among results of studies undertaken to determine temperature preferences of juvenile rainbow trout. Cherry et al. (1975) found that temperatures selected and avoided were a function of acclimation temperature; that rainbow juveniles selected temperatures ranging from 53 to 72° F when acclimated to 43 to 58° F; and that the lowest avoidance temperature was 41° F and the highest avoidance temperature was 77° F at the given acclimation temperatures. Coutant (1977) listed preferred temperatures of 64 to 66° F and 72° F, and avoidance temperatures of 57 and 72° F. Lee and Rinne (1980) found the critical thermal maxima for juveniles to be 84° F. McCauley et al. (1977) stated that acclimation temperatures had no significant effect upon preferred temperatures of juveniles, which ranged from 50 to 55° F. Kwain and McCauley (1978) found that temperature preferences were a function of age, and that juvenile rainbow trout 12 months after hatching preferred a temperature of 55° F. No studies were found which addressed maximum growth rate/low mortality temperatures. A final curve was drawn based on the limited information available and professional judgment (Fig. 4).

Adult. For the purposes of this model, rainbow trout are considered to be adult when they are greater than 7.9 inches in length at age II or III, and sexually mature (Carlander 1969).

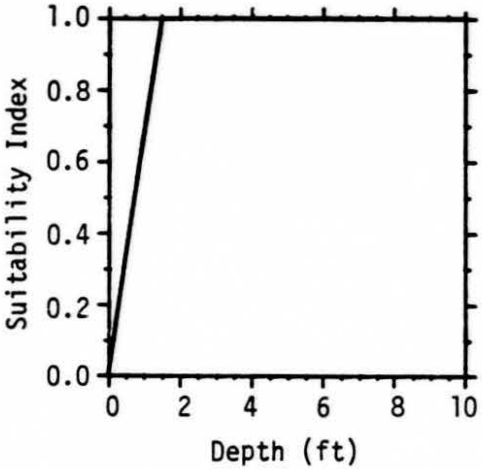
Adult velocity. Moyle et al. (1983) collected rainbow trout adults (which they defined to be > 4.7 inches in length) from Deer Creek (n=104; maximum velocity > 3.4 fps), Cherry and Eleanor Creeks (n=360; maximum velocity > 3.4 fps), and the Tuolumne River (n=93; maximum velocity = 2.4 fps). The resulting weighted normalized curve suggests that preferred mean column velocities are near 0.5 fps. Lewis (1969), however, found a positive correlation between rainbow trout density and water velocity of 1.65 fps. Gosse (1982) found that rainbow trout generally tended to reposition themselves in the water column as streamflow changed, and that fish nose velocity varied less than mean column velocity with changes in streamflow. Average fish nose velocities for stationary swimming during the winter ranged from 0.7 to 1.0 fps (n=640); during the summer they ranged from 0.9 to 1.1 fps (n=224); for random swimming during the winter they ranged from 0.5 to 0.7 fps (n=308); and during the summer they ranged from 0.4 to 0.6 fps. Average mean column velocities for stationary swimming during the winter ranged from 1.1 to 1.7 fps (n=606); during the summer they ranged from 1.5 to 2.0 fps (n=219); for random swimming during the winter they ranged from 0.6 to 0.8 fps (n=308); and during the summer they ranged from 0.6 to 0.7 fps (n=171). Therefore, the final curve (Fig. 5) reflects the range of mean water column velocities preferred by adult (> 5 to 9 inch lengths) rainbow trout, although preferred fish nose velocities ranged from 0.5 to 1.1 fps. An investigator may choose to develop new curves specific to the area of interest.

Coordinates

x	y
0.00	0.81
0.50	1.00
2.00	1.00
2.46	0.02
2.95	0.02
3.44	0.01
3.50	0.00
100.00	0.00



x	y
0.0	0.0
1.5	1.0
10.0	1.0
100.0	1.0



x	y
0.0	0.00
1.0	0.01
2.0	0.01
3.0	0.75
5.0	0.75
6.0	1.00
7.0	1.00
8.0	0.20
8.1	0.00
100.0	0.00

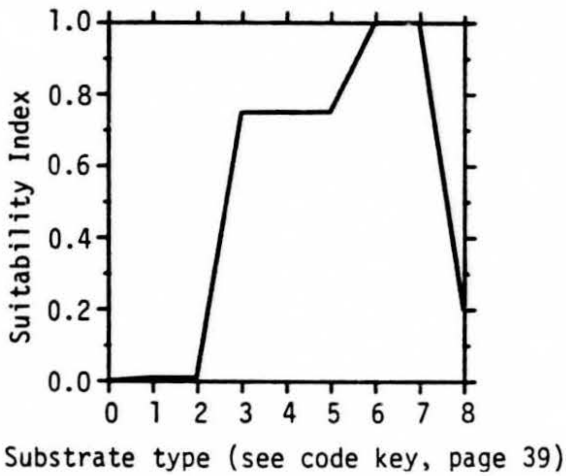


Figure 5. SI curves for rainbow trout adult velocity, depth, substrate, and temperature.

Coordinates	
x	y
0.0	0.0
32.0	0.0
55.4	1.0
70.0	1.0
84.2	0.0
100.0	0.0

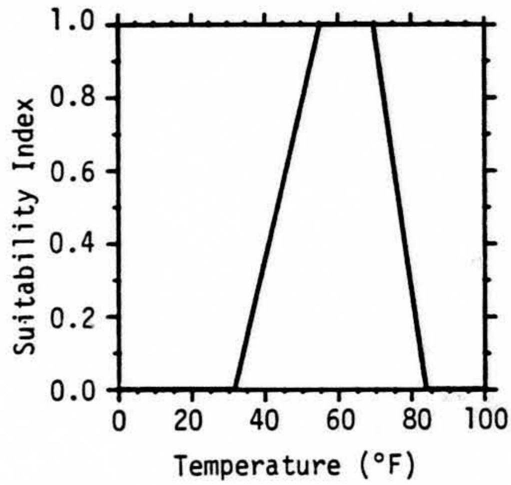


Figure 5. (concluded)

Adult depth. Depths most utilized (n=361) in Cherry and Eleanor Creeks ranged between 1.5 and 2.5 ft, whereas depths most utilized in the Tuolumne River ranged between 2.5 and 4.1 ft (Moyle et al. 1983). Adults in the Green River, however, primarily utilized depths ranging from 12 to 17 ft (Gosse 1982). Therefore, it may be assumed that the SI = 1.0 for all depths greater than 1.5 ft (Fig. 5), or curves may be developed that are specific to the area of investigation.

Adult substrate. Adults (> 4.7 inches) in Deer Creek (n=96) primarily utilized cobble; in Cherry Creek and Eleanor Creek, and the Tuolumne River (n=448) they primarily utilized boulders (Moyle et al. 1983). In the Green River, adults (> 9 to 10 inches in length) utilized cobble and boulders during stationary swimming; and silt, sand, and boulders during random swimming (Gosse 1982). The final curve (Fig. 5) is based on professional estimation.

Adult cover. Sufficient information was not located for the development of a curve for adult trout cover requirements. Lewis (1969) found a positive correlation between the amount of cover and adult density. Butler and Hawthorne (1968) found that rainbow trout had less affinity for cover than brook or brown trout. The investigator has several options when considering cover. Cover may be omitted as a model variable; cover curves may be developed independently; it may be assumed that cover is adequately addressed by substrate and depth; or the curve for V_6 (page 16) may be used to represent adult cover requirements.

Adult temperature. Preferred temperatures of rainbow trout adults have been found to be 55.4, 59.0, 61.7, 64.4, and 66.0 to 70.0° F (Coutant 1977; Spigarelli and Thommes 1979). Temperature selection may be a function of acclimation temperature, size of fish, and time of year. Lee and Rinne (1980) determined the critical thermal maxima at 84.2° F. The final curve (Fig. 5) is based on this information.

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16. Abstract (Limit: 200 words)

A review and synthesis of existing information was used to develop riverine and lacustrine habitat models for rainbow trout (Salmo gairdneri) a freshwater species. The models are scaled to produce indices of habitat suitability between 0 (unsuitable habitat) and 1 (optimally suitable habitat) for freshwater areas of the continental United States. Other habitat suitability models found in the literature are also included. Habitat suitability indices (HSI's) are designed for use with Habitat Evaluation Procedures previously developed by the U.S. Fish and Wildlife Service.

Also included are discussions of Suitability Index (SI) curves as used in the Instream Flow Incremental Methodology (IFIM) and SI curves available for an IFIM analysis of rainbow trout habitat.

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HABITAT SUITABILITY INDEX MODELS: COHO SALMON

by

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PREFACE

The habitat use information and Habitat Suitability Index (HSI) models presented in this document are intended for use in impact assessment and habitat management activities. Literature concerning a species, habitat requirements and preferences is reviewed and then synthesized into subjective HSI models, which are scaled to produce an index between 0 (unsuitable habitat) and 1 (optimal habitat). Assumptions used to transform habitat use information into these mathematical models are noted and guidelines for model application are described. Any models found in the literature which may also be used to calculate an HSI are cited. A section presenting Instream Flow Incremental Methodology (IFIM) will be included in this series in the near future. The IFIM section will include a discussion of Suitability Index (SI) curves, as are used in IFIM and a discussion of SI curves available for the IFIM analysis of coho salmon habitat.

Use of habitat information presented in this publication for impact assessment requires the setting of clear study objectives. Methods for modifying HSI models and recommended measurement techniques for model variables are presented in Terrell et al. (1982).[1] A discussion of HSI model building techniques is presented in U.S. Fish and Wildlife Service (1981).[2]

The HSI model presented herein is the combination of hypotheses of species-habitat relationships, not statements of proven cause and effect relationships. Results of model performance tests, when available, are referenced; however, models that have demonstrated reliability in specific situations may prove unreliable in others. For this reason, the U.S. Fish and Wildlife Service encourages model users to send comments and suggestions to help increase the utility and effectiveness of this habitat-based approach to incorporate the coho salmon in fish and wildlife planning. Please send comments to:

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[1]Terrell, J. W., T. E. McMahon, P. D. Inskip, R. F. Raleigh, and K. L. Williamson. 1982. Habitat suitability index models: Appendix A. Guidelines for riverine and lacustrine applications of fish HSI models with the Habitat Evaluation Procedures. U.S. Dept. Int., Fish Wildl. Serv. FWS/OBS-82/10.A. 54 pp.

[2]U.S. Fish and Wildlife Service. 1981. Standards for the development of habitat suitability index models. 103 ESM. U.S. Dept. Int., Fish Wildl. Serv., Div. Ecol. Serv. n.p.

CONTENTS

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COHO SALMON (Oncorhynchus kisutch)

HABITAT USE INFORMATION

General

The coho salmon (Oncorhynchus kisutch) is native to the northern Pacific Ocean, spawning and rearing in streams from Monterey Bay, California, to Point Hope, Alaska, and southward along the Asiatic coast to Japan. Its center of abundance in North America is from Oregon to Alaska (Briggs 1953; Godfrey 1965; Hart 1973; Scott and Crossman 1973). Coho salmon have been successfully introduced into the Great Lakes and reservoirs and lakes throughout the United States to provide put-and-grow sport fishing (Scott and Crossman 1973; Wigglesworth and Rawson 1974). No subspecies of coho salmon have been described (Godfrey 1965).

Age, Growth, and Food

Coho salmon typically return to spawn in freshwater at ages III or IV at lengths and weights ranging from 45 to 60 cm and 3.5 to 5.5 kg, respectively (Shapovalov and Taft 1954; Godfrey 1965; Scott and Crossman 1973). Coho from Alaska tend to be older and larger at spawning than those further south due to a longer period of freshwater residence (Drucker 1972; Crone and Bond 1976). A significant percentage of spawning runs, particularly in the southern portion of the coho's range, may consist of precocious males (jacks) that mature and return to spawn after only 6 to 9 months in the ocean (Shapovalov and Taft 1954).

Growth rate of coho during freshwater rearing is variable both between and within streams (Drucker 1972; Crone and Bond 1976) and is probably determined, to a large extent, by food availability and temperature. Size, as a function of growth, may play an important role in escapement and survival rate in coho populations; larger seaward migrant coho (smolts) have a higher probability of returning as adults and are larger and more fecund than smaller individuals of a cohort (Crone and Bond 1976; Bilton 1978).

Young coho feed mainly on drifting aquatic and terrestrial insects (Demory 1961; Mundie 1969; Scott and Crossman 1973). As they grow, coho become increasingly piscivorous, preying primarily on salmonid fry (Scott and Crossman 1973). In the ocean or in lakes and reservoirs, coho feed on fish and crustaceans (Grinols and Gill 1968; Hart 1973; Scott and Crossman 1973; Healey 1978). Coho do not feed during spawning migrations.

Reproduction

Coho salmon return to natal streams to spawn from midsummer to winter, depending on latitude. In the southern part of the range, spawning occurs in December and January (Briggs 1953; Shapovalov and Taft 1954). In Alaska, spawning occurs in October (Drucker 1972; Crone and Bond 1976) and, in the Great Lakes, in early September - October (Scott and Crossman 1973). Coho in North America migrate upstream during a single fall run, unlike other salmon, which may migrate upstream in multiple runs throughout the year (Scott and Crossman 1973). Entry into freshwater often coincides with rises in streamflow, particularly in streams with low summer flows (Shapovalov and Taft 1954).

Spawning behavior of coho has been summarized by Morrow (1980). Spawning occurs primarily in moderate-sized coastal streams and tributaries of larger rivers. Coho do not utilize main channels of large rivers for spawning as heavily as do chinook (O. tshawytscha) or intertidal reaches as heavily as do chum (O. keta) and pink (O. gorbuscha) salmon (Scott and Crossman 1973). Supplementation of declining runs of wild spawning stocks with hatchery fish is increasing in the Northwest (Fulton 1970; Korn 1977).

Incubation period varies inversely with temperature and usually lasts 35 to 50 days (Shapovalov and Taft 1954). Fry emerge 20 to 25 days after hatching (Mason 1976a).

Freshwater Residence

Coho fry emerge from the gravel from early March to mid-May. Newly emerged fry aggregate along stream margins, in shallow pools, and in backwaters and eddies (Lister and Genoe 1970; Stein et al. 1972). Fry gradually move into deeper pools, where they become aggressive and territorial. Fry unable to hold a territory emigrate downstream into the ocean (Hartman et al. 1982) or elsewhere in the stream system (Shapovalov and Taft 1954) because of intraspecific competition for food and space (Chapman 1966a). Coho that emigrate in their first spring or summer of life as age 0 fish [usually < 40 mm fork length (FL)] often constitute a major portion of the seaward migrants, but their probability of returning as adults is extremely low (Crone and Bond 1976; Hartman et al. 1982). Otto (1971) demonstrated that age 0 coho are poorly equipped physiologically to survive and grow in the high salinities encountered in the ocean.

Scales from returning adults indicate that the vast majority of coho reside in freshwater for at least 1 year prior to seaward migration. In the southern part of the range, coho commonly remain in freshwater for 1 to 2 years (Shapovalov and Taft 1954; Godfrey 1965). In Alaska, freshwater residence lasts from 2 to 4 years (Drucker 1972; Crone and Bond 1976).

Smoltification

Myriad processes and factors initiate, control, and affect parr-smolt transformation (smoltification) in coho and other anadromous salmonids. An important requirement of hatchery or naturally produced coho juveniles is that the resulting smolts be fully able, behaviorally and physiologically, to migrate to the sea, grow, develop normally, and return to their native stream and successfully spawn. Among the environmental factors that influence smoltification, photoperiod, temperature, and flow are especially critical (Parry 1960; Hoar 1965; Clarke et al. 1978; Clarke and Shelbourn 1980; Wedemeyer et al. 1980).

Smoltification and seaward migration in coho occurs in the spring (Shapovalov and Taft 1954; Drucker 1972; Crone and Bond 1976), with some exceptions (Chapman 1962). Migration often follows periods of rapid temperature warming (Shapovalov and Taft 1954). Coho smolts in California are reported to migrate to sea in April - May (Shapovalov and Taft 1954); in southeast Alaska, migration peaked in mid-June (Crone and Bond 1976).

Parr-smolt transformation is primarily a function of size, rather than of age. Minimum size for successful smoltification in coho is near 100 mm FL (Shapovalov and Taft 1954; Drucker 1972; Crone and Bond 1976). This size corresponds closely to the 90 mm threshold size of coho for maximum salinity tolerance (Conte et al. 1966). Smaller coho may show signs of transformation to smolts (e.g., silvery color, increased buoyancy, and salinity tolerance), but other critical aspects of the process are usually lacking (e.g., migratory behavior), and they do not develop fully until the threshold size is attained (Wedemeyer et al. 1980).

Due to the reduction in spawning habitat and spawning runs, natural reproduction of coho salmon is increasingly supplemented by release of hatchery-reared smolts. However, a perennial problem in the use of hatchery-produced juvenile salmonids is that ocean survival is often below estimated survival of naturally produced smolts. The failure to produce good quality smolts centers on the release of fish at a size, age, and time unsuitable for their ocean survival and on their exposure to environmental conditions that adversely affect growth and survival. Wedemeyer et al. (1980) have reviewed this problem in depth and propose guidelines for rearing and release of hatchery smolts to maximize the number returning as adults.

Specific Habitat Requirements

Coho salmon utilize a variety of freshwater habitats and tolerances and requirements change with season and age. Although most developmental changes and movements to different habitats are gradual, it is useful to delineate the freshwater life cycle into four distinct life stages and to specify factors assumed to affect habitat quality for each life stage. These life stages are defined as follows:

1. Adult. Sexually mature coho migrating from the ocean to natal stream to spawn.

2. Spawning/embryo/alevin. From period of egg deposition to hatching and emergence of fry from redds (Alevins = yolk-sac fry).

3. Parr. Fry (age 0) and juvenile (age I+) coho residing in rearing streams.

4. Smolt. Seaward migrant juveniles undergoing parr-smolt transformation.

Adult. Accessibility of the spawning stream and water quality appear to be the major factors affecting coho during upstream migration. Dams may completely block upstream passage, and other physical features may become impossible to cross at low (e.g., debris jams or waterfalls) or high (e.g., excessive velocities) flows (Reiser and Bjornn 1979). Thompson (1972) recommended a minimum depth of 0.18 m and a maximum velocity of 244 cm/sec as criteria for successful upstream migration of adult coho.

Water quality can affect upstream migration of coho through direct mortality, increasing the susceptibility of the coho to diseases, or adversely altering the timing of the migration and rate of maturation (Holt et al. 1975). Temperatures ≥ 25.5° are lethal to migrating adults (Bell 1973). Sublethal temperatures may result in major prespawning mortalities through activation of latent infections (Wedemeyer 1970). Disease infection rates in coho increase markedly at temperatures above 12.7° C (Fryer and Pilcher 1974; Holt et al. 1975; Groberg et al. 1978). Temperatures ≤ 13° C have been recommended to minimize prespawning mortality of coho during upstream migration (Wedemeyer, pers. comm.).

Dissolved oxygen (D.O.) levels > 6.3 mg/l are recommended for successful upstream migration of anadromous salmonids (Davis 1975). Lower D.O. concentrations adversely affect upstream migration by reducing the swimming ability of migrants and by eliciting avoidance responses. Maximum sustained swimming speed of coho is sharply reduced at D.O. levels < 6.5 mg/l at all temperatures (Davis et al. 1963). It is assumed that adult coho respond to low D.O. levels in a fashion similar to juveniles and avoid waters with D.O. concentrations < 4.5 mg/l (Whitmore et al. 1960).

Spawning/embryo/alevin. Coho salmon construct redds in swift, shallow areas at the head of riffles (Burner 1951; Briggs 1953; Shapovalov and Taft 1954). Preferred redd construction sites in riffle areas have velocities of 21 to 70 cm/sec and minimum depths ≥ 15 cm (Smith 1973). Gravel and small rubble substrate with low amounts of fine sediments is optimum for survival, growth, and development of embryos and alevins and for later emergence of fry (Platts et al. 1979). Percent composition of various size classes of substrate resulting in high survival of embryos and alevins has not been established. Reiser and Bjornn (1979) estimated that redds with 1.3 to 10.2 cm diameter substrate sizes and a low percentage of fines result in high survival of embryos. An inverse relationship between percent fines < 3.3 mm and emergence of fry has been well established in field (Koski 1966; Hall and Lantz 1969; Cloern 1976) and laboratory (Phillips et al. 1975) experiments. In all studies, emergence of coho fry was high at < 5% fines but dropped sharply at ≥ 15% fines.

Survival and emergence of embryos and alevins is greatly influenced by D.O. supply within the redd (Mason 1976a). D.O. concentrations ≥ 8 mg/l are required for high survival and emergence of fry. Embryo survival drops significantly at levels ≤ 6.5 mg/l; concentrations < 3 mg/l are lethal (Coble 1961; Shumway et al. 1964; Davis 1975). D.O. supply available to coho in redds is determined primarily by the interrelationship of gravel permeability, water velocity, and D.O. concentration. When any of these factors, acting alone or in combination, reduces the intragravel O_2 supply below saturation, hypoxial stress occurs, resulting in delayed hatching and emergence, smaller size of emerging fry, and increased incidence of developmental abnormalities (Alderice et al. 1958; Coble 1961; Silver et al. 1963; Shumway et al. 1964; Mason 1976a). D.O. concentrations at or near saturation, with temporary reductions no lower than 5 mg/l, are recommended as criteria necessary for successful reproduction of anadromous salmonids (Reiser and Bjornn 1979).

Burner (1951) observed coho spawning in Oregon at temperatures of 2.5 to 12.0° C. Temperatures of 4.4 to 9.4° C are considered suitable for spawning (Bell 1973). Temperatures in the 4.4 to 13.3° C range are considered optimum for embryo incubation; survival decreases if these thresholds are exceeded (Bell 1973; Reiser and Bjornn 1979).

Parr. Coho parr require an abundance of food and cover to sustain fast growth rates, avoid predation, and avoid premature displacement downstream to the ocean in order to successfully rear in freshwater and migrate to the sea as smolts (Mundie 1969). Mason and Chapman (1965) found that the number of coho parr remaining in stream channels is dependent on the amount of food and cover available; if food or cover is decreased, emigration from the area subsequently is increased. Mason (1976b) substantially increased summer carrying capacity of a coho stream by supplemental feeding; however, these gains were largely lost because numbers exceeded winter carrying capacity. Dill et al. (1981) found that territory size in coho is inversely related to the amount of available food. Low levels of food result in larger and fewer territories per unit area, increased emigration of resident fry, and slower growth rate of remaining fish. Small, slow growing parr may remain in freshwater for longer periods (with an attendant high mortality rate) until threshold size for smolting is reached or may migrate to the sea at a time when chances for survival are slim (Chapman 1966a).

Substrate composition, riffles, and riparian vegetation appear to be the most important factors influencing production of aquatic and terrestrial insects as food for coho (Mundie 1969; Giger 1973; Reiser and Bjornn 1979). Highest production of aquatic invertebrates is found in stream substrates comprised of gravel and rubble (Giger 1973; Reiser and Bjornn 1979). Pennak and Van Gerpen (1947) reported that the production of benthic invertebrates is greater in rubble > bedrock > gravel > sand. Because substrate size is a function of water velocity, larger substrate sizes are associated with faster currents. Thus, food production is also high in riffles (Ruggles 1966; Waters 1969). Pearson et al. (1970) found that coho production per unit area in Oregon streams is higher in pools with larger riffles upstream. However, increased fines in riffles can reduce production of benthic food organisms (Phillips 1971). Crouse et al. (1981) reported that coho production is lowest in laboratory stream channels when embeddedness of the rubble substrate is high (80 to 100%) and the percent (by volume) of fines (≤ 2.0 mm) exceeds 26%. Lastly, riparian vegetation along coho streams acts as habitat for terrestrial insects, as well as a source of leaf litter utilized by stream invertebrates as food (Chapman 1966b; Mundie 1969).

Coho parr are most abundant in large, deep [generally > 0.30 m (Nickelson, pers. comm.)] pools, where they congregate near instream and bank (overhead) cover of logs, roots, debris, undercut banks, and overhanging vegetation (Ruggles 1966; Lister and Genoe 1970; Mason 1976b). Nickelson and Reisenbichler (1977) and Nickelson et al. (1979) found positive correlations between standing crop of age 0+ coho and pool volume. Studies in Oregon by Nickelson (pers. comm.) suggest that pools of 10 to 80 m^3 or 50 to 250 m^2 in size with sufficient riparian canopy for shading are optimum for coho production. A pool to riffle ratio of 1:1 provides optimum food and cover conditions for coho parr. Ruggles (1966) found that the greatest number of coho fry remained in stream channels consisting of 50% pools and 50% riffles; numbers of fry remaining in channels of either 100% pools or 100% riffles could be 39% and 20% lower, respectively.

As water temperatures decrease below 9° C, coho fry become less active and seek deep (≥ 45 cm), slow (< 15 cm/sec) water in or very near (< 1 m) dense cover of roots, logs, and flooded brush (Hartman 1965; Bustard and Narver 1975a). Beaver ponds and quiet backwater areas, often some distance from the main stream channel and dry during summer low flow periods, are also utilized as winter habitat (Narver 1978). Several studies indicate that the amount of suitable winter habitat may be a major factor limiting coho production (Chapman 1966a; Mason 1976b; Chapman and Knudsen 1980). Swimming ability of coho is decreased as the water temperature drops; therefore, winter cover is critical for protection from predation, freezing, and, especially, displacement by winter freshets (Bustard and Narver 1975b; Mason 1976b; Hartman et al. 1982). Chapman and Knudsen (1980) found a very low winter biomass of coho in channelized and grazed sections of streams in Washington, which they attributed to the reduced pool volumes and amount of instream and bank cover present in those areas.

Several studies have shown a positive relationship between stream carrying capacity for coho and streamflow (McKernan et al. 1950; Mathews and Olson 1980; Scarnecchia 1981). Strong positive correlations have also been found between total stream area and measures of coho biomass (Pearson et al. 1970; Burns 1971). Lowest returns of adult coho coincide with low summer flows coupled with high winter floods (McKernan et al. 1950). Burns (1971) found that highest mortality of coho and other salmonids in the summer occurred during periods of lowest flows. Higher streamflows during rearing appear to provide more suitable habitat for growth and survival through increased production of stream invertebrates and availability of cover (Chapman 1966a; Giger 1973; Scarnecchia 1981). Stabilization of winter flows and increases in summer flows have led to increased production of coho (Lister and Walker 1966; Mundie 1969). Narver (1978) suggested that stream enhancement techniques aimed at reducing displacement downstream during winter floods and at providing deep pools during summer low flows could substantially increase stream rearing capacity for coho.

Growth rate and food conversion efficiency of coho fry is optimum at D.O. concentrations above 5 mg/l. Below 4.5 mg/l, growth and food conversion rapidly decreases to the point where growth ceases or is negative (below 3 mg/l) (Herrmann et al. 1962; Brett and Blackburn 1981). Swimming speed decreases below the saturation level, especially below 6 mg/l (Dahlberg et al. 1968). D.O. concentrations < 4.5 mg/l are avoided (Whitmore et al. 1960). Upper incipient lethal temperatures for coho fry range from 22.9 to 25.0° C (acclimation temperatures of 5 to 23° C) (Brett 1952). Significant decreases in swimming speed occur at temperatures > 20° C (Griffiths and Alderice 1972), and growth ceases at temperatures above 20.3° C (Bell 1973). Stein et al. (1972) found that the growth rate of coho fry was high in the 9 to 13° C temperature range, but slowed considerably at temperatures near 18° C. Brungs and Jones (1977) reported that growth of coho occurred from 5 to 17° C.

Streamside vegetation plays an important role in regulating the temperature in rearing streams. Cooler winter water temperatures may occur if the stream canopy is absent or reduced, adversely affecting egg incubation (Chapman 1962). Where streamside vegetation is intact but the surrounding watershed has been logged, warmer winter water temperatures may result, shifting the period of emergence of fry and downstream movement of smolts to earlier, and less favorable, periods (Hartman et al. 1982). In areas where the stream canopy has been reduced, the resultant warmer summer temperatures may make the habitat unsuitable if the temperature exceeds 20° C (Stein et al. 1972) or may increase the mortality of fry from disease (Hall and Lantz 1969). However, too much stream canopy can also reduce habitat suitability for coho fry. For example, Chapman and Knudsen (1980) found reduced coho biomass in stream sections where the canopy was very dense. Pearson et al. (1970) reported that coho fry appear to avoid areas of dense shade; they suggested that stream canopy enclosing > 90% of the sky may exceed the optimum level.

In summary, optimum rearing habitat for coho parr consists of a mixture of pools and riffles, abundant instream and bank cover, water temperatures that average between 10 to 15° C in the summer, D.O. near the saturation level, and riffles with low amounts of fine sediment (Reiser and Bjornn 1979). Streamside vegetation is an important component of coho habitat because it provides food, cover, temperature control, and bank stabilization (Narver 1978).

Smolt. The radical physiological and behavioral changes that occur during smoltification make this stage particularly sensitive to environmental stress factors. Blockage and delay of migration by dams, unfavorable stream flows and temperatures, fluctuations in food supplies, predation, gas supersaturation below dams, activation of latent infections due to environmental stress, interference with saltwater adaptation in estuaries because of gill infestations, and handling stress and descaling during transportation around dams are major sources of mortality and reduced ocean survivability of coho smolts (Wedemeyer et al. 1980).

Elevated water temperatures can accelerate the onset of smoltification and shorten the smolting period and may result in seaward migration of smolts at a time when conditions are unfavorable (Wedemeyer et al. 1980). Zaugg and McLain (1976) reported that the period of high gill ATPase activity (indicative of high salinity tolerance and other adaptations necessary for parr-smolt transformation) in coho smolts held at 20° C occurred from mid-March to early April; at 15° C, it occurred from mid-March to early May; and, at 10° C, a normal pattern resulted with a peak in ATPase activity from mid-March to early July. By shortening the duration of smolting and accelerating desmoltification, sublethal temperatures can lead to parr-reversion of coho smolts in estuaries where exposure to predation and risk of infection is high, thereby diminishing the number of coho smolts entering the ocean (Wedemeyer et al. 1980). Wedemeyer et al. (1980) recommend that temperatures follow a natural seasonal cycle as closely as possible to those present in the coho's native range to ensure optimum conditions for smoltification and timing of seaward migration. Specifically, temperatures should not exceed 10° C in late winter to prevent accelerated smolting; temperatures should not exceed 12° C during smolting and seaward migration in the spring to prevent shortened duration of smolting and premature onset of desmoltification and to reduce the risk of infection from pathogens (see Adult section).

Exposure to pollutants can have a major deleterious impact on smoltification and early marine survival of anadromous salmonids (see review by Wedemeyer et al. 1980). For example, Lorz and McPherson (1976) found that, at very low levels of copper (20 to 30 µg/l), migratory behavior and gill ATPase activity in coho smolts was greatly suppressed and high mortalities resulted from exposure to saltwater. Low concentrations of herbicides have also been found to inhibit smolt function and migratory behavior (Lorz et al. 1978).

The lethal threshold for gas supersaturation in coho smolts is 114.5%. No deaths were reported at 110% supersaturation, but the majority of fish exhibited symptoms of gas-bubble disease (Rucker and Kangas 1974; Nebeker and Brett 1976).

Specific D.O. requirements for coho smolts are unknown, but are probably similar to those for parr.

HABITAT SUITABILITY INDEX (HSI) MODEL

Model Applicability

Geographic area. The model was developed from information gathered on habitat requirements of coho salmon throughout its native and introduced range. This general model is designed to be applicable to all the above areas but is limited to the freshwater stage of the life cycle: upstream migrant; embryo; parr; and smolt.

Season. The model is structured to account for changes in seasonal as well as life stage requirements of coho salmon during those parts of the life cycle when they inhabit freshwater. Because rearing streams are utilized year-round, the model is developed to measure the suitability of a given habitat to support parr for the entire year and to support embryos during the spawning and incubation period.

Cover types. The model is oriented primarily to small coastal streams and tributaries of larger rivers, which are the major spawning and rearing areas of coho salmon. Habitat requirements of coho in large rivers, where some spawning and rearing occurs and which serve as "highways to the sea" for upstream and downstream migrant wild and hatchery-reared coho, are less well-known and are not adequately addressed in this model. Water quality variables are the only variables in this model that may be applicable when coho inhabit large rivers. Variables that measure habitat suitability for adult coho in lakes, reservoirs, estuaries, or the ocean are not included in this model.

Water quality. The model has limited utility in areas where water quality variables (e.g., toxic substances and gas supersaturation) are major factors limiting coho populations. If toxic substances are being discharged into a river, Wedemeyer et al. (1980) should be consulted for information on the types of substances that can adversely affect survival of smolts.

Verification level. The model represents the author's interpretation of how specific environmental factors combine to determine overall habitat suitability for coho salmon. The model has not been field tested.

Model Description

The HSI model that follows is an attempt to condense information on habitat requirements for coho into a set of habitat evaluation criteria, structured to produce an index of overall habitat quality. A positive relationship between HSI and carrying capacity of the habitat is assumed (U.S. Fish and Wildlife Service 1981), but this relationship has not been tested.

As a consequence of their homing to natal streams to spawn, coho and other anadromous salmonids commonly form local races and stocks, exhibiting adaptations to the particular set of environmental conditions present in the spawning streams (Larkin 1981; MacLean and Evans 1981). The generalized HSI model presented does not take into account the different stocks or subpopulations. The model was developed, and should be applied, with the following statement by Banks (1969:131) in mind: "... the consequences of man-made changes (on anadromous salmonids) ...can be predicted in general terms from the existing literature, but (due to the formation of local stocks) each situation is unique ... and requires studies of the special needs of each river system as well as the flexible application of general principles".

The model consists of those habitat variables that affect the growth, survival, abundance, distribution, behavior, or other measure of well-being of coho, and therefore can be expected to have an impact on the carrying capacity of a habitat. Coho salmon habitat quality, in this model, is based on parameters assumed to affect habitat suitability for each of four life stages of coho salmon during residence in freshwater (Fig. 1). Variables affecting habitat suitability for parr are further delineated into the life requisite components of: water quality; food; and cover. It was assumed that the most limiting factor (i.e., lowest SI score) defines the carrying capacity for coho salmon; thus,

HSI = minimum value for suitability indices V_1 to V_{15}.

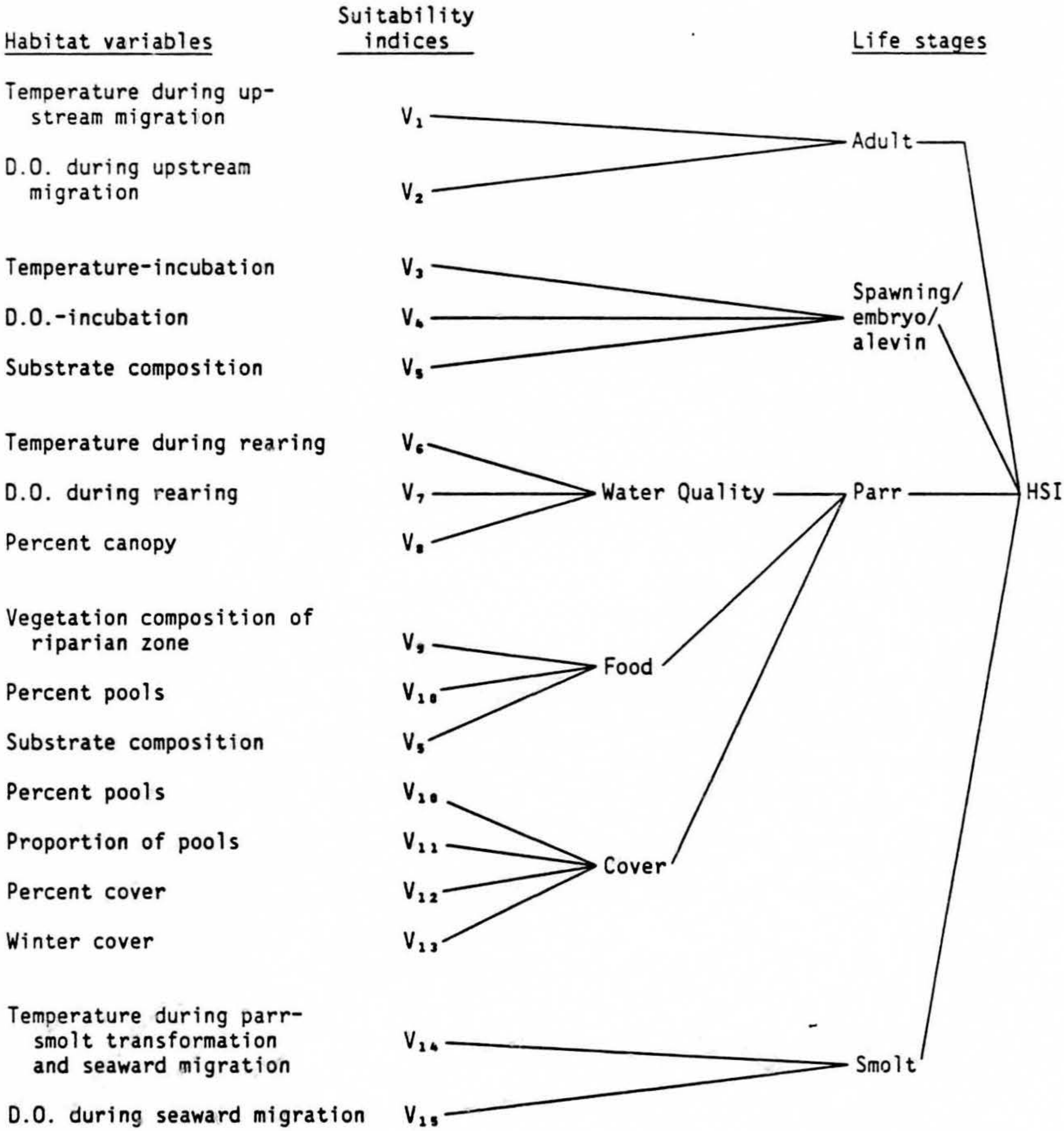


Figure 1. Diagram showing habitat variables included in the HSI model for coho salmon and the aggregation of the corresponding suitability indices (SI's) into an HSI. HSI = the lowest of the fifteen suitability index ratings.

Adult component. V_1 was included in this component because temperature can result in direct mortality, can increase coho susceptibility to infectious diseases, or can alter the timing of migration and rate of maturation of coho salmon during migration from the ocean to the spawning stream. Because D.O. levels below saturation can elicit avoidance behavior and reduce the swimming ability in coho, D.O. (V_2) also was included as a variable that affects habitat suitability for upstream migrants.

No specific variables were included in this component as measures of the accessibility of the spawning stream. Nevertheless, physical features encountered by coho while migrating upstream should be considered when evaluating habitat suitability. Features that impede or delay migrants from moving upstream (see Adult section) would make suitable habitat, as defined by the model, less useable.

Spawning/embryo/alevin component. V_3 was included in this component because embryo survival decreases when temperatures during incubation exceed the optimum temperature boundary of 13.3° C. V_4 was included because D.O. levels below the saturation level induce hypoxial stress in embryos and alevins and lead to decreased quantity and quality of emerging fry. V_5 was included because percent emergence of fry is related to substrate composition of spawning redds.

Parr component. Water quality: V_6 was included because temperature affects swimming speed, growth, and survival of coho parr. V_7 was included because D.O. concentration affects growth, food conversion, swimming speed, and avoidance behavior of parr. V_8 was included because coho numbers (or biomass) are related to the quantity of stream canopy cover.

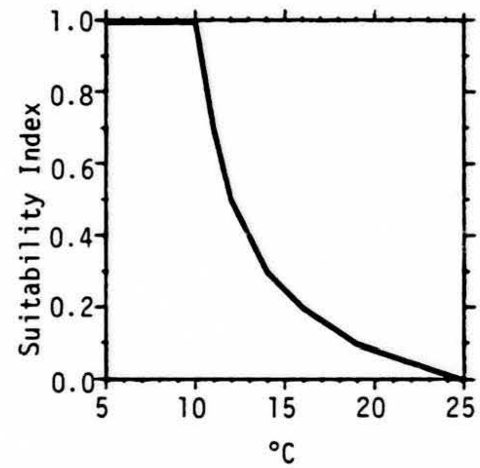
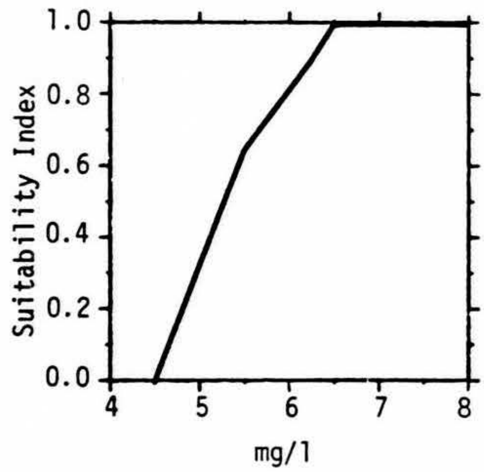
Food: V_9 was included because it was assumed that the direct (terrestrial insects) and indirect (leaf litter as food for aquatic insects) production of food utilized by coho parr varies with the amount and type of riparian vegetation present. V_{10} was included because the production of aquatic insects, as well as coho parr, has been related to the amount of riffle areas present in a stream. V_5 was included because the production potential of aquatic insects is related to the substrate composition.

Cover: V_{10} and V_{11} were included because the abundance of coho parr varies with the amount (V_{10}) and type (V_{11}) of pools present in a stream. V_{12} was included because coho parr are commonly associated with instream and bank cover. V_{13} was included because the amount of suitable winter cover may be a major factor affecting coho production.

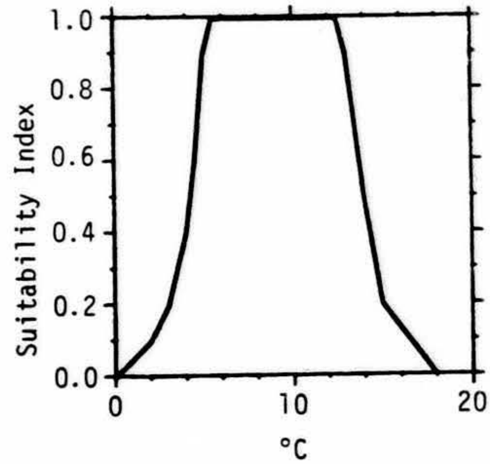
Smolt component. V_{14} was included because temperature greatly affects the timing and duration of parr-smolt transformation, can alter the timing of seaward migration, and can affect the susceptibility of smolts to infection. Although specific data are lacking, V_{15} was included because D.O. concentration could potentially impact smolt migration through its effects on swimming ability, by eliciting avoidance behavior, or by resulting in the direct mortality of smolts.

Suitability Index (SI) Graphs for Model Variables

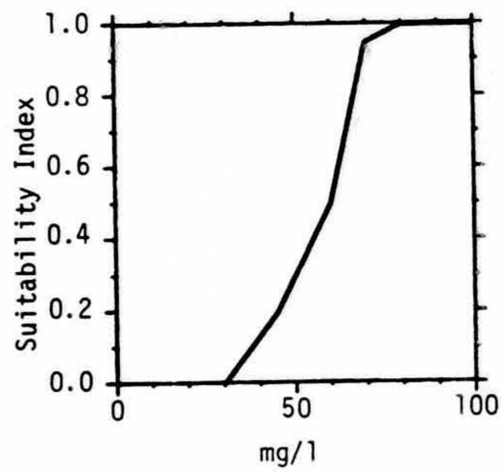
All variables pertain to riverine (R) habitat. Table 1 lists the information sources and assumptions used in constructing each SI graph.

Habitat	Variable		Suitability graph
R	V_1	Maximum temperature during upstream migration.	
R	V_2	Minimum dissolved oxygen concentration during upstream migration.	

R V$_3$ Maximum temperature from spawning to emergence of fry.



R V$_4$ Minimum dissolved oxygen concentration from spawning to emergence of fry.

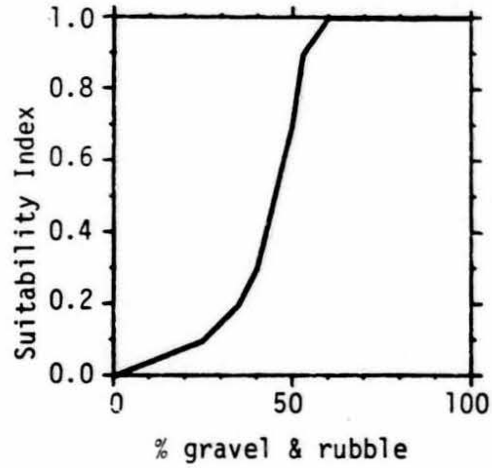


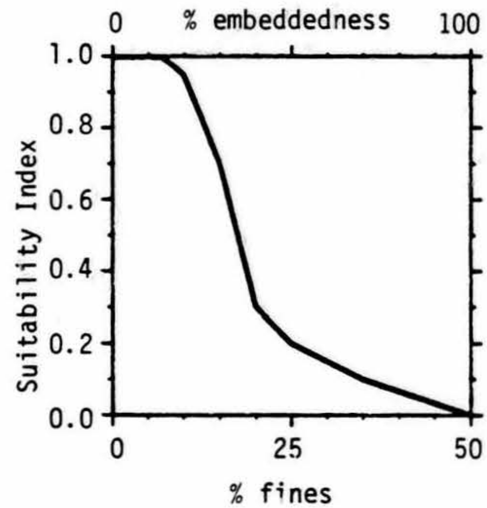
R V_5 Substrate composition in riffle/run areas.

A. Percent of gravel (10 to 60 mm) and rubble (61 to 250 mm) present.

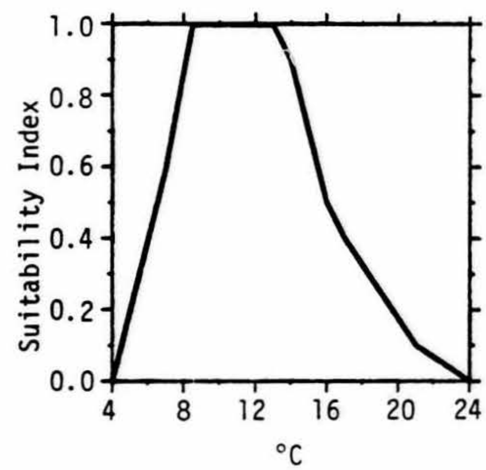
B. Percent fines (< 6 mm) or percent embeddedness of substrate.

$SI = \frac{A + B}{2}$, where B = % fines or % embeddedness, whichever is lower.

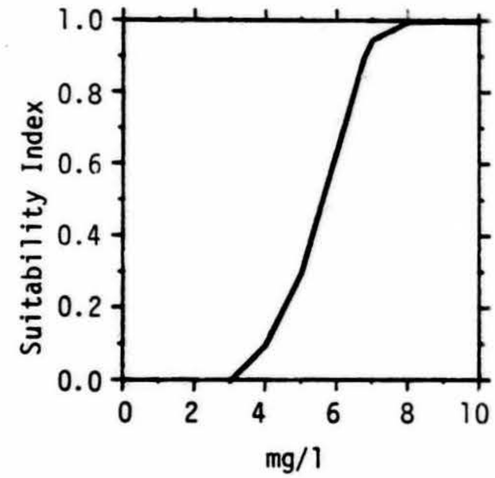




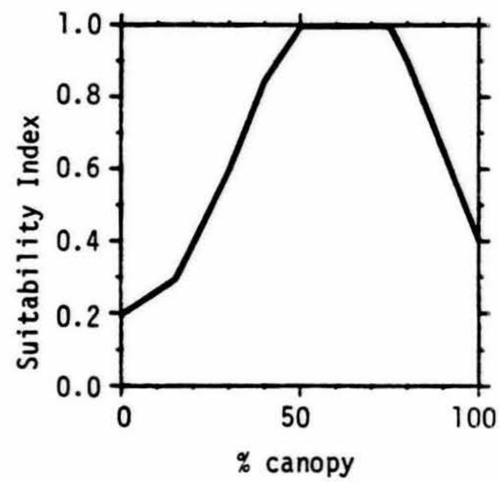
R V_6 Maximum temperature during rearing (parr).



R V_7 Minimum dissolved oxygen concentration during rearing (parr).

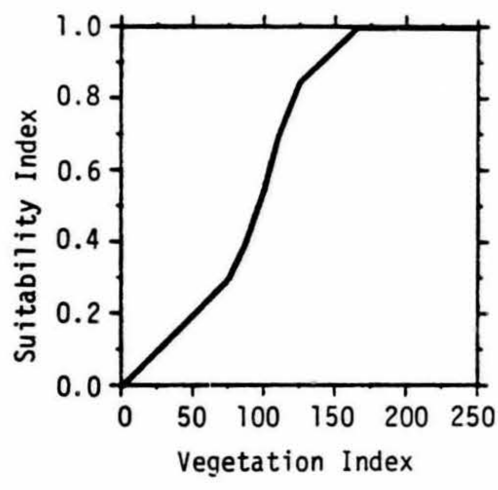


R V_8 Percent vegetative canopy over rearing stream.

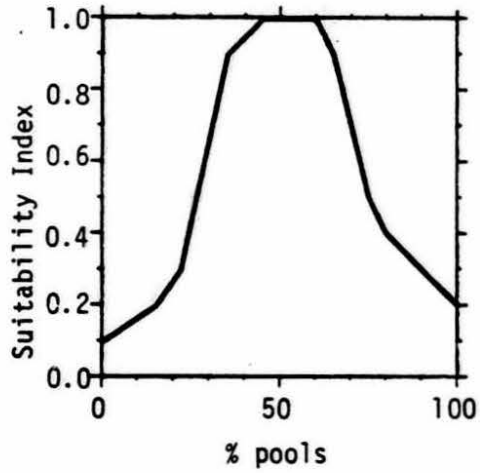


R V_9 Vegetation index of riparian zone during summer.

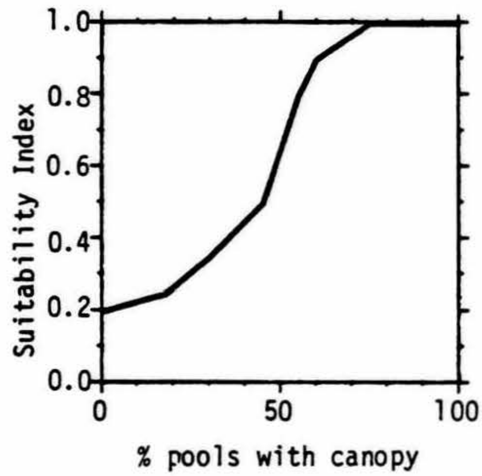
Vegetation Index = 2 (% canopy cover of deciduous trees and shrubs) + (% canopy cover of grasses and forbs) + (% canopy cover of conifers). For measurement techniques, see Terrell et al. (1982), p. A.19 and A.37.



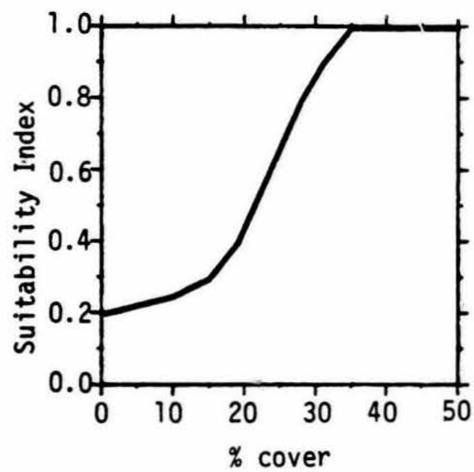
R V_{10} Percent pools during summer low flow period.



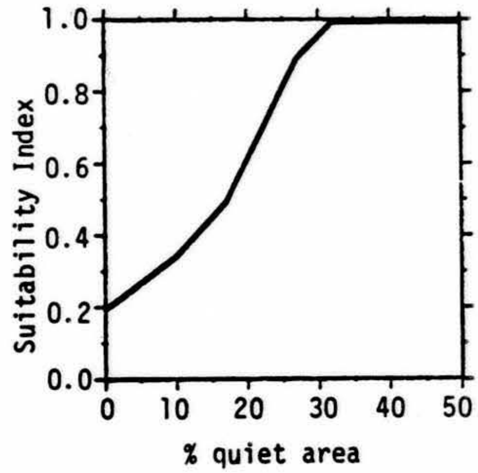
R V_{11} Proportion of pools during summer low flow period that are 10 to 80 m^3 or 50 to 250 m^2 in size and have sufficient riparian canopy to provide shade.



R V_{12} Percent instream and bank cover present during summer low flow period.

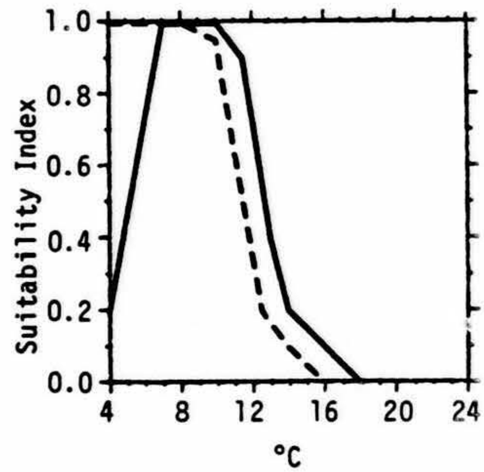


R V_{13} Percent of total area consisting of quiet backwaters and deep (≥ 45 cm) pools with dense cover of roots, logs, debris jams, flooded brush, or deeply-undercut banks during winter.



R V_{14} Maximum temperature during (A) winter (Nov.-March) in rearing streams and (B) spring-early summer (April-July) in streams where seaward migration of smolt occurs.

A. - - - -
B. ————



R V_{15} Minimum dissolved oxygen concentration during April-July in streams where seaward migration occurs.

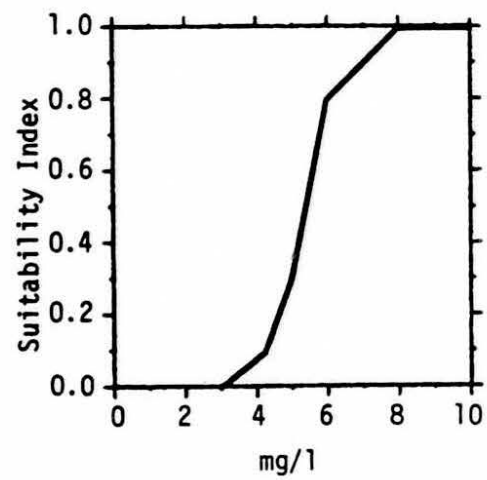


Table 1. Sources of information and assumptions used in construction of the suitability index graphs are listed below. "Excellent" habitat for coho salmon was assumed to correspond to an SI of 0.8 to 1.0, "good" habitat to an SI of 0.5 to 0.7, "fair" habitat to an SI of 0.2 to 0.4, and "poor" habitat to an SI of 0.0 to 0.1.

Variable	Assumptions and sources
V_1	Temperatures that are lethal or that correspond to high mortality rates in infected coho are poor (Bell 1973; Fryer and Pilcher 1974; Holt et al. 1975). Temperatures where mortality of infected coho is moderate or where activation of latent infections begins to increase are fair (Fryer and Pilcher 1974; Groberg et al. 1978). Temperatures that correspond to low disease mortality (Fryer and Pilcher 1974; Holt et al. 1975) and that are recommended for minimizing prespawning mortality are excellent (Wedemeyer pers. comm.).
V_2	D.O. levels that correspond to undiminished swimming ability (Davis et al. 1963) and that are recommended for successful upstream migration (Davis 1975) are excellent. Levels where swimming speed is greatly reduced (Davis et al. 1963) and avoidance is high (Whitmore et al. 1960) are poor.
V_3	Temperature ranges corresponding to those recommended as optimum for spawning and for incubation of embryos (Bell 1973) are excellent. Temperatures outside of this range are less suitable.
V_4	D.O. levels at or near the saturation level corresponded to the highest survival and emergence of fry and, therefore, are excellent. Levels that correspond to reduced emergence, delays in hatching or emergence, smaller size of fry, or increased incidences of developmental abnormalities (Alderice et al. 1958; Cobel 1961; Silver et al. 1963; Shumway et al. 1964; Mason 1976a) are fair. D.O. levels below 5 mg/l (Reiser and Bjornn 1979) or that approach lethal conditions (3 mg/l) (Coble 1961; Shumway et al. 1964; Davis 1975) are poor.

Table 1. (continued).

Variable	Assumptions and sources
V_5	(Embryo) Substrate composition that corresponds to high embryo survival and high emergence of fry is excellent. Compositions that contribute to reduced emergence (high percentage of fines, high embeddedness) are good-poor depending on the severity of the impact on survival and emergence (Koski 1966; Hall and Lantz 1969; Phillips et al. 1975; Cloern 1976; Platts et al. 1979; Reiser and Bjornn 1979). (Parr-Food) Gravel-rubble substrate composition corresponds to a high production of aquatic invertebrates (Giger 1973; Reiser and Bjornn 1979) and, therefore, is excellent in providing food for coho. Other substrates produce decreasing amounts of invertebrates in this order: rubble > bedrock > gravel > sand (Pennak and Van Gerpen 1947). It is assumed that the higher the percentage fines or percent embeddedness, the lower the production of aquatic invertebrates (Phillips 1971; Crouse et al. 1981).
V_6	Temperatures that correspond to high growth (9 to 13° C) (Stein et al. 1972) are excellent. Temperatures that correspond to reduced growth (Stein et al. 1972) are fair. Temperatures that are lethal or where growth of parr ceases are poor.
V_7	D.O. levels that correspond to the highest growth and food conversion rates (Herrmann et al. 1962; Brett and Blackburn 1981) are excellent. Levels that correspond to greatly reduced swimming speed (Dahlberg et al. 1968), avoidance behavior (Whitmore et al. 1960), and cessation of growth are poor.
V_8	It is assumed that 50 to 75% canopy enclosure is excellent. Other percentages are less suitable because cooler winter and warmer summer temperatures, associated with low canopy cover, result in decreased survival of embryos and fry (Chapman 1962; Hall and Lantz 1969; Stein et al. 1972). Lower biomass of coho corresponds to a high percent (> 90%) of canopy closure (Pearson et al. 1970; Chapman and Knudson 1980), so percentages ≥ 90% are fair.
V_9	Based on the work of Chapman (1966b), deciduous trees and shrubs are excellent as habitat for terrestrial insects and in providing high amounts of leaf litter used as food for aquatic invertebrates. Grasses/forbs and conifers are less suitable. The equation was formulated so that no riparian vegetation rates poor and so that ≥ 75% deciduous trees and shrubs rates excellent. It was based on the assumption that deciduous trees and shrubs provide twice the amount of terrestrial insects and leaf litter per unit area as do grasses/forbs and conifers.

Table 1. (continued)

Variable	Assumptions and sources
V_{10}	(Food-Cover) A pool to riffle ratio of 1:1 in streams is excellent in providing both food and cover for coho parr because: (1) food production is highest in riffles (Ruggles 1966; Waters 1969); (2) coho fry are most abundant in pools (Ruggles 1966; Lister and Genoe 1970; Mason 1976b); and (3) the highest number of coho fry remained in stream channels with a 1:1 ratio (Ruggles 1966). Higher or lower percentages of pools are less suitable because fewer coho fry remain in the stream channels (Ruggles 1966). This variable should be measured during summer low flow because this is the critical summer period for parr (Burns 1971).
V_{11}	The graph is based on studies on Oregon streams by Nickelson and colleagues where: (1) positive correlations were found between standing crop of age 0+ coho and pool volume (Nickelson and Reisenbichler 1977; Nickelson et al. 1979); and (2) coho fry biomass was highest in pools 10 to 80 m^3 or 50 to 250 m^2 in size (Nickelson pers. comm.). It is assumed that a positive relationship exists between proportion of pools 10 to 80 m^3 or 50 to 250 m^2 in size and habitat suitability (= carrying capacity) for coho fry. If such pools are absent from the reach, it is assumed that some other pool habitat would exist but would be poor, capable of supporting parr in relatively small numbers (therefore, SI = 0.2 at 0%).
V_{12}	Because there is a positive relationship between number of coho parr remaining in an area and amount of instream cover (Mason and Chapman 1965) and, because parr are most abundant near instream and bank cover (Ruggles 1966; Lister and Genoe 1970; Mason 1976b), it is assumed that habitat suitability is proportional to the amount of instream or bank cover present in a reach. Zero percent cover is assigned an SI of 0.2 because the stream may still be able to support coho parr, although at a greatly reduced level.
V_{13}	It is assumed that quiet backwaters and deep pools with dense cover are excellent winter habitat for coho parr because parr are most abundant in these areas during the winter (Hartman 1965; Bustard and Narver 1975a). Because several studies infer that the amount of suitable winter habitat may be a major factor limiting rearing capacity and smolt production (Chapman 1966a; Mason 1976b; Chapman and Knudsen 1980), it is assumed that habitat suitability is proportional to the amount of suitable winter habitat available. Zero percent winter cover has an SI rating of 0.2 because it is assumed that other potential sites can still support some overwintering parr. Thirty percent and above has an SI of 1.0, because it is assumed that optimum values of this variable are obtainable in conjunction with optimum riffle-pool ratios (V_{10}).

Table 1. (concluded).

Variable	Assumptions and sources
V_{14}	Temperatures that correspond to a long and normal pattern of gill ATPase activity during smoltification (Zaugg and McLain 1976) are excellent, as are temperatures recommended for optimum smoltification and timing of seaward migration; i.e., ≤ 10° C during winter and ≤ 12° C during spring (Wedemeyer et al. 1980; Wedemeyer pers. comm.). It is assumed that the shorter the duration of gill ATPase activity, the less suitable the temperature. Also, temperatures > 12° C are considered fair-poor because the risk of infections from pathogens is assumed to be higher than at lower temperatures (Fryer and Pilcher 1974; Holt et al. 1975).
V_{15}	It is assumed that D.O. requirements for smolts are similar to those of parr, thus the same assumptions and sources used in developing the D.O. graph for parr (V_7) were used in constructing the SI graph for V_{15}.

Interpreting Model Outputs

The model described above is a generalized description of habitat requirements for coho salmon and, as such, the output is not expected to discriminate among different habitats with a high resolution at this stage of development (see discussion in Terrell et al. 1982). Each model variable is considered to have some effect on habitat quality for coho, and the suitability index graphs depict what the measurable response is assumed to be. However, the graphs are derived from a series of untested assumptions, and it is unknown how accurately they depict habitat suitability for coho salmon. The model assumes that each model variable alone can limit coho production, but this has not been tested. A major potential weakness in the model is that, while the model variables may be necessary in determining suitability of habitat for coho, they may not be sufficient. Species interactions and other factors not included in this model may determine carrying capacity to a greater degree than the variables included in this model. Data describing measurable responses for additional factors are, however, scarce or nonexistent and, therefore, the variables do not meet the standards for consideration as variables in HSI model development (U.S. Fish and Wildlife Service 1981).

I recommend interpreting model outputs as indicators (or predictors) of excellent (0.8 to 1.0), good (0.5 to 0.7), fair (0.2 to 0.4), or poor (0.0 to 0.1) habitat for coho salmon. The output of the generalized model provided should be most useful as a tool in comparing different habitats. If two study areas have different HSI's, the one with the higher HSI is expected to have the potential to support more coho salmon. The model also should be useful as a basic framework for formulating revised models that incorporate site specific factors affecting habitat suitability for coho salmon and more detailed variable measurement techniques on a site-by-site basis.

ADDITIONAL HABITAT MODELS

No other habitat models that could be utilized in habitat evaluation for coho salmon were located in the literature. The user is referred to Terrell et al. (1982) and U.S. Fish and Wildlife Service (1981) for techniques to modify this model to meet project needs.

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A review and synthesis of existing information were used to develop a riverine habitat model for Coho Salmon (Oncorhychus kitsutch) an anadromous species. The model is scaled to produce an index of habitat suitability between 0 (unsuitable habitat) and 1 (optimally suitable habitat) for riverine areas of the continental United States. Habitat suitability indices (HSI's) are designed for use with Habitat Evaluation Procedures previously developed by the U.S. Fish and Wildlife Service.

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HABITAT SUITABILITY INDEX MODELS: BROOK TROUT

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PREFACE

The habitat use information and Habitat Suitability Index (HSI) models presented in this document are an aid for impact assessment and habitat management activities. Literature concerning a species' habitat requirements and preferences is reviewed and then synthesized into HSI models, which are scaled to produce an index between 0 (unsuitable habitat) and 1 (optimal habitat). Assumptions used to transform habitat use information into these mathematical models are noted, and guidelines for model application are described. Any models found in the literature which may also be used to calculate an HSI are cited, and simplified HSI models, based on what the authors believe to be the most important habitat characteristics for this species, are presented.

Use of the models presented in this publication for impact assessment requires the setting of clear study objectives and may require modification of the models to meet those objectives. Methods for reducing model complexity and recommended measurement techniques for model variables are presented in Terrell et al. (in press)[1]. A discussion of HSI model building techniques, including the component approach, is presented in U.S. Fish and Wildlife Service (1981).[2]

The HSI models presented herein are complex hypotheses of species-habitat relationships, not statements of proven cause and effect relationships. Results of model performance tests, when available, are referenced; however, models that have demonstrated reliability in specific situations may prove

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[1]Terrell, J. W., T. E. McMahon, P. D. Inskip, R. F. Raleigh, and K. W. Williamson (in press). Habitat suitability index models: Appendix A. Guidelines for riverine and lacustrine applications of fish HSI models with the Habitat Evaluation Procedures. U.S. Dept. Int., Fish Wildl. Serv. FWS/OBS-82/10.A.

[2]U.S. Fish and Wildlife Service. 1981. Standards for the development of Habitat Suitability Index models. 103 ESM. U.S. Dept. Int., Fish Wildl. Serv., Div. Ecol. Serv. n.p.

unreliable in others. For this reason, the U.S. Fish and Wildlife Service encourages model users to send comments and suggestions that might help us increase the utility and effectiveness of this habitat-based approach to fish and wildlife planning. Please send comments to:

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CONTENTS

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BROOK TROUT (Salvelinus fontinalis)

HABITAT USE INFORMATION

General

The native range of brook trout (Salvelinus fontinalis Mitchill) originally covered the eastern two-fifths of Canada northward to the Arctic Circle, the New England States, and southward through Pennsylvania, along the crest of the Appalachian Mountains to northeastern Georgia. Western limits included Manitoba southward through the Great Lake States. Reductions in the original range have resulted from environmental changes, such as pollution, siltation, and stream warming due to deforestation (MacCrimmon and Campbell 1969).

Since the late 19th century, brook trout have been introduced into 20 additional States and have sustaining populations in 14 States (MacCrimmon and Campbell 1969). Introductions have not been attempted in most of the central plains and the southern States.

Brook trout can be separated into two basic ecological forms: a short-lived (3-4 years), small (200-250 mm) form, typical of small, cold stream and lake habitats and a long-lived (8-10 years), large (4-6 kg), predaceous form associated with large lakes, rivers, and estuaries. The smaller, short-lived form is typically found south of the Great Lakes region and south of northern New England, while the larger form is located in the northern portion of its native range (Behnke 1980). Although no subspecies designation has been recognized for these two forms, they respond as two different species to environmental interactions influencing life history (Flick and Webster 1976; Flick 1977).

Brook trout can be hybridized artificially with lake trout (to produce a fertile hybrid called splake trout) and with rainbow trout (Buss and Wright 1957). In rare cases, natural hybrids occur between brook trout and brown trout (Salmo trutta); the hybrid is termed tiger trout (Behnke 1980). Behnke (1980) also collected brook trout and bull trout (Salvelinus confluentis) hybrids in the upper Klamath Lake basin, Oregon. Brook trout appear to be sensitive to introductions of brown and rainbow trout and are usually displaced by them. However, brook trout have displaced cutthroat trout and grayling in headwaters and tributaries of western streams (Webster 1975).

Age, Growth, and Food

Brook trout appear to be opportunistic sight feeders, utilizing both bottom-dwelling and drifting aquatic macroinvertebrates and terrestrial insects (Needham 1930; Dineen 1951; Wiseman 1951; Benson 1953; Reed and Bear 1966). Such feeding habits make them particularly susceptible to even moderate turbidity levels, which can reduce their ability to locate food (Bachman 1958; Herbert et al. 1961a, 1961b; Tebo 1975). Drifting forms may be selected over benthic forms when they are available (Hunt 1966). The choice of particular drift organisms is apparently either a function of seasonal availability and/or the overall availability of terrestrial forms in a particular situation. Between age groups, there may be a tendency for selection of food items based on size. In Idaho, age group 0 trout selected smaller drifting organisms (Diptera and Ephemeroptera) with less variation than did older trout, while age group I trout seemed to prefer larger Trichoptera larvae (Griffith 1974). Fish are an important food item in lake populations (Webster 1975).

Reproduction

Age at sexual maturity varies among populations, with males usually maturing before females (Mullen 1958). Male brook trout may mature as early as age 0+ (Buss and McCreary 1960; Hunt 1966). In Wisconsin (Lawrence Creek), the smallest mature male was approximately 8.9 cm (3.5 inches) long (McFadden 1961).

Spawning typically occurs in the fall and has been described by several authors (Greeley 1932; Hazzard 1932; Smith 1941; Brasch et al. 1958, Needham 1961). Spawning may begin as early as late summer in the northern part of the range and early winter in the southern part of the range (Sigler and Miller 1963). The spawning behavior of brook trout is very similar to that of rainbow and cutthroat trout (Smith 1941). In streams and ponds, areas of ground water upwelling appear to be highly preferred (Webster and Eiriksdottier 1976; Carline and Brynildson 1977) and to override substrate size as a site selection factor (Mullen 1958; Everhart 1966). Brook trout can be highly successful spawners in lentic environments in upwelling areas of springs (Webster 1975). Spawning occurs at temperatures ranging from 4.5-10° C (White 1930; Hazzard 1932; McAfee 1966). The fertilized ova are deposited in redds excavated by the female in the stream gravels (Smith 1947). Spawning success is reduced as the amount of fine sediments is increased and the intergravel oxygen concentration is diminished (McFadden 1961; Peters 1965; Harshbarger 1975).

Migration and Anadromy

With the exception of the sea-run New England populations, brook trout migrations are generally limited to movements into headwater streams or tributaries for spawning (Brasch et al. 1958) or relatively short seasonal migrations to avoid temperature extremes (Powers 1929; Scott and Crossman 1973). Some brook trout may spend their entire lives, including spawning periods, within a restricted stream area, as opposed to more migratory salmonids (McFadden et al. 1967). However, some movement upstream or downstream may occur due to space-related aggressive behavior following emergence from the redd (Hunt 1965).

Some coastal populations of brook trout may move into salt water from coastal streams of eastern Canada and northeastern United States. Sea-run individuals caught in salt water may differ in appearance, form, and coloration from trout that have never or have not recently been in salt water (Smith and Saunders 1958). Not all brook trout in the same stream will necessarily move to sea. In a study by White (1940), 79% of the brook trout going to sea were age 2, and the rest were age 3. Smith and Saunders (1958) stated that age 1 brook trout also migrated to the sea.

Smith and Saunders (1958) reported brook trout going to sea on Prince Edward Island during spring and early summer and during fall and early winter. Movement was observed in every month of the year, although very few fish were observed migrating during midwinter and midsummer. Smith and Saunders (1958) observed that approximately half of the brook trout migrating to salt water returned to freshwater within a month. As temperatures decline in freshwater, brook trout tend to spend more time in saltwater, and some may overwinter in saltwater (Smith and Saunders 1958).

Specific Habitat Requirements

Brook trout are the most generalized and adaptable of all *Salvelinus* species. They inhabit small headwater streams, large rivers, ponds, and large lakes in inland and coastal areas. Typical brook trout habitat conditions are those associated with a cold temperate climate, cool spring-fed ground water, and moderate precipitation (MacCrimmon and Campbell 1969). Warm water temperatures appear to be the single most important factor limiting brook trout distribution and production (Creaser 1930; Mullen 1958; McCormick et al. 1972). In a comparative distribution study between brook and brown trout from headwater tributaries of the South Platte River, Colorado, Vincent and Miller (1969) found that, as the elevation increased and the streams became smaller and colder, brook trout became more abundant.

Optimal brook trout riverine habitat is characterized by clear, cold spring-fed water; a silt-free rocky substrate in riffle-run areas; an approximate 1:1 pool-riffle ratio with areas of slow, deep water; well vegetated stream banks; abundant instream cover; and relatively stable water flow, temperature regimes, and stream banks. Brook trout south of Canada tend to occupy headwater stream areas, especially when rainbow and brown trout are present in the same river system (Webster 1975). They tend to inhabit large rivers in the northern portion of their native range (Behnke 1980).

Optimal lacustrine habitat is characterized as clear, cold lakes and ponds that are typically oligotrophic. Brook trout are typically stream spawners, but spawning commonly occurs in gravels surrounding spring upwelling areas of lakes and ponds.

Cover is recognized as one of the basic and essential components of trout streams. Boussu (1954) was able to increase the number and weight of trout in stream sections by adding artificial brush cover and to decrease numbers and weight by removing brush cover and undercut banks. Lewis (1969) found that the amount of cover present was important in determining the number of trout in sections of a Montana stream. Cover for trout consists of areas of low stream bottom visibility, suitable water depths (> 15 cm), and low current velocity (< 15 cm/s) (Wesche 1980). Cover can be provided by overhanging vegetation, submerged vegetation, undercut banks, instream objects (stumps, logs, roots, and large rocks), rocky substrate, depth, and water surface turbulence (Giger 1973). In a study to determine the amount of shade utilized by brook, rainbow, and brown trout, Butler and Hawthorne (1968) reported that rainbow trout showed the lowest preference for shade produced by artificial surface cover. Brown trout showed the highest use of shade while brook trout were intermediate between brown and rainbow trout. Brook trout in two Michigan streams showed a strong preference for overhead cover along the stream margin (Enk 1977). The major limiting factor for brook trout in these streams was bank cover.

Canopy cover is important in maintaining shade for stream temperature control and in providing allochthonous materials to the stream. Too much shade, however, can restrict primary productivity in a stream. Stream temperatures can be increased or decreased by controlling the amount of shade. About 50-75% midday shade appears optimal for most small trout streams (Anonymous 1979). Shading becomes less important as stream gradient and size increases. In addition, a well vegetated riparian area helps to control watershed erosion. In most cases, a buffer strip about 30 m deep, 80% of which is either well vegetated or has stable rocky stream banks, will provide adequate erosion control and maintain undercut stream banks characteristic of good trout habitat. The presence of fines in riffle-run areas can adversely affect embryo survival, food production, and cover for juveniles.

There is a definite relationship between the annual flow regime and the quality of trout habitat. The most critical period is typically the base flow (lowest flows of late summer to winter). A base flow ≥ 55% of the average annual daily flow is considered excellent, a base flow of 25 to 50% is considered fair, and a base flow of < 25% is considered poor for maintaining quality trout habitat (adapted from Wesche 1974; Binns and Eiserman 1979; Wesche 1980).

Hunt (1976) listed average depth, water volume, average depth of pools, amount of pool area, and amount of overhanging bank cover as the most important parameters relating to brook trout carrying capacity in Lawrence Creek, Wisconsin. The main use of summer cover is probably for predator avoidance and resting. Salmonids occupy different habitat areas in the winter than in the summer (Hartman 1965; Everest 1969; Bustard and Narver 1975a).

In some streams, the major factor limiting salmonid densities may be the amount of adequate overwintering habitat rather than summer rearing habitat (Bustard and Narver 1975a). Everest (1969) suggested that some salmonid population levels were regulated by the availability of suitable overwintering areas. Winter hiding behavior in salmonids is triggered by low temperatures (Chapman and Bjornn 1969; Everest 1969; Bustard and Narver 1975a,b). Bustard and Narver (1975a) indicated that, as water temperatures dropped to 4-8° C, feeding was reduced in young salmonids and most were found within or near cover; few were more than 1 m from potential cover. Everest (1969) found juvenile rainbows 15-30 cm deep in the substrate, which was often covered by 5-10 cm of anchor ice. Lewis (1969) reported that adult rainbow trout tended to move into deeper water during winter. The major advantages in seeking winter cover are prevention of physical damage from ice scouring (Hartman 1965; Chapman and Bjornn 1969) and conservation of energy (Chapman and Bjornn 1969; Everest 1969). A cover area ≥ 25% for adults and ≥ 15% for juveniles of the entire stream habitat appears adequate for most brook trout populations.

Optimum turbidity values for brook trout growth are approximately 0-30 JTU's, with a range of 0-130 JTU's (adapted from Sykora et al. 1972). An accelerated rate of sediment deposition in streams may reduce local brook trout production because of the adverse effects on production of food organisms, smothering of eggs and embryos in the redd, and loss of escape and overwintering habitat.

Brook trout appear to be more tolerant than other trout species to low pH (Dunson and Martin 1973; Webster 1975). Laboratory studies indicate that brook trout are tolerant of pH values of 3.5-9.8 (Daye and Garside 1975). Brook trout fingerlings in Pennsylvania inhabited a bog stream with a pH less than 4.75 and occassionally dropping to 4.0-4.2 (Dunson and Martin 1973). Parsons (1968) reported brook trout inhabiting a stream in Missouri with a pH of 4.1-4.2. Creaser (1930) believed that brook trout tolerated pH ranges greater than the range of most natural waters (4.1-9.5). Menendez (1976) demonstrated that continued exposure to a pH below 6.5 resulted in decreased hatching and growth in brook trout. The selection of spawning sites may be associated with the pH of upwelling water; neutral or alkaline waters (pH 6.7 and 8) were selected by brook trout held at pH levels of 4.0, 4.5, and 5.0 (Menendez 1976). The optimal pH range for brook trout appears to be 6.5-8.0, with a tolerance range of 4.0-9.5.

Brook trout occur in waters with a wide range of alkalinity and specific conductance, although high alkalinity and high specific conductance usually increase brook trout production (Cooper and Scherer 1967). Brook trout populations in the Smoky Mountains, North Carolina, are becoming increasingly restricted to low alkalinity headwater streams, apparently due to competition from introduced rainbow trout (Salmo gairdneri), and are frequently in poor condition (Lennon 1967). The small size of the trout in the headwater areas has been attributed to the infertility of the water, which has been linked to low total alkalinities (10 ppm or less) and TDS values less than 20 ppm. TDS values in the Smoky Mountains are lower than values from similar streams in Shenandoah National Park, Virginia, and the White Mountains National Forest, New Hampshire, where trout populations appear to be more robust.

Headwater trout streams are relatively unproductive. Most energy inputs to the stream are in the form of allochthonous materials, such as terrestrial vegetation and terrestrial insects (Idyll 1942; Chapman 1971; Hunt 1975). Aquatic invertebrates are most abundant and diverse in riffle areas with rubble substrate and on submerged aquatic vegetation (Hynes 1970). However, optimal substrate for maintenance of a diverse invertebrate population consists of a mosaic of gravel, rubble, and boulders with rubble being dominant. The invertebrate fauna is much more abundant and diverse in riffles than in pools (Hynes 1970), but a ratio of about 1:1 of pool to riffle area (about 40-60% pool area) appears to provide an optimum mix of trout food producing and rearing areas (Needham 1940). In riffle areas, the presence of fines (> 10%) reduces the production of invertebrate fauna (based on Cordone and Kelly 1961; Platts 1974).

Adult. The reported upper and lower temperature limits for adult brook trout vary; this may reflect local and regional population acclimation differences. Bean (1909) reported that brook trout will not live and thrive in temperatures warmer than 20° C. McAfee (1966) indicated that brook trout usually do poorly in streams where water temperature exceeds 20° C for extended periods. Brasch et. al (1958) reported that brook trout exposed to temperatures of 25° C for more than a few hours did not survive. Embody (1921) observed brook trout living in temperatures of 24-27° C for short durations and recommended 23.8° C as the maximum tolerable limit. Kendall (1924) agreed that 23.9° C represented the limit of even temporary endurance, but stated that the optimum temperature should not exceed 15.6° C. Hynes (1970) stated that brook trout can withstand temperatures from 0-25.3° C, but acclimation is necessary. The upper tolerable limit is raised by approximately 1° for every 7° rise in acclimation temperature up to 18° C, where it levels off at the absolute limit of 25.3° C. Fish kept at 24° C and above cannot tolerate temperatures as low as 0° C. Seasonal temperature cycles from summer highs to winter lows provide the necessary acclimation period needed to tolerate annual temperature extremes. The overall temperature range of 0-24° C was observed by MacCrimmon and Campbell (1969.

The above upper and lower tolerance limits probably do not reflect the range of temperatures that is most conducive to good growth. Baldwin (1951) cites an optimum growth rate at 14° C. He further contends that 11-16° C is best suited for overall welfare, while trout exist at a relative disadvantage in terms of activity and growth at higher and lower, albeit tolerable, temperatures. Mullen (1958) gave the optimum temperature range for activity and feeding for brook trout as between 12.8° C and 19° C. We assume that the temperature range for brook trout is 0-24° C, with an optimal range for growth and survival of 11-16° C.

Brook trout normally require high oxygen concentrations with optimum conditions at dissolved oxygen concentrations near saturation and temperatures above 15° C. Local or temporal variations should not decrease to less than 5 mg/l (Mills 1971). Dissolved oxygen requirements vary with age of fish, water temperature, water velocity, activity level, and concentration of substances in the water (McKee and Wolf 1963). As temperatures increase, the dissolved oxygen saturation level in the water decreases, while the dissolved oxygen requirements of the fish increases. As a result, an increase in temperature resulting in a decrease in dissolved oxygen can be detrimental to the fish. Optimum oxygen levels for brook trout are not well documented but appear to be ≥ 7 mg/l at temperatures < 15° C and ≥ 9 mg/l at temperatures ≥ 15° C. Doudoroff and Shumway (1970) demonstrated that swimming speed and growth rates for salmonids declined with decreasing dissolved oxygen levels. In the summer (temperatures ≥ 10° C), cutthroat trout generally avoid water with dissolved oxygen levels of less than 5 mg/l (Trojnar 1972; Sekulich 1974). Fry (1951) stated that the lowest dissolved oxygen concentrations where brook trout can exist is 0.9 ppm at 10° C and 1.6-1.8 ppm at 20° C. Embody (1927) contends that the dissolved oxygen concentration should not be less than 3 cc per liter (4.3 ppm).

Elson (1939) reported that brook trout prefer moderate flows. Griffith (1972) reported that focal point velocities for adult brook trout in Idaho ranged from 7-11 cm/sec, with a maximum of 25 cm/sec. In a Wyoming study, 95% of all brook trout observed were associated with point velocities of less than 15 cm/sec (Wesche 1974).

The carrying capacity of adult brook trout in streams is dependent, at least in part, on cover provided by pools, undercut banks, submerged brush and logs, large rocks, and overhanging vegetation (Saunders and Smith 1955, 1962; Elwood and Waters 1969; O'Connor and Power 1976). Enk (1977) reported that the biomass and number of brook trout ≥ 150 mm in size were significantly correlated with bank cover in two Michigan streams. Wesche (1980) reported that cover for adult trout should be located in stream areas with water depths ≥ 15 cm and velocities of < 15 cm/sec. We assume that an area ≥ 25% of the total stream area occupied by brook trout will provide adequate cover.

Embryo. Temperatures in the range of 4.5-11.5° C have been reported as optimum for egg incubation (MacCrimmon and Campbell 1969). Length of egg incubation is about 45 days at 10° C, 165 days at 2.8° C (Brasch et al. 1958), and 28 days at 14.8° C (Embody 1934). Brook trout eggs develop slightly faster than brown trout eggs at 2° C or colder, but the reverse is true at 3° C or above (Smith 1947). We assume that the range of acceptable temperatures for brook trout embryos is similar to that for cutthroat trout (*Salmo clarki*).

Dissolved oxygen concentrations should not fall below 50% saturation in the redd for embryo development (Harshbarger 1975). We assume that oxygen requirements for embryos are similar to those of adults. Peters (1965) observed high mortality rates when water velocity in the redd was reduced. Water velocity is important in flushing out fines in the redds. Because brook trout can successfully spawn in spawning areas of lakes, velocity is not necessary for successful spawning as long as oxygen levels are high and the redd is free of silt. Spawning velocities for brook trout range from 1 cm/sec (Smith 1973) to 92 cm/sec (Thompson 1972; Hooper 1973). Spawning velocities measured for brook trout in Wyoming ranged from 3-34 cm/sec (Reiser and Wesche 1977).

Reiser and Wesche (1977) stated that optimum substrate size for brook trout embryos ranges from 0.34-5.05 cm. Duff (1980) reported a range of suitable spawning gravel size of 3-8 cm in diameter for trout. Most workers agree that both water velocity and dissolved oxygen in the intergravel environment determine the adequacy of the substrate for the hatching and survival of salmonid embryos and fry. Increases in sediment that alter gravel permeability reduces velocities and intergravel dissolved oxygen availability to the embryo and results in smothering of eggs (Tebo 1975). In a California study, brook trout survival was lower as the volume of materials less than 2.5 mm in diameter increased (Burns 1970). In a 30% sand and 70% gravel mixture, only 28% of implanted steelhead embryos hatched; of those that hatched, only 74%

emerged (Bjornn 1971; Phillips et al. 1975). We assume that suitable spawning gravel conditions include gravels 3-8 cm in size (depending on size of spawners) with ≤ 5% fines.

Fry. McCormick et al. (1972) cited temperature as an important limiting factor of growth and distribution of young brook trout. Fry emerge from gravel redds from January to April, depending on the local temperature regime (Brasch et al. 1958). Temperatures from 9.8-15.4° C were considered suitable, with 12.4-15.4° C optimum; temperatures greater than 18° C were considered detrimental. The optimum temperature for brook trout fry, in a laboratory study, was between 8-12°C (Peterson et al. 1979). Upper lethal temperatures are between 21 and 25.8° C (Brett 1940), possibly a reflection of different acclimatization temperatures. Latta (1969) reported that upwelling ground water was an important consideration for the well-being of fry in streams; Carline and Brynildson (1977) reported the same situation for fry in spring ponds. Menendez (1976) found that fry survival increased as pH increased from 5 to 6.5. Griffith (1972) reported that focal point velocities for brook trout fry in Idaho ranged from 8-10 cm/sec, with a maximum of 16 cm/sec. Because brook trout fry occupy the same stream reaches as adults, we assume that temperature and dissolved oxygen requirements for brook trout fry are similar to those for adults.

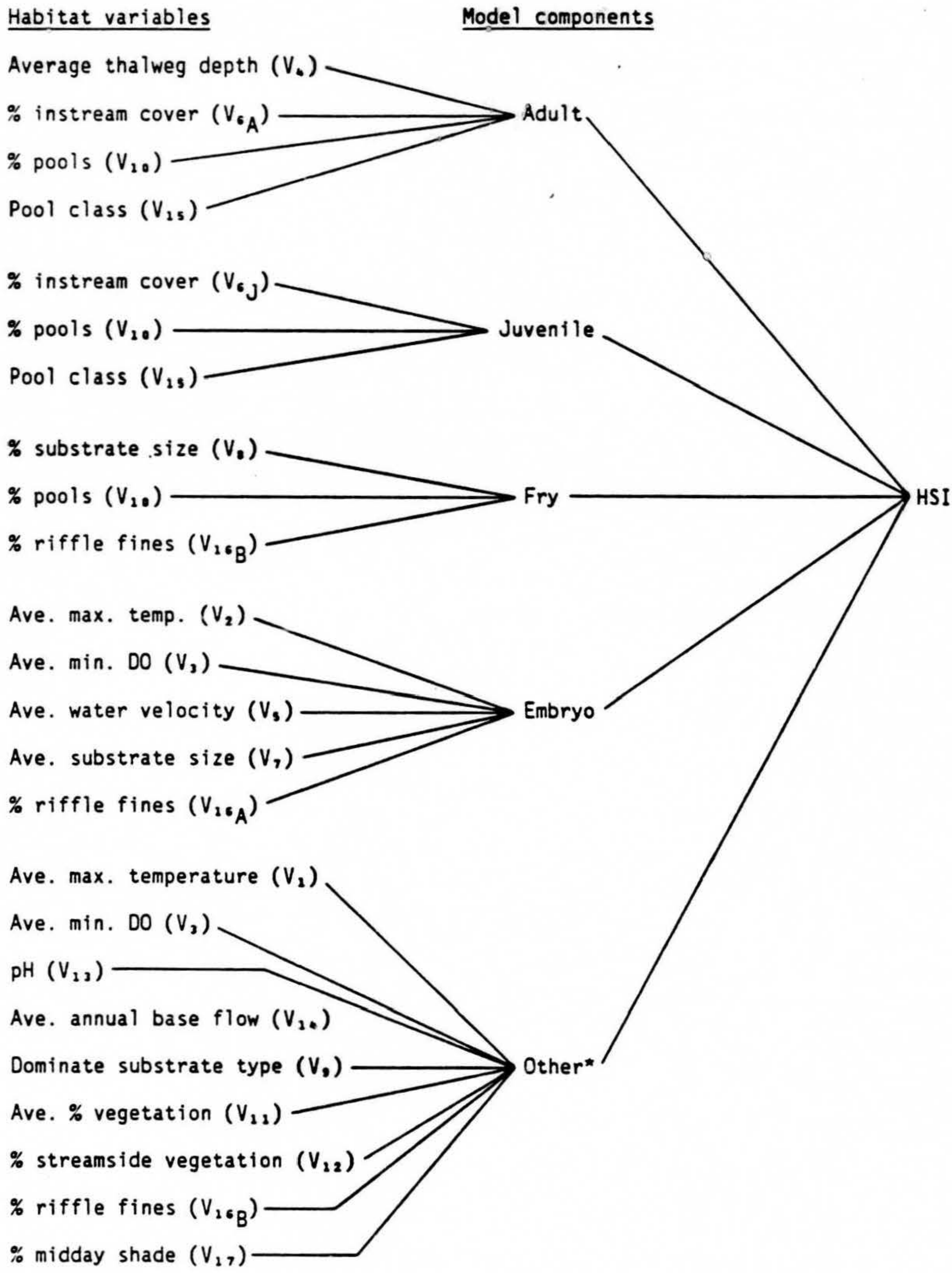
Trout fry usually overwinter in shallow areas of low velocity, with rubble being the principal cover (Everest 1969; Bustard and Narver 1975a). Optimum size of substrate used as winter cover by steelhead fry and small juveniles ranges from 10-40 cm in diameter (Hartman 1965; Everest 1969). A relatively silt-free area of substrate of this size class (10-40 cm), ≥ 10% of the total habitat, will probably provide adequate cover for brook trout fry and small juveniles. The use of smaller diameter rocks for winter cover may result in increased mortality due to shifting of the substrate (Bustard and Narver 1975a).

Juvenile. Davis (1961) stated that temperatures of 11-14° C are optimum for fingerling growth. Griffith (1972) reported focal point velocities for juvenile brook trout that ranged from 8.0-9.0 cm/sec, with a maximum of 24 cm/sec. We assume that temperature and dissolved oxygen requirements for juvenile brook trout are similar to those for adults.

Wesche (1980) reported that brook trout fry and small juveniles < 15 cm long were associated more with instream cover objects (rubble substrate) than overhead stream bank cover. An area of cover ≥ 15% of the total stream area appears adequate for juvenile brook trout.

HABITAT SUITABILITY INDEX (HSI) MODELS

Figure 1 depicts the theoretical relationships among model variables, components, and HSI for the brook trout model.



*Variables that affect all life stages.

Figure 1. Diagram illustrating the relationships among model variables, components, and HSI.

Model Applicability

Geographic area. The following model is applicable over the entire range of brook trout distribution. Where differences in habitat requirements have been identified for different races of brook trout, suitability index graphs have been constructed to reflect these differences. For this reason, care must be excercised in use of the individual graphs and equations.

Season. The model rates the freshwater habitat of brook trout for all seasons of the year.

Cover types. The model is applicable to freshwater riverine or lacustrine habitats.

Minimum habitat area. Minimum habitat area is the minimum area of contiguous habitat that is required for a species to live and reproduce. Because brook trout can move considerable distances to spawn or locate suitable summer or winter rearing habitat, no attempt has been made to define a minimum habitat size for the species.

Verification level. An acceptable level of performance for this brook trout model is for it to produce an index between 0 and 1 that the authors and other biologists familiar with brook trout ecology believe is positively correlated with the carrying capacity of the habitat. Model verification consisted of testing the model outputs from sample data sets developed by the author to simulate high, medium, and low quality brook trout habitat and model review by biologists familiar with brook trout ecology.

Model Description - Riverine

The riverine HSI model consists of five components: Adult (C_A); Juvenile (C_J); Fry (C_F); Embryo (C_E); and Other (C_O). Each life stage component contains variables specifically related to that component. The component C_O contains variables related to water quality and food supply that affect all life stages of brook trout.

The model utilizes a modified limiting factor procedure. This procedure assumes that model variables and components with suitability indices in the average to good range, > 0.4 to < 1.0, can be compensated for by higher suitability indices of other, related model variables and components. However, variables and components with suitabilities ≤ 0.4 cannot be compensated for and, thus, become limiting factors on habitat suitability.

Adult component. Variable V_6, percent instream cover, is included because standing crops of adult trout have been shown to be correlated with the amount of cover available. Percent pools (V_{10}) is included because pools provide cover and resting areas for adult trout. Variable V_{10} also quantifies the amount of pool habitat that is needed. Variable V_{15}, pool class, is included because pools differ in the amount and quality of escape cover, winter cover, and resting areas that they provide. Average thalweg depth (V_4) is included because average water depth affects the amount and quality of pools and instream cover available to adult trout and migratory access to spawning and rearing areas.

Juvenile component. Variables V_6, percent instream cover; V_{10}, percent pools; and V_{15}, pool class are included in the juvenile component for the same reasons listed above for the adult component. Juvenile brook trout use these essential stream features for escape cover, winter cover, and resting areas.

Fry component. Variable V_8, percent substrate size class, is included because trout fry utilize substrate as escape cover and winter cover. Variable V_{10}, percent pools, is included because fry use the shallow, slow water areas of pools and backwaters as resting and feeding stations. Variable V_{16}, percent fines, is included because the percent fines affects the ability of the fry to utilize the rubble substrate for cover.

Embryo component. It is assumed that habitat suitability for trout embryos depends primarily on water temperature, V_2; dissolved oxygen content, V_3; water velocity, V_5; spawning gravel size, V_7; and percent fines, V_{16}. Water velocity, V_5; gravel size, V_7; and percent fines, V_{16}, are interrelated factors that affect the transport of dissolved oxygen to the embryo and the removal of the waste products of metabolism from the embryo. These functions have been shown to be vital to the survival of trout embryos. In addition, the presence of too many fines in the redds will block movement of the fry from the incubating gravels to the stream.

Other component. This component contains model variables for two subcomponents, water quality and food supply, that affect all life stages. The subcomponent water quality contains four variables: maximum temperature (V_1); minimum dissolved oxygen (V_3); pH (V_{13}); and base flow (V_{14}). All four variables affect the growth and survival of all life stages except embryo, whose water quality requirements are included with the embryo component. The subcomponent food supply contains three variables: substrate type (V_9); percent vegetation (V_{11}); and percent fines (V_{16}). Dominant substrate type (V_9) is included because the abundance of aquatic insects, an important food item for brook trout, is correlated with substrate type. Variable V_{16}, percent fines in riffle-run and spawning areas, is included because the presence of excessive fines in riffle-run areas reduces the production of aquatic insects. Variable V_{11} is included because allochthonous materials are an important source of nutrients to cold, unproductive trout streams. The waterflow of all streams fluctuate on an annual seasonal cycle. A correlation exists between the average annual daily streamflow and the annual low base flow period in maintaining desirable stream habitat features for all life stages. Variable V_{14} is included to quantify the relationship between annual water flow fluctuations and trout habitat suitability.

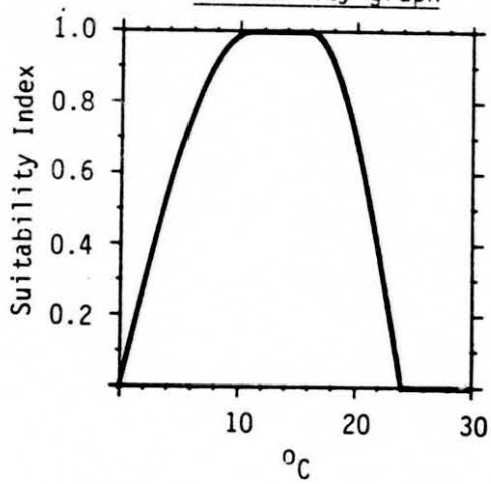
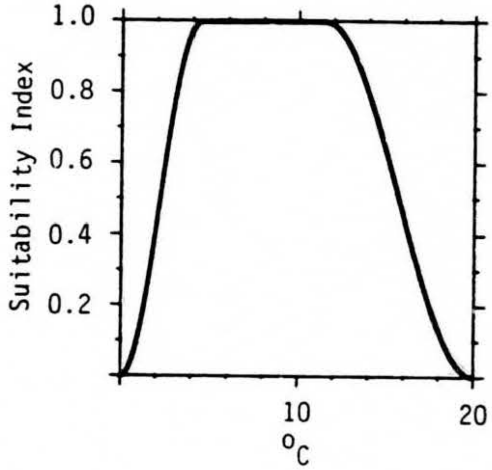
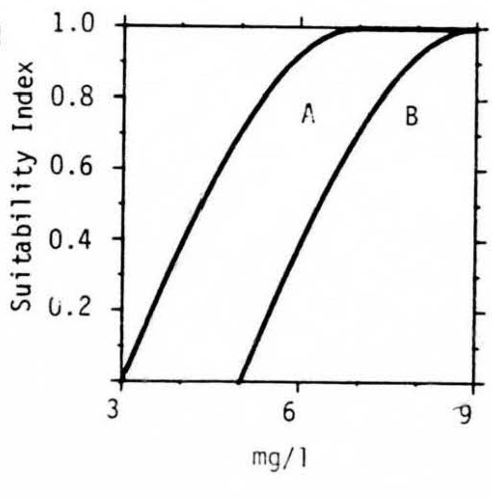
Variables V_{11}, V_{12}, and V_{17} are optional variables to be used only when needed and appropriate. Average percent vegetation for nutrient supply, V_{11}, should be used only on small (< 50 m wide) streams with summer temperatures > 10° C. Percent streamside vegetation, V_{12}, is included because streamside vegetation is an important means of controlling soil erosion, a major source of fines in streams. Variable V_{17}, percent midday shade, is included because the amount of shade can affect water temperature and photosynthesis in streams. Variables V_{11}, V_{12}, and V_{17} are used primarily for streams ≤ 50 m wide with temperature, photosynthesis, or erosion problems or when changes in the riparian vegetation is part of a potential project plan.

Suitability Index (SI) Graphs for Model Variables

This section contains suitability index graphs for 17 model variables. Equations and instructions for combining groups of variable SI scores into component scores and component scores into brook trout HSI scores are included.

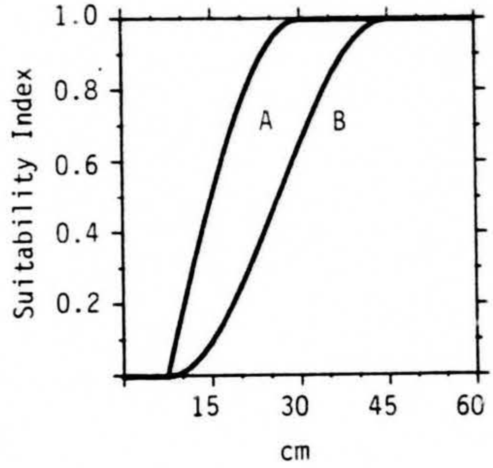
The graphs were constructed by quantifying information on the effect of each habitat variable on the growth, survival, or biomass of brook trout. The curves were built on the assumption that increments of growth, survival, or biomass originally plotted on the y-axis of the graph could be directly converted into an index of suitability from 0.0 to 1.0 for the species; 0.0 indicates unsuitable conditions and 1.0 indicates optimum conditions. Graph trend lines represent the author's best estimate of suitability for the various levels of each variable presented. The graphs have been reviewed by biologists familiar with the ecology of the species, but obviously some degree of SI variability exists. The user is encouraged to vary the shape of the graphs when existing regional information indicates a different variable suitability relationship.

The habitat measurements and SI graph construction are based on the premise that extreme, rather than average, values of a variable most often limit the carrying capacity of a habitat. Thus, measurement of extreme conditions, e.g., maximum temperatures and minimum dissolved oxygen levels, are often the data used with the graphs to derive the SI values for the model. The letters R and L in the habitat column identify variables used to evaluate riverine (R) or lacustrine (L) habitats.

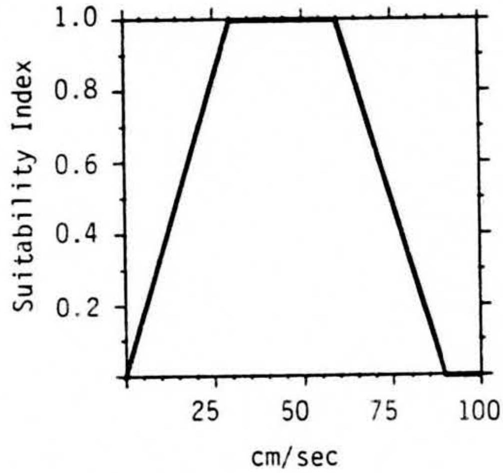
Habitat	Variable		Suitability graph
R,L	V_1	Average maximum water temperature (°C) during the warmest period of the year (adult, juvenile, and fry). For lacustrine habitats, use temperature strata nearest optimum in dissolved oxygen zones of > 3 mg/l.	
R	V_2	Average maximum water temperature (°C) during embryo development.	
R,L	V_3	Average minimum dissolved oxygen (mg/l) during the late growing season low water period and during embryo development (adult, juvenile, fry, and embryo). For lacustrine habitats, use the dissolved oxygen readings in temperature zones nearest to optimum where dissolved oxygen is > 3 mg/l. A = ≤ 15° C B = > 15° C	

R V_4 Average thalweg depth (cm) during the late growing season low water period.

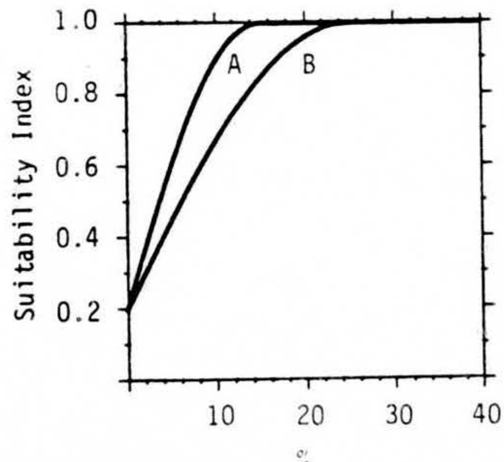
A = stream width ≤ 5 m
B = stream width > 5 m



R V_5 Average velocity (cm/sec) over spawning areas during embryo development.

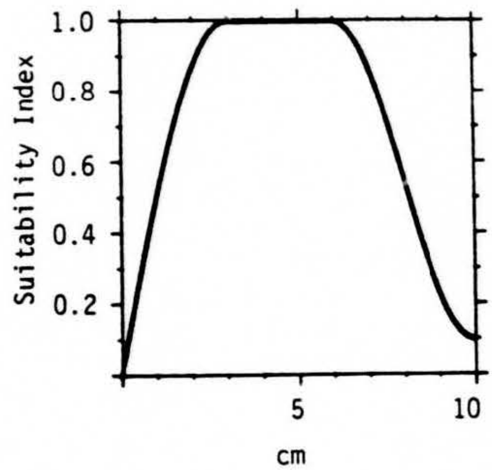


R V_6 Percent instream cover during the late growing season low water period at depths ≥ 15 cm and velocities < 15 cm/sec.
A = Juveniles
B = Adults

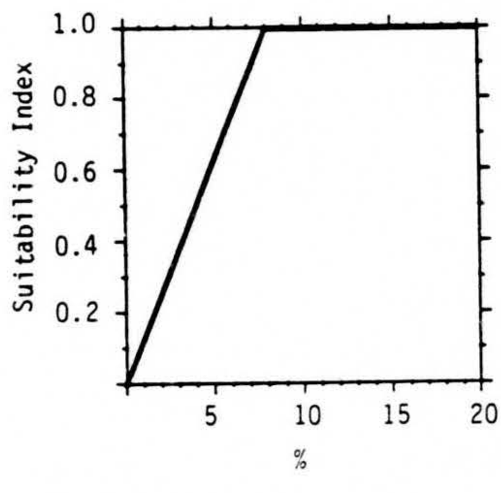


R V_7 Average size of substrate between 0.3-8 cm diameter in spawning areas, preferably during the spawning period.

To derive an average value for use with graph V_7, include areas containing the best spawning substrate sampled until all potential spawning sites are included or the sample contains an area equal to 5% of the total brook trout habitat being evaluated.

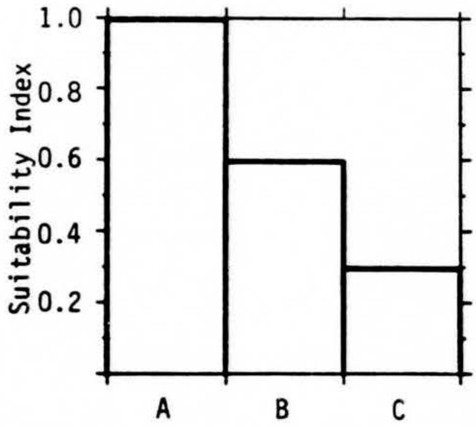


R V_8 Percent substrate size class (10-40 cm) used for winter and escape cover by fry and small juveniles.

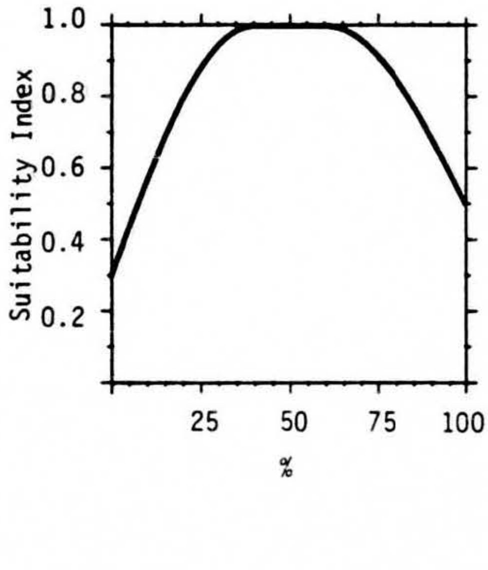


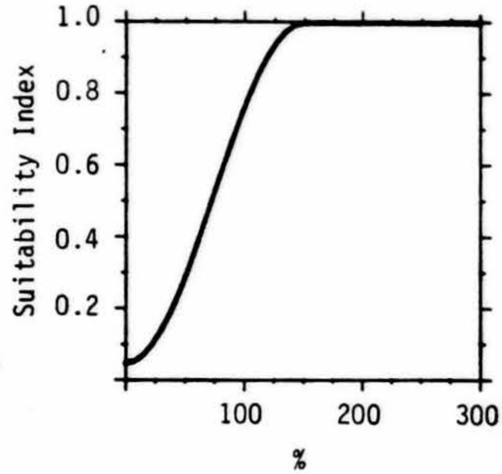
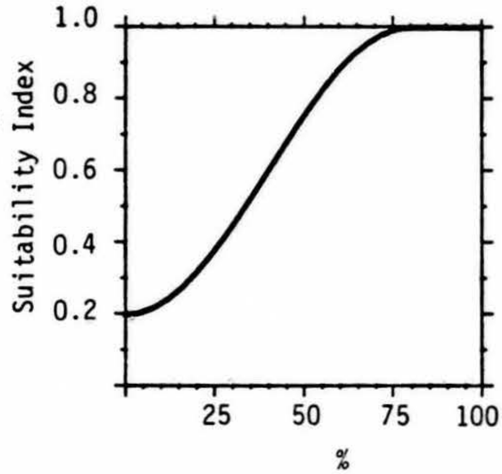
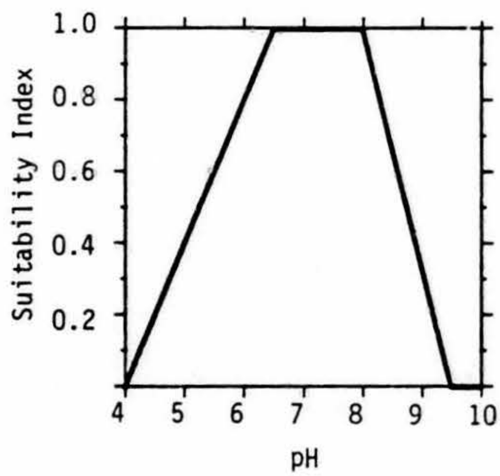
R V_9 Dominant (≥ 50%) substrate type in riffle-run areas for food production.

A) Rubble or small boulders or aquatic vegetation in spring areas dominant, with limited amounts of gravel, large boulders, or bedrock.
B) Rubble, gravel, boulders, and fines occur in approximately equal amounts or gravel is dominant. Aquatic vegetation may or may not be present.
C) Fines, bedrock, or large boulders are dominant. Rubble and gravel are insignificant (≤ 25%).

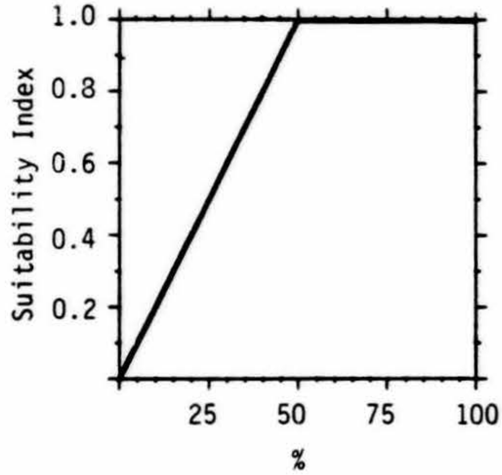


R V_{10} Percent pools during the late growing season low water period.



R	V_{11} Optional	Average percent vegetation (trees, shrubs, and grasses-forbs) along the streambank during the summer for allochthonous input. Vegetation Index = 2 (% shrubs) + 1.5 (% grasses) + (% trees) + 0 (% bareground). (For streams ≤ 50 m wide)	
R	V_{12} Optional	Average percent rooted vegetation and stable rocky ground cover along the streambank during the summer (erosion control).	
R,L	V_{13}	Annual maximal or minimal pH. Use the measurement with the lowest SI value. For lacustrine habitats, measure pH in the zone with the best combination of dissolved oxygen and temperature.	

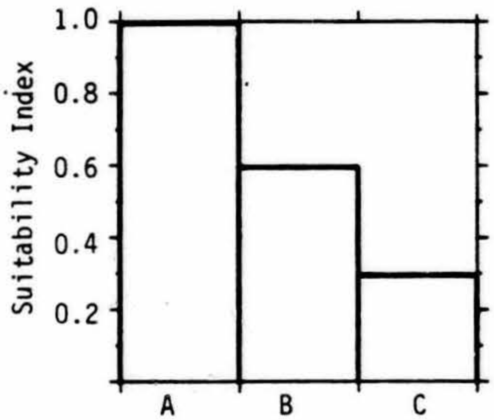
R V_{14} Average annual base flow regime during the late summer or winter low flow period as a percent of the average annual daily flow.



R V_{15} Pool class rating during the late growing season low flow period (Aug-Oct). The rating is based on the percent of the area containing pools of the three classes described below.

A) ≥ 30% of the area is comprised of first-class pools.
B) ≥ 10% but < 30% first-class pools or ≥ 50% second-class pools.
C) < 10% first-class pools and < 50% second-class pools.

(See pool class descriptions below)



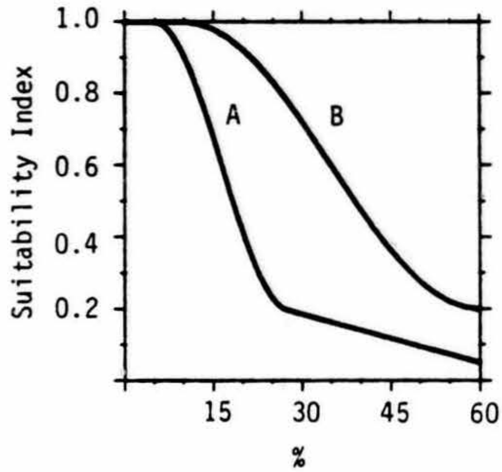
A) First-class pool: Large and deep. Pool depth and size are sufficient to provide a low velocity resting area for several adult trout. More than 30% of the pool bottom is obscured due to depth, surface turbulence, or the presence of structures, e.g., logs, debris piles, boulders, or overhanging banks and vegetation. Or, the greatest pool depth is ≥ 1.5 m in streams ≤ 5 m wide or ≥ 2 m deep in streams > 5 m wide.

B) Second-class pool: Moderate size and depth. Pool depth and size are sufficient to provide a low velocity resting area for a few adult trout. From 5 to 30% of the bottom is obscured due to surface turbulence, depth, or the presence of structures. Typical second-class pools are large eddies behind boulders and low velocity, moderately deep areas beneath overhanging banks and vegetation.

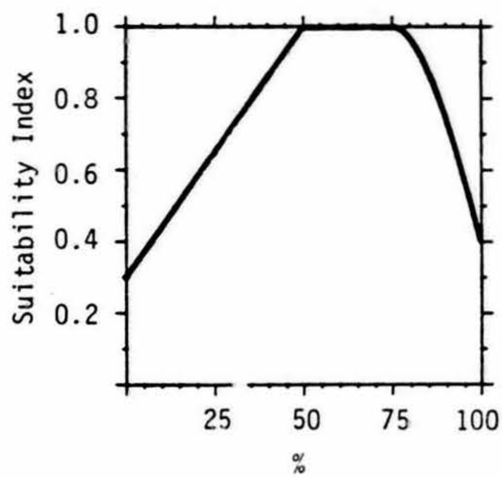
C) Third-class pool: Small or shallow or both. Pool depth and size are sufficient to provide a low velocity resting area for one to very few adult trout. Cover, if present, is in the form of shade, surface turbulence, or very limited structures. Typical third-class pools are wide, shallow pool areas of streams or small eddies behind boulders.

R V_{16} Percent fines (< 3 mm) in riffle-run and in spawning areas during average summer flows.

A = Spawning
B = Riffle-run



R V_{17} Optional Percent of stream area shaded between 1000 and 1400 hrs (for streams ≤ 50 m wide). Do not use on cold (< 16° C max. temp.), unproductive streams.



References to sources of data and the assumptions used to construct the above suitability index graphs for brook trout HSI models are presented in Table 1.

Table 1. Data sources for brook trout suitability indices.

	Variable and source	Assumption
V_1	Bean 1909 Embody 1921 Kendall 1924 Baldwin 1951 Brasch et al. 1958 Mullen 1958 Davis 1961 McAfee 1966 MacCrimmon & Campbell 1969 Hynes 1970	Average maximum daily temperatures have a greater effect on trout growth and survival than minimum temperature.
V_2	Embody 1934 Smith 1947 Brasch et al. 1958 MacCrimmon & Campbell 1969	The average maximum daily water temperature during embryo development related to the highest survival of embryos and normal development is optimum.
V_3	Embody 1927 Fry 1951 Doudoroff & Shumway 1970 Mills 1971 Trojnav 1972 Sekulich 1974 Harshbarger 1975	The average minimum daily dissolved oxygen level during embryo development and the late growing season that is related to the greatest growth and survival of brook trout and trout embryos is optimum. Levels that reduce survival and growth are suboptimum.
V_4	Delisle and Eliason 1961 Estimated by authors	The average thalweg depths that provide the best combination of pools, instream cover, and instream movement of adult trout is optimum.
V_5	Thompson 1972 Hooper 1973 Hunter 1973 Reiser and Wesche 1977	The average velocity over the spawning areas affects the dissolved oxygen concentration and the manner in which waste products are removed from the developing embryos. Average velocities that result in the highest survival of embryos are optimum. Velocities that result in reduced survival are suboptimum.

Table 1 (continued).

Variable and source		Assumption
		6. There are 10,000 m^2 per hectare $\frac{500}{10,000} = 5\%$ of total area
V_8	Hartman 1965 Everest 1969 Bustard and Narver 1975a	The substrate size range selected for escape and winter cover by brook trout fry and small juveniles is assumed to be optimum.
V_9	Pennak and Van Gerpen 1947 Hynes 1970	The dominant substrate type containing the greatest numbers of aquatic insects is assumed to be optimum for insect production.
V_{10}	Needham 1940 Elser 1968 Hunt 1971	The percent pools during late summer low flows that is associated with the greatest trout abundance is optimum.
V_{11}	Idyll 1942 Delisle and Eliason 1961 Chapman 1971 Hunt 1975	The average percent vegetation along the streambank is related to the amount of allochthanous materials deposited annually in the stream. Shrubs are the best source of allochthanous materials, followed by grasses and forbs, and then trees. The vegetational index is a reasonable approximation of optimum and suboptimum conditions for most trout stream habitats.
V_{12}	Anonymous 1979 Raleigh and Duff 1981	The average percent rooted vegetation and rocky ground cover that provides adequate erosion control to the stream is optimum.
V_{13}	Creaser 1930 Parsons 1968 Dunson & Martin 1973 Daye & Garside 1975 Webster 1975 Menendez 1976	The average annual maximum or minimum pH levels related to high survival of trout are optimum.

Table 1 (concluded).

Variable and source		Assumption
V_{14}	Binns 1979 Adapted from Duff and Cooper 1976	Flow variations affect the amount and quality of pools, instream cover, and water quality. Average annual base flows associated with the highest standing crops are optimum.
V_{15}	Needham 1940 Lewis 1969 Hunt 1976	Pool classes associated with the highest standing crops of trout are optimum.
V_{16}	Cordone & Kelly 1961 Bjornn 1969 Sykora et al. 1972 Platts 1974 Phillips et al. 1975	The percent fines associated with the highest standing crops of food organisms, embryos, and fry in each designated area is optimum.
V_{17}	Sabean 1976, 1977 Anonymous 1979	The percent of stream area that is shaded that is associated with optimum water temperatures and photosynthesis rates is optimum.

The above references include data from studies on related salmonid species. This information has been selectively used to supplement, verify, or complete data gaps on the habitat requirements of brook trout.

The suitability curves are a compilation of published and unpublished information on brook trout. Information from other life stages or species or expert opinion was used to formulate curves when data for a particular habitat parameter or life stage were insufficient. Data are not sufficient at this time to refine the habitat suitability curves that accompany this narrative to reflect subspecific or regional differences. Local knowledge should be used to regionalize the suitability curves if that information will yield a more precise suitability index score. Additional information on this species that can be used to improve and regionalize the suitability curves should be forwarded to the Habitat Evaluation Group, U.S.D.I. Fish and Wildlife Service, 2625 Redwing Road, Fort Collins, CO 80526.

Riverine Model

This model uses a life stage approach with five components: adult; juvenile; fry; embryo; and other.

Adult (C_A). C_A variables: V_4; V_6; V_{10}; and V_{15}

Case 1: Where V_6 is > $(V_{10} \times V_{15})^{1/2}$;

$$C_A = [V_4 \times V_6 \ (V_{10} \times V_{15})^{1/2}]^{1/3}$$

Case 2: Where V_6 is ≤ $(V_{10} \times V_{15})^{1/2}$;

$$C_A = [V_4 \ (V_{10} \times V_{15})^{1/2}]^{1/2}$$

If V_4 or $(V_{10} \times V_{15})^{1/2}$ is ≤ 0.4 in either equation, then C_A = the lowest score.

Juvenile (C_J). C_J variables: V_6; V_{10}; and V_{15}

$$C_J = \frac{V_6 + V_{10} + V_{15}}{3}$$

Or, if any variable is ≤ 0.4, C_J = the lowest variable score.

Fry (C_F). C_F variables: V_8; V_{10}; and V_{16}

$$C_F = [V_{10} \ (V_8 \times V_{16})^{1/2}]^{1/2}$$

Or, if V_{10} or $(V_8 \times V_{16})^{1/2}$ is ≤ 0.4, C_F = the lowest factor score.

Embryo (C_E). C_E variables: V_2; V_3; V_5; V_7; and V_{16}

Steps:

A. A potential spawning site is an ≥ 0.5 m² area of gravel, 0.3-8.0 cm in size, covered by flowing water ≥ 15 cm deep. At each spawning site sampled, record:

1. The average water velocity over the site;
2. The average size of all gravel between 0.3-8.0 cm;
3. The percent fines < 0.3 cm in the gravel; and
4. The total area in m² of each site.

B. Derive a spawning site suitability index (V_s) for each site by combining V_5, V_7, and V_{16} values follows:

$$V_s = (V_5 \times V_7 \times V_{16})^{1/3}$$

C. Derive a weighted average ($\bar{V}_s$) for all sites included in the sample.

Select the best V_s scores until all sites are included, or until brook trout habitat has been included, whichever comes first.

$$\bar{V}_s = \frac{\sum_{i=1}^{n} A_i V_{si}}{\text{total habitat area}} /0.05 \text{ (output cannot > 1.0)}$$

where A_i = the area of each spawning site in m² (ΣA_i cannot exceed 5% of the total brook trout habitat).

V_{si} = the individual SI scores from the best spawning areas until all spawning sites have been included or until SI's from an area equal to 5% of the total brook trout habitat being evaluated has been included, whichever occurs first.

D. Derive C_E

C_E = the lowest score of V_2, V_3, or $\bar{V}_s$

Other (C_O). C_O variables: V_1; V_3; V_9; V_{11}; V_{12}; V_{13}; V_{14}; V_{16}; and V_{17}

$$C_O = \left[\frac{(V_9 \times V_{16})^{1/2} + V_{11}}{2} \times (V_1 \times V_3 \times V_{12} \times V_{13} \times V_{14} \times V_{17})^{1/N} \right]^{1/2}$$

where N = the number of variables within the parentheses. Note that variables V_{11}, V_{12} and V_{17} are optional and, therefore, can be omitted.

HSI determination. HSI scores can be derived for a single life stage, a combination of two or more life stages, or all life stages combined. In all cases, except for the embryo component (C_E), an HSI is obtained by combining one or more life stage component scores with the other component (C_O) score.

1. Equal Component Value Method. The equal component value method assumes that each component exerts equal influence in determining the HSI. This method should be used to determine the HSI unless information exists that individual components should be weighted differently. Components: C_A; C_J; C_F; C_E; and C_O.

$$HSI = (C_A \times C_J \times C_F \times C_E \times C_O)^{1/N}$$

Or, if any component is ≤ 0.4, the HSI = the lowest component value; if C_A is < the equation value, the HSI = C_A.

where N = the number of components in the equation.

Solve the equation for the number of components included in the evaluation. There will be a minimum of two, one or more life stage components and the component (C_O), unless only the embryo life stage (C_E) is being evaluated, in which case the HSI = C_E.

2. Unequal Component Value Method. This method also uses a life stage approach with five components: adult (C_A); juvenile (C_J); fry (C_F); embryo (C_E); and other (C_O). However, the C_O component is divided into two subcomponents, food (C_{OF}) and water quality (C_{OQ}). It is assumed that the C_{OF} subcomponent can either increase or decrease the suitability of the habitat by its effect on growth at each life stage except embryo.

The C_{OQ} subcomponent is assumed to exert an influence equal to the combined influence of all other model components in determining habitat suitability. The method also assumes that water quality is excellent, C_{OQ} = 1. When C_{OQ} is < 1, the HSI is decreased. In addition, when a basis for weighting exists, model component and subcomponent weights can be increased by multiplying each index value by multipliers > 1. Model weighting procedures must be documented.

Components and subcomponents: C_A; C_J; C_F; C_E; C_{OF}; and C_{OQ}

Steps:

A. Calculate the subcomponents (C_{OF} and C_{OQ}) of C_O

$$C_{OF} = \frac{(V_9 \times V_{16})^{1/2} + V_{11}}{2}$$

$$C_{OQ} = (V_1 \times V_3 \times V_{13} \times V_{14})^{1/4}$$

Or, if any variable is ≤ 0.4, C_{OQ} = the value of the lowest variable.

B. Calculate the HSI by either the noncompensatory or the compensatory option.

Noncompensatory option. This option assumes that degraded water quality conditions cannot be compensated for by good physical habitat conditions. This assumption is most likely true for small streams (≤ 5 m wide) and for persistent degraded water quality conditions.

$$HSI = (C_A \times C_J \times C_F \times C_E \times C_{OF})^{1/N} \times C_{OQ}$$

where N = the number of components and subcomponents inside the parentheses or, if the model components or subcomponents have unequal weights, N = Σ of weights selected.

Or, if any component is ≤ 0.4, HSI = the lowest component value x C_{OQ}.

If only the embryo component is being evaluated, HSI = C_E x C_{OQ}.

Compensatory option. This method assumes that moderately degraded water quality conditions can be partially compensated for by good physical habitat conditions. This assumption is useful for large rivers ($\geq$ 50 m wide) and for temporary, or short term, poor water quality conditions.

1) $HSI' = (C_A \times C_J \times C_F \times C_E \times C_{OF})^{1/N}$

where N = the number of components and subcomponents in the equation or, if the model components or subcomponents have unequal weights, $N = \Sigma$ of weights selected.

Or, if C_A is ≤ 0.4, the $HSI' = C_A$

2) If C_{OQ} is < HSI', HSI = the HSI' $\times [1 - (HSI' - C_{OQ})]$; if $C_{OQ} \geq$ HSI', the HSI = HSI'.

3) If only the embryo component is being evaluated, follow the procedure in step 2, substituting C_E for HSI'.

Lacustrine Model

The following model can be used to evaluate brook trout lacustrine habitat. The lacustrine model consists of two components: water quality and reproduction.

Water Quality (C_{WQ}). C_{WQ} variables: V_1; V_3; and V_{13}

$$C_{WQ} = (V_1 \times V_3 \times V_{13})^{1/3}$$

Or, if the SI scores for V_1 or V_3 are ≤ 0.4, C_{WQ} = the lowest SI score for V_1 or V_3.

Note: Lacustrine brook trout can spawn in spring upwelling areas of lacustrine habitats but will utilize tributary streams for spawning and embryo development when available and suitable. If the embryo life stage riverine habitat is included in the evaluation, use the embryo component steps and equations in the riverine model above, except that the area of spawning gravel needed is only about 1% of the total surface area of the lacustrine habitat.

Embryo (C_E). C_E variables: V_2; V_3; V_5; V_7; and V_{16}

$$\bar{V}_s = \frac{\sum_{i=1}^{n} A_i \, V_{si}}{\text{total habitat area}} \; /0.01 \text{ (output cannot > 1.0)}$$

HSI determination.

$$HSI = (C_{WQ} \times C_E)^{1/2}$$

If only the lacustrine habitat is evaluated, the HSI = C_{WQ}.

Interpreting Model Outputs

Model HSI scores for individual life stages, composite life stages, or for the species are a relative indicator of habitat suitability. The HSI models, in their present form, are not intended to reliably predict standing crops of fishes throughout the United States. Standing crop limiting factors, such as interspecific competition, predation, disease, water nutrient levels, and length of growing season, are not included in the aquatic HSI models. The models contain physical habitat variables important in maintaining viable populations of brook trout. If the model is correctly structured, a high HSI score for a habitat indicates near optimum regional conditions for brook trout for those factors included in the model, intermediate HSI scores indicate average habitat conditions, and low HSI scores indicate poor habitat conditions. An HSI of 0 does not necessarily mean that the species is not present; it does indicate that the habitat is very poor and that the species is likely to be scarce or absent.

Brook trout tend to occupy riverine habitats where very few other fish species are present. They are usually competitively excluded by other salmonid species, except cutthroat. Thus, disease, interspecific competition, and predation usually have little affect on the model. When the brook trout model is applied to brook trout streams with similar water quality and lengths of growing season, it should be possible to calibrate the model output to reflect size of standing crops within some reasonable confidence limits. This possibility, however, has not been tested with the present model.

Sample data sets selected by the author to represent high, intermediate, and low habitat suitabilities are in Table 2, along with the SI's and HSI's generated by the brook trout riverine model. The model outputs calculated from the sample data sets (Tables 3 and 4) reflect what I believe carrying capacity trends would be in riverine habitats with the listed characteristics.

The models also have been reviewed by biologists familiar with brook trout ecology; therefore, the model meets the previously specified acceptance level.

ADDITIONAL HABITAT MODELS

Model 1

Optimum riverine brook trout habitat is characterized by:

1. Clear, cold water with an average maximum summer temperature of < 22° C;

2. Approximately a 1:1 pool-riffle ratio;

3. Well vegetated, stable stream banks;

4. ≥ 25% of stream area providing cover;

5. Relatively stable water flow regime, < 50% annual fluctuation from average annual daily flow;

6. Relatively stable summer temperature regime, averaging about 13° C ± 4° C;

7. A relatively silt-free rocky substrate in riffle-run areas; and

8. Relatively good water quality (e.g., DO and pH).

$$HSI = \frac{\text{number of attributes present}}{8}$$

Table 2. Sample data sets using the riverine brook trout HSI model.

Variable		Data set 1 Data	SI	Data set 2 Data	SI	Data set 3 Data	SI
Max. temperature (°C)	V_1	14	1.0	15	1.0	16	1.0
Max. temperature (°C)	V_2	12	1.0	15	0.6	16	0.4
Min. dissolved O_2 (mg/l)	V_3	9	1.0	5	0.7	6	0.4
Ave. depth (cm)	V_4	25	0.9	17	0.6	17	0.6
Ave. velocity (cm/s)	V_5	30	1.0	20	0.7	20	0.7
% cover	V_6	20	A 0.9 J 1.0	10	A 0.7 J 0.9	10	A 0.7 J 0.9
Ave. gravel size (cm)	V_7	4	1.0	3	1.0	2.5	1.0
% substrate 10-40 cm in diameter	V_8	15	1.0	6	0.7	6	0.7
Dom. substrate class	V_9	A	1.0	B	0.6	B	0.6
% pools	V_{10}	55	1.0	15	0.7	10	0.6
% Alloch. vegetation	V_{11}	225	1.0	175	1.0	200	1.0
% bank vegetation	V_{12}	95	1.0	40	0.6	35	0.5
Max. pH	V_{13}	7.1	1.0	7.2	1.0	7.2	1.0
% ann. base flow	V_{14}	39	0.8	30	0.6	25	0.5

Table 2. (concluded).

Variable		Data set 1		Data set 2		Data set 3	
		Data	SI	Data	SI	Data	SI
Pool class	V_{15}	A	1.0	B	0.6	C	0.3
% fines (A)	V_{16}	5	1.0	20	0.4	20	0.4
% fines (B)	V_{16}	20	0.9	35	0.6	35	0.6
% shade	V_{17}	60	1.0	60	1.0	60	1.0

Table 3. Equal component value method.

Variable	Data set 1		Data set 2		Data set 3	
	Data	SI	Data	SI	Data	SI
Component						
C_A		0.95		0.65		0.56
C_J		1.00		0.73		0.30
C_F		0.97		0.67		0.62
C_E		1.00		0.60		0.40
C_O		0.97		0.79		0.74
Species HSI		0.98		0.68		0.50

Table 4. Unequal component value method.

Variable	Data set 1		Data set 2		Data set 3	
	Data	SI	Data	SI	Data	SI
Component						
C_A		0.95		0.65		0.56
C_J		1.0		0.73		0.30
C_F		0.97		0.67		0.62
C_E		1.00		0.60		0.40
C_{OF}		0.97		0.80		0.80
C_{OQ}		1.00		0.81		0.40
Species HSI						
Noncompensatory		0.98		0.56		0.12
Compensatory		0.98		0.69		0.51

Model 2

A riverine trout habitat model has been developed by Binns and Eiserman (1979) Transpose the model output of pounds per acre to an index of 0-1:

$$HSI = \frac{\text{model output of pounds per acre}}{\text{regional optimum pounds per acre}}$$

Model 3

Optimum lacustrine brook trout habitat is characterized by:

1. Clear, cold water with an average summer midepilimnion temperature of $< 22°$ C;

2. A midepilimnion pH of 6.5 to 8.5;

3. Dissolved oxygen content of epilimnion of ≥ 8 mg/l; and

4. Presence of spring upwelling areas or access to riverine spawning tributaries.

$$HSI = \frac{\text{number of attributes present}}{4}$$

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16. Abstract (Limit: 200 words)

Literature describing the habitat preferences of the brook trout (Salvelinus fontinalis) is reviewed, and the relationships between habitat variables and life requisites are synthesized into a Habitat Suitability Index (HSI) model. HSI models are designed for use with the Habitat Evaluation Procedures (HEP) in impact assessment and habitat management activities.

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Habitability
Habit
Fishes
Trout

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Salvelinus fontinalis
Habitat Suitability Index (HSI)
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Classification of Surface Waters

DEPARTMENT OF ENVIRONMENTAL PROTECTION
· STATE OF MAINE ·

September 1979

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Title 38
Chapter 3

DEPARTMENT OF ENVIRONMENTAL PROTECTION

Ray Building

Augusta Mental Health Institute Complex

Augusta, Maine 04333

OFFICE OF THE COMMISSIONER

BUREAU OF WATER QUALITY CONTROL

INDEX
TITLE 38
CHAPTER 3
CLASSIFICATION OF SURFACE WATERS

REGULATIONS

§ 363. Standards of classification of fresh waters

The board shall have 4 standards for the classification of fresh surface waters.

1972, c. 618.

Class A shall be the highest classification and shall be of such quality that it can be used for recreational purposes, including bathing, and for public water supplies after disinfection. The dissolved oxygen content of such waters shall not be less than 75% saturation or as naturally occurs, and contain not more than 20 fecal coliform bacteria per 100 milliliters.

1977, c. 373, § 1.

These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease or scum. There shall be no disposal of any matter or substance in these waters which would impart color, turbidity, taste or odor other than that which naturally occurs in said waters, nor shall such matter or substance alter the temperature or hydrogen-ion concentration of these waters or contain chemical constituents which would be harmful or offensive to humans or which would be harmful to animal or aquatic life. No radioactive matter or substance shall be permitted in these waters other than that occurring from natural phenomena.

There shall be no discharge of sewage or other pollutants into water of this classification and no deposits of such material on the banks of these waters in any manner that transfer of sewage other other pollutants into the waters is likely, except that existing licensed discharges into waters of this classification will be allowed to continue until practical alternatives exist. New discharges will be permitted only if, in addition to satisfying all the requirements of this chapter, the discharged effluent will be equal to or better than the existing water quality of the receiving waters. Prior to issuing a discharge license, the board shall require the applicant to objectively demonstrate to the board's satisfaction that the discharge is necessary and that there are no other reasonable alternatives available.

1971, c. 461, §2; 1977, c. 373, §2; 1979, c. 529.

Class B, the 2nd highest classification, shall be divided into 2 designated groups as **B-1** and **B-2**.

B-1. Waters of this class shall be considered the higher quality of the Class B group and shall be acceptable for recreational purposes, including water contact recreation, for use as potable water supply after adequate treatment and for a fish and wildlife habitat. The dissolved oxygen of such waters shall be not less than 75% of saturation, and not less than 5 parts per million at any time. The fecal coliform bacteria shall not exceed 60 per 100 milliliters.

1977, c. 373, §3.

These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease or scum. There shall be no disposal of any matter or substance in these waters which imparts color, turbidity, taste or odor which would impair the usages ascribed to this classification nor shall such matter or substance alter the temperature or hydrogen-ion concentration of these waters so as to render such waters harmful to fish or other aquatic life. There shall be no discharge to these waters which will cause the hydrogen-ion concentration or "pH" of these waters to fall outside of the 6.0 to 8.5 range. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans, animals or squatic life or which adversely affect any other water use in this class. No radioactive matter or substances shall be discharged to these waters which will raise the radio-nuclide concentrations above the standards as established by the United States Public Health Service as being acceptable for drinking water. These waters shall be free of any matter or substance which alters the composition of bottom fauna, which adversely affects the physical or chemical nature of bottom material, or which interferes with the propagation of fish.

There shall be no disposal of sewage, industrial wastes or other wastes in such waters, except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the uasges of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

B-2. Waters of this class shall be acceptable for recreational purposes including water contact recreation, for industrial and potable water supplies after adequate treatment, and for a fish and wildlife habitat. The dissolved oxygen of such waters shall not be less than 60% of saturation, and not less than 5 parts per million at any time. The fecal coliform bacteria is not to exceed 200 per 100 millilieters.

1977, c. 373, §4.

These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease and scum. There shall be no disposal of any matter or substance in these waters which imparts color, turbidity, taste or odor which would impair the usages ascribed to this classification, nor shall such matter or substance alter the temperature or hydrogen-ion concentration of the waters so as to render such waters harmful to fish or other aquatic life. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans, animal or aquatic life, or which adversely affect any other water use in this class. There shall be no discharge to these waters which will cause the hydrogen-ion concentration of "pH" of these waters to fall outside of the 6.0 to 8.5 range. No radioactive matter or substance shall be discharged to these waters which will raise the radio-nuclid concentrations above the standards as established by the United States Public Health Service as being acceptable for drinking water. These waters shall be free of any matter or substance which alters the composition of bottom fauna, which adversely affects the physical or chemical nature of bottom material, or which interferes with the propagation of fish.

There shall be no disposal of sewage, industrial wastes or other wastes in such waters except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

Class C. waters, The 3rd highest classification, shall be of such quality as to be satisfactory for recreational boating and fishing, for a fish and wildlife habitat and for other uses except potable water supplies and water contact recreation, unless such waters are adequately treated.

The dissolved oxygen content of such waters shall not be less than 5 parts per million, except in those cases where the board finds that the natural dissolved oxygen of any such body of water falls below 5 parts per million, in which case the board may grant a variance to this requirement. In no event shall the dissolved oxygen content of such waters be less than 4 parts per million. The fecal coliform bacteria is not to exceed 1,000 per 100 milliliters.

1973, c. 423, §5; 1977, c. 373, §5.

These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease or scum. There shall be no disposal of any matter or substance in these waters which imparts color, turbidity, taste, or odor which would impair the usages ascribed to this classification, nor shall such matter or substance alter the temperature or hydrogen-ion content of the waters so as to render such waters harmful to fish or other aquatic life. There shall be no discharge to these waters which will cause the hydrogen-ion concentration or "pH" of these waters to fall outside of the 6.0 to 8.5 range. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans, animal or aquatic life or which adversely affect any other water use in this class. No radioactive material or substance shall be discharged to these waters which will raise the radio-nuclide concentration above the standards as established by the United States Public Health Service as being acceptable for drinking water.

There shall be no disposal of sewage, industrial wastes or other wastes in such waters, except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

Class D waters shall be assigned only where a higher water classification cannot be attained after utilizing the best practicable treatment or control of sewage or other wastes.

Waters of this class may be ...ed
waters after adequate treatm...t.

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health or impair any usages as ...be

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With respect to all classifi... ion...
may be appropriate for the b... in
tion is temporarily lowered due to a...

R.S. 1954, c. 79, §2; 1955, c. ...
§1; 1967, c. 475, §4; 1969, c ... 31,

§363-A. Standards of class...ati...

The board shall have 2 standard...

Class GP-A shall be the highest ...
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plies after disinfection. Such ...ter
meters or as naturally occur... and
milliliters. Total phosphorus conc...
phyll A concentration shall not ex...
near the surface of the water.

These waters shall be free ...om
scum. No radioactive matter or su...
occurring from natural phenomena.

There shall be no direct or ...ire
ful to water quality or aquat... ife
tion 371-A and 413. No mate...ls
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therefrom may flow or leach into th...

1979, c. 495.

Class GP-B, the 2nd hig...t c...
including water contact recreation
and for a fish and wildlife habitat
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measured in samples taken at ...ne...

These waters shall be free from
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imparts color, turbidity, taste or o...
tion nor shall such matter or ...bs...
these waters so as to render ...h

Waters of this class may be used for power generation, navigation and industrial process waters after adequate treatment.

Dissolved oxygen of these waters shall not be less than 2.0 parts per million. The numbers of coliform bacteria allowed in these waters shall be only those amounts which will not, in the determination of the Commission, indicate a condition harmful to the public health or impair any usages ascribed to this classification.

These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease or scum. There shall be no disposal of any matter or substance in these waters which imparts color, turbidity, taste or odor which would impair the usages ascribed to this classification, nor shall such matter or substance alter the temperature or hydrogen-ion concentration of the waters to impair the usages of this classification. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans or which adversely affect any other water use in this class. No radioactive matter or substance shall be permitted in these waters which would be harmful to humans, animal or aquatic life and there shall be no disposal of any matter or substance which would result in radio-nuclide concentrations in edible fish or other aquatic life thereby rendering them dangerous for human consumption.

There shall be no disposal of sewage, industrial wastes or other wastes in such waters, except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the usages of this classification. Treated wastes discharging to these waters shall not create a public nuisance as defined in Title 17, Section 2802, by the creation of odor producing sludge banks and deposits or other nuisance conditions.

With respect to all classifications hereinbefore set forth, the board may take such actions as may be appropriate for the best interests of the public, when it finds that any such classification is temporarily lowered due to abnormal conditions of temperature or stream flow.

R.S. 1954, c. 79, §2; 1955, c. 425, §5; 1959, c. 295, §2; 1961, c. 305, §3; 1963, c. 274, §1; 1967, c. 475, §4; 1969, c. 431, §§ 1, 2; 1972, c. 618; 1979, c. 529.

§363-A. Standards of classification of great ponds

The board shall have 2 standards for the classification of great ponds.

Class GP-A shall be the highest classification and shall be of such quality that it can be used for recreational purposes, including bathing, fish and wildlife habitat and for public water supplies after disinfection. Such waters shall have a Secchi disk transparency of not less than 2.0 meters or as naturally occurs, and contain not more than 20 fecal coliform bacteria per 100 milliliters. Total phosphorus concentration shall not exceed 15 parts per billion, and chlorophyll A concentration shall not exceed 8 parts per billion as measured in samples taken at or near the surface of the water.

These waters shall be free from sludge deposits, solid refuse, floating solids, oils, grease and scum. No radioactive matter or substance shall be permitted in these waters other than that occurring from natural phenomena.

There shall be no direct or indirect discharge of sewage, pollutants or other substances harmful to water quality or aquatic life into waters of this classification except as provided in section 371-A and 413. No materials shall be placed on the shores or banks thereof in such a manner that the same may fall or be washed into the waters or in such a manner that the drainage therefrom may flow or leach into those waters.

1979, c. 495.

Class GP-B, the 2nd highest classification, shall be acceptable for recreational purposes, including water contact recreation, for use as potable water supply after adequate treatment, and for a fish and wildlife habitat. The fecal coliform bacteria count is not to exceed 60 per 100 milliliters. The total phosphorus concentration shall not exceed 50 parts per billion as measured in samples taken at or near the surface of the water.

These waters shall be free from sludge deposits, solid refuse and floating solids, such as oils, grease or scum. There shall be no disposal of any matter or substance in these waters which imparts color, turbidity, taste or odor which would impair the usages ascribed to this classification nor shall such matter or substance alter the temperature or hydrogen-ion concentration of these waters so as to render such waters harmful to fish or other aquatic life. There shall be no discharge to these waters which will cause the "pH" of these waters to fall outside of the 5.5 to 8.5 range. There shall be no disposal of any substance that contains chemical constituents which are harmful to humans, animals or aquatic life or which adversely affect any other water use in this class. No radioactive matter or substances shall be discharged to these waters which will raise the radio-nuclide concentrations above the standards established by the United States Public Health Service as being acceptable for drinking water. These waters shall be free of any matter or substance which alters the composition of bottom fauna, which adversely affects the physical or chemical nature of bottom material, or which interferes with the propagation of fish.

There shall be no disposal of sewage, industrial wastes or other wastes in such waters, except those which have received treatment for the adequate removal of waste constituents, including, but not limited to, solids, color, turbidity, taste, odor and toxic material, such that these treated wastes will not cause any violation of water quality standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to squatic life or cause it to be dangerous for human consumption. There shall be no additional discharge of phosphorus to waters of this classification, which discharge does not employ the best available technology for phosphorus removal.

1977, c. 373, §6; 1979, c. 495, §§ 1, 2.

§363-B. Standards of classification of ground water

The board shall have 2 standards for the classification of ground water.

Class GW-A shall be the highest classification and shall be of such quality that it can be used for public water supplies. These waters shall be free of radioactive matter or any matter that imparts color, turbidity, taste or odor which would impair usage of these waters, other than that occurring from natural phenomena.

Class GW-B, the 2nd highest classification, shall be suitable for all usages other than public water supplies.

1979, c. 472, §10.

§ 364. Tidal or Marine Waters

The board shall have 5 standards for classification of tidal waters.

1971, c. 470, § 2; 1972, c. 618.

Class SA, shall be suitable for all clean water usages, including water contact recreation, and fishing. Such waters shall be suitable for the harvesting and propagation of shellfish and for a fish and wildlife habitat. These waters shall contain not less than 6.0 parts per million of dissolved oxygen at all times. The median numbers of coliform bacteria in any series of samples representative of waters in the shellfish growing area or non-shellfish growing area shall not be in excess of 70 per 100 milliliters, nor shall more than 10% of the samples exceed 230 coliform bacteria per 100 milliliters. The median numbers of fecal coliform bacteria in any series of samples representative of waters in the shellfish growing area or non-shellfish growing area shall not be in excess of 14 per 100 milliliters, nor shall more than 10% of the samples exceed 43 fecal coliform bacteria per 100 milliliters.

1977, c. 373, § 7.

There shall be no floating solids, settleable solids, oil or sludge deposits attributable to sewage, industrial wastes or other wastes and no deposit garbage, cinders, ashes, oils, sludge or other refuse. There shall be no discharge of sewage or other wastes, except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

There shall be no toxic wastes, deleterious substances, colored or other waste or heated liquids discharged to waters of this classification either singly or in combinations with other substances or wastes in such amounts or at such temperatures as to be injurious to edible fish or shellfish or to the culture or propagation thereof, or which in any manner shall adversely affect the flavor, color, odor or sanitary condition thereof; and otherwise none in sufficient amounts to make the waters unsafe or unsuitable for bathing or impair the waters for any other best usage as determined for the specific waters assigned to this class. There shall be no discharge which will cause the hydrogen-ion concentration or "pH" of these waters to fall outside of the 6.7 to 8.5 range.

There shall be no disposal of any matter or substances that contains chemical constituents which are harmful to humans, animal or aquatic life or which adversely affect any other water use in this class. No radioactive matter or substance shall be permitted in these waters would be harmful to humans, animal or aquatic life and there shall be no disposal of any matter or substance which would result in radio-nuclide concentrations in edible fish or other aquatic life thereby rendering them dangerous for human consumption. These waters shall be free of any matter or substance which alters the composition of bottom fauna, which adversely affects the physical or chemical nature of bottom material, or which interferes with the propagation of fish or shellfish if indigenous to the area.

Class SB-1 shall be suitable for all clean water usages including water contact recreation, and fishing. Such waters shall be suitable for the harvesting and propagation of shellfish, and for a fish and wildlife habitat. These waters shall contain not less than 6.0 parts per million of dissolved oxygen at all times. The median numbers of coliform bacteria in any series of samples representative of waters in the shellfish growing area shall not be in excess of 70 per 100 milliliters, nor shall more than 10% of the samples exceed 230 coliform bacteria per 100 milliliters. The median numbers of fecal coliform bacteria in any series of samples representative of waters in the shellfish growing area shall not be in excess of 14 per 100 milliliters, nor shall more than 10% of the samples exceed 43 fecal coliform bacteria per 100 milliliters. In a non-shellfish growing area the median numbers of coliform bacteria in a series of samples representative of the waters shall not exceed 240 per 100 milliliters, nor shall more than 10% of the samples exceed 50 coliform bacteria per 100 milliliters. In a non-shellfish growing area the median numbers of fecal coliform bacteria in a series of samples representative of the waters shall not exceed 50 per 100 milliliters, nor shall more than 10% of the samples exceed 150 fecal coliform bacteria per 100 milliliters.

1977, c. 373, § 8.

There shall be no floating solids, settleable solids, oil or sludge deposits attributable to sewage, industrial wastes or other wastes and no deposit of garbage, cinders, ashes, oils, sludge or other refuse. There shall be no discharge of sewage or other wastes, except those which have received treatment for the adequate removal of waste constituents including but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

There shall be no toxic wastes, deleterious substances, colored or other wastes or heated liquids discharged to waters of this classification, either singly or in combination with other substances or wastes in such amounts or at such temperatures as to be injurious to edible fish or shellfish or to the culture or propagation thereof, or which in any manner shall adversely affect the flavor, color, odor or sanitary condition thereof; and otherwise none in sufficient amounts to make the waters unsafe or unsuitable for bathing or impair the waters for any other best usage as determined for the specific waters which are assigned to this class. There shall be no waste discharge which will cause the hydrogen-ion concentration or "pH" of these waters to fall outside the 6.7 to 8.5 range. There shall be no disposal of matter or substance that contains chemical constituents which are harmful to humans, animal or aquatic life or which adversely affects any other water use in this class. No radioactive matter or substance shall be permitted in these waters which would be harmful to humans, animal or aquatic life and there shall be no disposal of any matter or substance which would result in radio-nuclide concentrations in edible fish or other aquatic life thereby rendering them dangerous for human consumption. These waters shall be free of any matter or substance which alters the composition of bottom fauna, which adversely affects the physical or chemical nature of bottom material or which interferes with the propagation of fish or shellfish if indigenous to the area.

Class SB-2 shall be suitable for recreational usages, including water contact, and fishing. Such waters shall be suitable for the harvesting and propagation of shellfish, for a fish and wildlife habitat, and suitable for industrial cooling and process uses. These waters shall contain not less than 6.0 parts per million of dissolved oxygen at all times. The median numbers of coliform bacteria in any series of samples representative of waters in the shellfish growing area shall not be in excess of 70 per 100 milliliters, nor shall more than 10% of the samples exceed 230 coliform bacteria per 100 milliliters. The median numbers of fecal coliform bacteria in any series of samples representative of waters in the shellfish growing area shall not be in excess of 14 per 100 milliliters, nor shall more than 10% of the samples exceed 43 fecal coliform bacteria per 100 milliliters. In a non-shellfish growing area the median numbers of coliform bacteria in a series of samples representative of the waters shall not exceed 500 per 100 milliliters, nor shall more than 10% of the samples exceed 1,000 coliform bacteria per 100 milliliters. In a non-shellfish growing area the median numbers of fecal coliform bacteria in a series of samples representative of the waters shall not exceed 100 per 100 milliliters, nor shall more than 10% of the samples exceed 200 fecal coliform bacteria per 100 milliliters. There shall be no floating solids, settleable solids, oil or sludge deposits attributable to sewage, industrial wastes or other wastes and no deposit of garbage, cinders, ashes, oils, sludge or other refuse. There shall be no discharge of sewage or other wastes, except those having received treatment for the adequate removal of waste constituents including but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

1977, c. 373, § 9.

There shall be no toxic wastes, deleterious substances, colored or other wastes or heated liquids discharged to waters of this classification either singly or in combination with other substances or wastes in such amounts or at such temperatures as to be injurious to edible fish or shellfish or to the culture or propagation thereof, or which in any manner shall adversely affect the flavor, color, odor or sanitary condition thereof; and otherwise none in sufficient amounts to make the waters unsafe or unsuitable for bathing or impair the waters for any other best usage as determined for the specific waters assigned to this class. There shall be no waste discharge which will cause the hydrogen-ion concentration or "pH" of the receiving waters to fall outside of the 6.7 to 8.5 range. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans, animal or aquatic life or which adversely affects any other water use in this class. No radioactive matter or substance shall be permitted in these waters which would be harmful to humans, animal or aquatic life and there shall be no disposal of any matter or substance which would result in radio-nuclide concentrations in edible fish or other aquatic life thereby rendering them dangerous for human consumption. These waters shall be free of any matter or substance which alters the composition of bottom fauna, which adversely affects the physical or chemical nature of bottom material, or which interferes with the propagation of fish or shellfish if indigenous to this area.

Class SC, the 4th highest classification, shall be of such quality as to be satisfactory for recreational boating, fishing and other similar uses except primary water contact. Such waters may be used for the propagation of indigenous shellfish to be harvested for depuration purposes, for a fish and wildlife habitat, and for industrial cooling and process uses. The dissolved oxygen content of such waters shall not be less than 5 parts per million at any time. The median numbers of coliform bacteria in any series of samples representative of waters in the shellfish growing area shall not be in excess of 700 per 100 milliliters, nor shall more than 10% of the samples exceed 2,300 coliform bacteria per 100 milliliters. The median numbers of fecal coliform bacteria in any series of samples representative of waters in the shellfish growing area shall not be in excess of 150 per 100 milliliters, nor shall more than 10% of the samples exceed 500 fecal coliform bacteria per 100 milliliters. In a non-shellfish growing area the median number of coliform bacteria in a series of samples representative of the waters shall not exceed 1,500 per 100 milliliters nor shall more than 10% of the samples exceed 5,000 coliform bacteria per 100 milliliters.

In a non-shellfish growing area the median numbers of fecal coliform bacteria in a series of samples representative of the waters shall not exceed 300 per 100 milliliters, nor shall more than 10% of the samples exceed 1,000 fecal coliform bacteria per 100 milliliters.

There shall be no floating solids, settleable solids, oil or sludge deposits attributable to sewage, industrial waste or other wastes, and no deposit of garbage, cinders, ashes, oils, sludge or other refuse. There shall be no discharge of sewage or other wastes, except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic materials, such that these treated wastes will not lower the standards or alter the usages of this classification, nor shall such disposal of sewage or waste be injurious to aquatic life or render such dangerous for human consumption.

There shall be no toxic wastes, deleterious substances, colored or other wastes or heated liquids discharged to waters of this classification either singly or in combinations with other substances or wastes in such amounts or at such temperatures as to be injurious to edible fish or shellfish or to the culture or propagation thereof, or which in any manner shall adversely affect the flavor, color, or odor thereof or impair the waters for any other discharge ascribed to waters of this classification. There shall be no waste discharge which will cause the hydrogen-ion concentration or "pH" of the receiving waters to fall outside the 6.7 to 8.5 range. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans, animal or aquatic life or which adversely affects any other water use in this class. No radioactive matter or substance shall be permitted in these waters which would be harmful to humans, animals or aquatic life and there shall be no disposal of any matter or substance which would result in radio-nuclide concentrations in edible fish or other aquatic life thereby rendering them dangerous for human consumption.

Class SD waters shall be assigned only where a higher water classification cannot be attained after utilizing the best practicable treatment or control of sewage or other wastes. Waters of this class may be used for power generation, navigation, industrial process waters or cooling waters, and for migration of fish. Dissolved oxygen of these waters shall not be less than 3.0 parts per million at any time. The numbers of coliform bacteria allowed in these waters shall be only those amounts which will not, in the determination of the board, indicate a condition harmful to the public health or impair any usages ascribed to this classification.

1972, c. 618.

These waters shall be free from sludge deposits, solid refuse and floating solids such as oils, grease or scum. There shall be no disposal of any matter or substance in these waters which imparts color, turbidity, taste or odor which would impair the usages ascribed to this classification, nor shall such matter or substance alter the temperature or hydrogen-ion concentration of the waters so as to impair the usages of this classification. There shall be no disposal of any matter or substance that contains chemical constituents which are harmful to humans or which adversely affect any other water use in this class. No radioactive matter or substance shall be permitted in these waters which would be harmful to humans, animal or aquatic life and there shall be no disposal of any matter or substance which would result in radio-nuclide concentrations in edible fish or other aquatic life thereby rendering them dangerous for human consumption.

There shall be no disposal of sewage, industrial wastes or other wastes in such waters, except those which have received treatment for the adequate removal of waste constituents including, but not limited to, solids, color, turbidity, taste, odor or toxic material, such that these treated wastes will not lower the standards or altar the usages of this classification. Treated wastes discharged to these waters shall not create a public nuisance as defined in Title 17, Section 2802, by the creation of odor-producing sludge banks and deposits or other nuisance conditions.

With respect to all classifications hereinbefore set forth, the Board may take such actions as may be appropriate for the best interests of the public, when it finds that any such classification is temporarily lowered due to abnormal conditions of temperature or stream flow.

1963, c. 274, § 2; 1967, c. 475, § 5; 1969, c. 431, § 3; 1970, c. 581, § 2; 1971, c. 476, §§ 2, 3; 1972, c. 618.

§ 368. Inland waters

Androscoggin River Basin

1957, c. 322, § 1

Little Androscoggin River Drainage

1. All segments of the Little Androscoggin River drainage system not otherwise specified – Class B-1.

1967, c. 451, § 1.

2. Andrews Brook (Woodstock and Paris) – Class B-2.

3. Bird Brook (Norway) – Class C.

4. Bog Brook, in Hebron, Mechanic Falls and Minot and tributaries not otherwise specified – Class B-2.

5. Davis Brook (Poland) – Class C.

6. Hodgkins Brook (Auburn) tributary of Taylor Pond – Class B-2.

7. Indian Brook (Minot) – Class B-2.

8. Lapham Brook (Auburn) tributary of Taylor Brook – Class B-2.

9. Little Androscoggin River, main stem, from a point 0.25 miles above the bridge at West Paris to the confluence with Andrews Brook – Class C.

10. Little Androscoggin River, main stem, from the Andrews Brook confluence to the Route 26 Bridge in South Paris – Class B-2.

11. Little Androscoggin River, main stem, from the Route 26 Bridge in South Paris to the confluence with the outlet of Thompson Lake in Oxford – Class D.

12. Little Androscoggin River, main stem, from the confluence of the Thompson Lake Outlet (Oxford) to the confluence with the Androscoggin River in Auburn – Class C.

1967, c. 451, § 1.

13. Meadow Brook (Oxford and Poland) – Class B-2.

14. Minister Brook (Oxford) – Class B-2.

15. Moose Pond Outlet at Otisfield (does not include Greeley Brook) – Class B-2.

16. Morgan Brook (Minot) – Class B-2.

17. Outlet of Little Pennesseewassee Lake (Norway) – Class B-2.

18. Outlet of Thompson Lake (Oxford) – Class C.

19. Pennesseewassee Lake Outlet (Norway) – Class C.

1967, c. 304, § 9.

20. Range Brook and its tributaries, Mechanic Falls and Poland – Class B-1.

21. Taylor Brook (Auburn) – Class B-2.

1969, c. 88.

22. Unnamed Stream entering Bryant Pond, rising in the vicinity of Bucks Lodge and flowing through Bryant Pond Village – Class B-2.

23. Unnamed Brook (Minot) the first stream entering the Little Androscoggin River on upstream of and on the same side of the river as Morgan Brook – Class B-2.

24. Unnamed Brook in Auburn which enters the Little Androscoggin River from the north about 1.3 miles east of Minot Village – Class C.

25. West Branch of unnamed stream which enters north and Pennessewassee Lake from vicinity of Nobles Corner – Class B-2.

26. West Branch of Bog Brook and tributaries, Gardiner Brook and tributaries, and Brickwell Brook and tributaries, all in the Bog Brook drainage in the Mechanic Falls, Minot and Hebron – Class B-1.

Main Stem, Androscoggin River, that portion located below the most downstream crossing of the Maine-New Hampshire boundary to a line formed by the extension of the Brunswick-West Bath town line across Merrymeeting Bay in a northwesterly direction – Class C.

1967, c. 451, § 2.

Minor tributaries, Androscoggin River, those tributaries located below the most downstream crossing of the Maine-New Hampshire boundary a line formed by the extension of the Brunswick-West Bath town line across Merrymeeting Bay in a northwesterly direction.

1. All tributaries, direct and indirect of the Androscoggin River, lying wholly within the State of Maine, not otherwise specified or classified – Class B-1.

A. No Name Brook (Lewiston) – Class C.

1969, c. 120.

B. Logan Brook (Auburn) – Class C.

1969, c. 120.

C. Penley Brook (Auburn) – Class C.

1969, c. 120.

2. All tributaries, direct and indirect, of the Androscoggin River between the New Hampshire boundary and the bridge at West Peru not otherwise specified or classified – Class B-1.

3. Alder River, Bethel, main stem, from the confluence of Kendall Brook to the Androscoggin River – Class B-2.

1977, c. 373, § 10.

4. Alder River, main stem, from the outlet of South Pond at Lockes Mills Village to the confluence with Kendall Brook – Class B-2.

5. Austin Brook (or Abbott Brook), in Mexico, from Fourth Street to the Androscoggin River – Class C.

6. Bean Brook (or Swain Brook), Rumford, from its confluence with the Androscoggin River to the dam at the rendering company – Class C.

7. Chapman Brook and its tributaries above the bridge at the highway leading from Bethel to Gilead on the north side of the Androscoggin River – Class A.

8. Childs Brook (Canton) and its tributaries – Class B-2.

9. Ellis River from its confluence with the Androscoggin River to the sawmill dam at East Andover including West Branch of the Ellis River to the sawmill dam at Andover – Class B-2.

10. Keith Brook (Livermore) – Class B-2.

11. Lake Auburn Outflow (Auburn) – Class B-2.

12. Mill Brook, Bethel, from its confluence with the Androscoggin River to the Route 5 Bridge near the Bethel Inn Golf Course – Class B-2.

1977, c. 373, § 10.

13. Nezinscot River, main stem, from its junction with the outlet of South Pond to its confluence with the Androscoggin River – Class B-2.

14. Sabattus River (from Sabattus Lake to limits of Lisbon urban area) – Class C.

15. Sevenmile Stream (Jay) – Class B-2.

1977, c. 373, § 10.

16. Spear Stream, Peru, from the sawmill dam to the Androscoggin River – Class C.

17. Swift River from point at which Mexico-Rumford town boundary leaves the river at Osgood Avenue to the Androscoggin River – Class C.

18. Unnamed Stream (one mile below Livermore Falls Bridge) – Class B-2.

19. Webb River, Dixfield, from the White Bridge to the Androscoggin River – Class C.

20. Whitney Brook (Canton) and its tributaries – Class C.

21. All tributaries, direct and indirect, of the Androscoggin River which are interstate in nature by virtue of having portions of their drainage areas in New Hampshire and portions in Maine are classified as follows:

A. Drainage systems of Wild River in the Township of Gilead – Class B-1.

1965, c. 82, § 1.

B. Drainage systems of Lary Brook and Ingalls Brook in the Townships of Gilead and

Riley – Class B-1.

1965, c. 83, § 1.

Upper Androscoggin Drainage, that portion lying above the most upstream crossing of the Maine-New Hampshire boundary.

1. All waters and segments thereof of the Androscoggin River Drainage System lying above the most upstream crossing of the Maine-New Hampshire boundary and wholly within the State of Maine, not otherwise specified or classified – Class B-1.

2. Cupsuptic Stream and its tributaries above its confluence with Cupsuptic Lake – Class A.

1954, c. 79, § 15.

3. Kennebago Stream and its tributaries above its confluence with Mooselookmeguntic Lake – Class A.

1954, c. 79, § 15.

4. The Magalloway River and its tributaries above the first crossing of the Maine-New Hampshire state line – Class A.

1954, c. 79, § 15.

5. Mill Stream, Rangeley – Class B-2.

1977, c. 373, § 11.

6. All waters and segments thereof of the Androscoggin River Watershed which are interstate in nature by virtue of having portions of their drainage area in New Hampshire and portions in Maine are classified as follows:

A. Waters tributary to the Stearns Brook (Milan & Success, New Hampshire) drainage in the Township of Riley – Class B-1.

B. Waters tributary to the Chickwolnepy Stream (Milan, New Hampshire) drainage in the Township of Grafton – Class B-1.

C. Waters tributary to the Mollidgewock Stream, (Errol, New Hampshire,) drainage in the Township of Upton – Class B-1.

D. Waters, not otherwise classified, tributary to the Umbagog Lake drainage in the Townships of Upton, Grafton, Andover, North Surplus, C-Surplus, Township C and Magalloway Plantation – Class B-1.

E. Waters not otherwise classified tributary to the Magalloway River drainage in the Townships of Magalloway Plantation, Lincoln Plantation, Parkertown, Lynchtown, Parmachenee and Bowmantown – Class B-1.

1965, c. 83, § 2.

Aroostook River Basin

1957, c. 322, § 4

Aroostook River, Main Stem

1. Aroostook River above the junction with St. Croix Stream – Class A.

1954, c. 79, § 15.

2. Aroostook River from junction with St. Croix Stream to injunction with Machias River – Class B-1.

3. Aroostook River from Machias River confluence to the Castle Hill-Ashland Town line – Class B-2.

1967, c. 19, § 1; 1977, c. 373, § 12.

4. Aroostook River from the Ashland-Castle Hill town line to the Wade-Washburn town line – Class B-2.

5. Aroostook River from the Wade-Washburn town line to the crossing of the Aroostook Valley Railroad about 6 miles below Washburn – Class B-2.

1967, c. 19, § 1; 1977, c. 373, § 12.

6. Aroostook River from the crossing of the Aroostook Valley Railroad about 6 miles below Washburn to the junction with Presque Isle Stream – Class B-2.

1977, c. 373, § 13.

7. Aroostook River from the entrance of Presque Isle Stream to the international boundary, except for that portion beginning at a point 100 yards below the intake of the Caribou water supply and extending upstream a distance of 3 miles – Class C.

1967, c. 19, § 1.

8. Aroostook River from a point 100 yards downstream of the Caribou water supply intake to a point 3 miles upstream from this starting point – Class B-1.

Tributaries

1. All tributaries, direct and indirect, of the Aroostook River above the junction with St. Croix Stream – Class A.

1954, c. 79, § 15.

2. All tributaries, direct and indirect, of the Aroostook River from its junction with St. Croix Stream to its junction with the Machias River, unless otherwise specified or classified – Class B-1.

3. All waters of the Aroostook River Basin not otherwise specified or classified which are wholly within the State of Maine – Class B-2.

4. Amsden Brook below the starch factory dam (Fort Fairfield) – Class B-1.

1967, c. 19, § 2.

5. Bryant Brook, Fort Fairfield, from Fisher Street to the Aroostook River confluence – Class B-2.

1967, c. 304, § 10; 1977, c. 373, § 14.

6. Butterfield Brook, Limestone, from the northern fence of Loring Air Force Base to its junction with Limestone Stream – Class B-1.

7. Butterfield Brook above Loring Air Force Base – Class B-1.

8. Caribou Stream from Colby Siding Road Bridge to its confluence with the Aroostook River – Class B-2.

1977, c. 373, § 15.

9. Dudley Brook, Castle Hill, above confluence with North Branch of Presque Isle Stream – Class B-1.

10. Four Corners Brook (Limestone) – Class B-1.

11. Goodrich Brook (also known as Colony Brook), Fort Fairfield, below the dam at the starch factory – Class B-1.

1967, c. 19, § 2.

12. Hardwood Brook (Caribou & Presque Isle) – Class B-1.

13. Libby Brook above the Mapleton-Washburn Road – Class B-1.

14. Limestone Stream, from the Route 165 Bridge in Limestone Village to the Long Road Bridge – Class B-2.

1967, c. 19, § 2; 1977, c. 373, § 16.

14-A. Limestone Stream from the Long Road Bridge to the Canadian border – Class C.

1977, c. 373, § 17.

15. Little Machias River and its tributaries – Class A.

1954, c. 79, § 15.

16. Little Madawaska River and tributaries including Madawaska Lake tributaries above the Route 161 Highway Bridge in Stockholm – Class A.

1954, c. 79, § 15.

17. Machias River and its tributaries above the Garfield Plantation-Ashland town line – Class A.

1954, c. 79, § 15.

18. Machias River, Ashland, from immediately upstream of the starch factory outfall to the Ashland-Garfield Plantation boundary – Class B-1.

19. Machias River, Ashland, from a point immediately above the starch factory outfall to its junction with the Aroostook River – Class B-1.

1967, c. 19, § 2.

20. Otter Brook (Caribou) – Class B-1.

21. Pattee Brook at Fort Fairfield and its tributaries above the dam just upstream of the highway bridge on Route 167 – Class A.

1954, c. 79, § 15.

22. Pattee Brook, Fort Fairfield, from dam at starch factory to confluence with Aroostook River – Class B-1.

1967, c. 19, § 2.

23. Presque Isle Stream and its tributaries above its confluence with, but not including, the North Branch of Presque Isle Stream – Class A.

1954, c. 79, § 15.

24. Presque Isle Stream, from its confluence with the Aroostook River to the Bangor and Aroostook Railroad Bridge nearest Chapman and High Streets in Presque Isle – Class B-2.

1967, c. 19, § 2; 1977, c. 373, § 16.

25. Prestile Brook (Caribou) – Class B-1.

26. Rand Pond Outlet (Presque Isle) – Class B-1.

27. St. Croix Stream and its tributaries above its confluence with the Aroostook River – Class A.

28. Salmon Brook and tributaries upstream of the dam immediately upstream of Washburn Village – Class B-1.

29. Salmon Brook, Washburn, from the dam immediately above the village to its junction with the Aroostook River – Class C.

1967, c. 304, § 10.

30. Silver Springs Brook – Class B-1.

31. Small stream (unnamed) in Presque Isle near vining station on Washburn Road – Class C.

32. Spring Brook, Mapleton, above confluence with North Branch of Presque Isle Stream – Class B-1.

33. Squapan Stream and tributaries above the B. & A. Railroad Bridge – Class A.

Kennebec River Basin

1957, c. 322, § 2

Carrabassett River

1. Carrabasset River, all portions, tributaries and portions of tributaries not otherwise specifically described and otherwise classified – Class B-1.

2. Carrabasset River and its tributaries above a point immediately downstream of its junction with the West Branch of the Carrabasset River in Kingfield – Class A.

1954, c. 79, § 15.

3. Carrabasset River, main stem, from junction with West Branch at Kingfield to a point 1 mile above the railroad bridge in North Anson – Class B-2.

4. Carrabasset River, main stem, from point 1 mile above railroad bridge at North Anson to its junction with the Kennebec River – Class C.

5. Gilman Stream, main stem, from bridge at New Portland to confluence with the Carrabasset River – Class C.

6. Harris Brook, New Portland, below Route 16 in Village of North New Portland to its confluence with Gilman Stream – Class C.

7. Lemon Stream, main stem, from outlet of Mill Pond in New Vineyard to its confluence with Carrabasset River – Class B-2.

8. Mill Stream, Anson, from the railroad bridge in North Anson Village to the confluence with the Carrabasset River – Class C.

9. Stanley Stream, Kingfield – Class C.

Cobbosseecontee Stream Drainage System

1. All water and portions of the Cobbosseecontee Drainage System not otherwise specified or classified – Class B-1.

2. Carleton Pond Outlet and its tributaries from Carleton Pond to Upper Narrows Pond – Class B-2.

1977, c. 373, § 18.

2-A. Cobbosseecontee Stream, main stem, from its confluence with the Kennebec River to the Dam at latitude 44°13.3', longitude 69°47.2' (approximately) – Class C.

1967, c. 304, § 1.

3. Miniwah (Jock) Stream, Wales, and its tributaries – Class B-2.

1977, c. 373, § 18.

4. Outlet Lake Maranacook between Lake Maranacook and Lake Annabessacook – Class B-2.

1977, c. 373, § 18.

5. Tributaries of Lake Annabessacook with the exception of Wilson Stream and others specifically defined – Class B-2.

6. Tributaries of Tacoma Lakes, direct and indirect, and the outlet of Tacoma Lakes to Cobbosseecontee Stream – Class B-1.

1977, c. 373, § 18.

7. Unnamed stream entering Cobbosseecontee Lake through golf course from immediately south of Manchester Village – Class C.

8. Unnamed stream and its tributaries flowing from Loon Pond in Litchfield and the tributaries of Loon Pond – Class B-2.

9. Magotta Meadow Brook and its tributaries entering the southerly extremity of Pleasant Pond from the south – Class B-1.

1977, c. 373, § 18.

10. Unnamed stream and its tributaries entering the cove at the southwest extremity of Pleasant Pond – Class B-2.

11. Unnamed brook and its tributaries entering northerly cove of Lake Maranacook at Readfield across Route 17 – Class C.

12. Wilson Stream (Mud Mills Stream), southerly branch, and tributaries above its junction with the branch from Wilson Pond, including the outlet of Cochnewagan Pond – Class B-1.

1977, c. 373, § 18.

13. Wilson Stream (Monmouth), main stem, from outlet of Wilson Pond to the junction with the branch of Wilson Stream (Mud Mills Stream), entering from the vicinity of Monmouth Village below the tracks of the Maine Central Railroad – Class B-2.

1967, c. 304, § 2; 1977, c. 373, § 19.

14. Wilson Stream (Monmouth), main stem, from the junction with the branch of Wilson Stream (Mud Mills Stream), entering from the vicinity of Monmouth Village below the tracks of the Maine Central Railroad to its entrance to Annabessacook Lake – Class B-2.

1977, c. 373, § 18.

Kennebec River, Main Stem, and those portions of tributaries affected by the rise and fall of the tide from Wyman Dam at Moscow to a line drawn between the most easterly point of land at the southerly end of Popham Beach in Phippsburg and the southernmost extension of Bay Point in Georgetown, not including the Androscoggin River and tributaries northwest of a line formed by the extension of the Brunswick-West Bath town line across Merrymeeting Bay in a northwesterly direction.

1. From Wyman Dam in Moscow to Fall Brook, Solon – Class B-1.

1977, c. 373, § 20.

1-A. From Fall Brook, Solon, to the head of the island immediately below Great Eddy in Skowhegan – Class C.

1977, c. 373, § 21.

2. From the head of the island immediately below Great Eddy to the power company dam in Fairfield – Class B-2.

1961, c. 300.

3. From the power company dam in Fairfield to a point 0.5 miles above the southerly boundary of the Towns of Fairfield and Benton – Class C.

1961, c. 332.

4. From a point 0.5 miles above the southerly boundary of Fairfield and Benton to a line across the Kennebec River Tidal Estuary drawn due west from the southerly extension of Green Point on the easterly shore of the Kennebec River across the channel east of Swan Island to the island, along the easterly shore of Swan Island to southernmost point of the island, thence due west to the westerly shore of the river – Class C.

1967, c. 304, § 3.

5. From a line drawn due west across the Kennebec River Tidal Estuary from the southerly extension of Green Point on the easterly shore of the Kennebec River, across the channel east of Swan Island to the island, along the easterly shore of Swan Island to the southernmost point of the island, thence due west to the westerly shore of the river, to a line drawn across the Tidal Estuary of the Kennebec River, due east, from Abagadasset Point, and including tidal portions of tributaries not otherwise classified – Class C.

6. From a line drawn across the tidal estuary of the Kennebec River, due east from Abagadasset Point, and bounded by a line across the southwesterly arm of Merrymeeting Bay formed by an extension of the Brunswick-West Bath town line across the bay in a northwesterly direction to the westerly shore of Merrymeeting Bay and to a line drawn from Chop Point in Woolwich to West Chop Point in Bath and including tidal portions of tributaries not otherwise classified – Class B-2.

1963, c. 274, § 4.

7. From a line drawn from Chop Point to Woolwich to West Chop Point in Bath to a line across the Kennebec River bearing due west from Bluff Head in the Town of Arrowsic and including tidal portions of tributaries not otherwise classified – Class SB-2.

1963, c. 274, § 4.

8. From a line extending due west from Bluff Head in the Town of Arrowsic to a line drawn between the most easterly point of land at the southerly end of Popham Beach in Phippsburg and the southernmost extension of Bay Point in Georgetown and including tidal portions of tributaries not otherwise classified – Class SB-1.

1963, c. 274, § 4.

9. With respect to subsections 1 to 8, a municipality, sewer district, person, firm, corporation, the State or any subdivision thereof, or other legal entity shall not be deemed to be in violation of section 451 at any time or times prior to October 1, 1976 with respect to any of said classifications if by such time or times he or it, with regard to a project designed to achieve compliance with the applicable classification, shall have completed all of the steps required to be then completed by the following schedule:

A. Preliminary plans and engineers' estimates shall be completed and submitted to the Board of Environmental Protection on or before October 1, 1964.

B. Arrangements for administration and financing shall be completed on or before October 1, 1968. This period, in the case of municipalities, shall encompass all financing including obtaining of state and federal grants.

C. Detailed engineering and final plan formulation shall be completed on or before October 1, 1969.

D. Review of final plans with the Board of Environmental Protection shall be completed and construction commenced on or before October 1, 1970.

E. Construction shall be completed on or before October 1, 1976.

1961, c. 330.

Messalonskee Stream Drainage System

1. All waters and segments of the Messalonskee Drainage System not otherwise specified or classified – Class B-1.

2. Clear Brook, between East Pond and North Pond – Class B-2.

3. Messalonskee Stream, main stem from outlet of Messalonskee Lake to Rice's Rips Dam – Class C.

1967, c. 304, § 4.

4. Messalonskee Stream, main stem, from Rice's Rips Dam to Union Dam in Waterville – Class C.

5. Messalonskee Stream, main stem, from Union Dam to junction with Kennebec River – Class C.

1967, c. 304, § 4.

6. Tributaries of Messalonskee Stream entering between the outlet of Messalonskee Lake and its junction with the Kennebec River – Class C.

Minor Tributaries below Wyman Dam in Moscow

1. All tributaries, direct and indirect, or portions thereof, not otherwise classified or specified, entering the Kennebec River between Wyman Dam and Chop Point in Bath. (This is in no way intended to include the Androscoggin River) – Class B-1.

2. All waters of the Carrabasset Stream System not specifically mentioned or otherwise classified – Class B-2.

3. Austin Stream and its tributaries above the highway bridge on Route 201 in the Town of Bingham – Class A.

1954, c. 79, § 15.

4. Bog Brook, West Athens Vicinity, above confluence with Bradbury Stream – Class B-2.

5. Bond Brook and its tributaries below the crossing of Route 11 prior to reconstruction of this route in 1955 – Class C.

1967, c. 304, § 5.

6. Cold Brook and its tributaries (Wesserunsett Drainage) – Class B-2.

7. Currier Brook, Skowhegan, from Fairview Avenue to its confluence with the Kennebec River – Class C.

8. Fall Brook, Solon, from the dam upstream of Route 201 in Solon Village to its confluence with the Kennebec River – Class C.

9. Greeley Pond Brook, below the outfall of the V.A. Hospital sewage treatment plant to the confluence with Togus Stream – Class B-2.

10. Kennedy Brook, Augusta – Class B-2.

11. Mill Stream, in the village of Norridgewock, below the upstream bridge in the village – Class C.

12. Mill Stream, and tributaries, Norridgewock, above upstream bridge in Norridgewock Village – Class B-2.

13. Sevenmile Stream from the entrance of Webber Pond Outlet to the Kennebec River – Class B-2.

14. Togus Stream, from Greeley Pond Brook junction to the Kennebec River – Class B-2.

15. Twomile Brook, Augusta, from the entrance of the Cushnoc Housing Development sewer to the Kennebec River – Class C.

16. Unnamed stream, in the village of Anson, below its upstream bridge to its confluence with the Kennebec River – Class B-2.

17. Unnamed stream and tributaries crossing Bangor Street in Augusta near Coca Cola bottling plant – Class C.

18. Unnamed tributary of Cathance River in Bowdoinham which enters the tidal portion of the West Branch of the Cathance River approximately 0.7 miles above the bridge in Bowdoinham from a northwesterly direction – Class C.

1967, c. 304, § 5.

19. Webber Pond Outlet, Vassalboro, from Webber Pond to the confluence with Sevenmile Stream – Class B-2.

20. West Branch of Wesserunsett Stream, bewteen Wesserunsett Lake and Smith Pond; including Pain Brook, Kincaid Stream, Haley Stream and Longley Brook – Class B-2.

Sandy River

1. All tributaries, or portions thereof, of Sandy River not otherwise classified or recommended for classifications – Class B-1.

2. Bean Brook, Strong, between its junction with Doctor Brook and with Valley Brook – Class C.

3. Cascade Brook, Farmington, between the Route 2 Bridge and Sandy River – Class B-2.

1977, c. 373, § 22.

4. Lemon Stream, Starks, from dam in Starks Village to its confluence with the Sandy River – Class C.

5. Little Norridgewock Stream and tributaries above confluence with Wilson Stream – Class B-2.

6. Meadow Brook, Wilton, from Depot Street to its confluence with Wilson Stream – Class C.

7. Sandy River and its tributaries above Phillips at the highway bridge on Route 142 – Class A.

1954, c. 79, § 15.

8. Sandy River, main stem, from the Route 142 Bridge in Phillips to Route 4 Bridge in Farmington – Class B-2.

9. Sandy River, main stem, from Route 4 Bridge in Farmington to the entrance of Beales Brook – Class C.

1967, c. 304, § 6.

10. Sandy River, main stem, from the entrance of Beales Brook to its confluence with the Kennebec River – Class C.

11. Temple Stream, between the bridge in the Village of Temple and Sandy River – Class C.

12. Unnamed stream, Farmington, urban area vicinity of Middle Streat – Class C.

13. Unnamed stream, below canning factory in New Sharon Village – Class C.

1967, c. 304, § 6.

14. Valley Brook, Strong, between the Route 145 Bridge and the main stem of the Sandy River – Class C.

15. Wilson Stream, main stem, from outlet of Wilson Pond to the Route 133 crossing – Class C.

1967, c. 304, § 6.

16. Wilson Stream, main stem, from Route 133 crossing to junction with Sandy River – Class C.

Sebasticook River, Main Stem, including East and West Branches.

1. All portions and segments of the main stem of the Sebasticook River not otherwise specified or classified above the dam of the Central Maine Power Company at Winslow – Class B-2.

2. East Branch from outlet of Lake Wassookeag to confluence of tributary entering from Puffers Pond – Class C.

1965, c. 336.

3. East Branch from the junction of Puffers Pond tributary to the outlet of Corundel Lake – Class C.

4. East Branch from outlet of Corundel Lake to Sebasticook Lake – Class C.

1965, c. 336.

5. East Branch from outlet of Sebasticook Lake to Eelweir Bridge – Class C.

1967, c. 304, § 7.

6. West Branch from outlet of Great Moose Lake to Route 43 Bridge in Hartland – Class C.

7. West Branch from Route 43 Bridge in Hartland to its junction with the East Branch – Class C.

1967, c. 304, § 7.

8. Main Stem from Eelweir Bridge to Pittsfield-Burnham town line – Class C.

1967, c. 304, § 7.

9. Main Stem from a point 0.5 mile above the highway bridge at Clinton to a point 1.0 mile above the highway bridge at Benton Falls – Class C.

10. Main Stem below dam of the Central Maine Power Company at Winslow – Class C.

1967, c. 304, § 7.

Tributaries of the Sebasticook River Drainage System

1. All portions and segments of waterways of the Sebasticook River Drainage not otherwise specified or classified – Class B-2.

1967, c. 304, § 8.

2. All tributaries on the west side of the main stem of the Sebasticook River below the dam of the Central Maine Power Company at Winslow – Class B-2.

3. Brackett Brook (Palmyra and Newport) – Class C.

1967, c. 304, § 8.

4. Carlton Stream and tributaries – Class C.

5. China Lake Outlet, main stem, from crossing of East Vassalboro to North Vassalboro road to junction with main stem Sebasticook River – Class C.

1967, c. 304, § 8.

6. China Lake Outlet, main stem, from crossing of highway between East Vassalboro and North Vassalboro to the Outlet of China Lake – Class C.

7. Farnham Brook below Route 100 – Class C.

1967, c. 304, § 8.

8. Fifteenmile Stream and tributaries below its junction with Mill Stream near Albion – Class C.

9. Higgins Brook, main stem, from crossing of Route 154 above Harmony to its outlet to Great Moose Lake – Class C.

10. Joaquin Brook and its tributaries – Class B-1.

11. Meloon Brook and its direct and indirect tributaries – Class B-1.

12. Mill Stream from immediately above crossing of Albion-Benton Road to junction with Fifteenmile Stream – Class C.

1967, c. 304, § 8.

13. Pratt Stream and its tributaries above its junction with Fifteenmile Stream – Class B-1.

14. Puffers Pond tributary and all branches thereof – Class B-1.

15. Sandy Stream, main stem, from its junction with Bacon Brook to a point ½ mile from the entrance of Mussey Brook – Class C.

1967, c. 304, § 8.

16. Sandy Stream, main stem, from outlet of Sandy Pond to its junction with Halfmoon Stream – Class C.

1967, c. 304, § 8.

17. Small streams and tributaries, direct or indirect, not otherwise specified or classified, entering the Sebasticook River from the east between Twentyfivemile Stream and Fifteenmile Stream – Class C.

18. Small streams and their tributaries not otherwise specified entering the Sebasticook River from the east between the outlet of Fifteenmile Stream and the point of discharge of China Lake Outlet – Class C.

Upper Kennebec River Basin,

that portion lying above Wyman Dam in Moscow.

1. All waters tributary to the flowage of Long Falls Dam on the Dead River with the exception of the North Branch of Dead River, the South Branch of Dead River and Stratton Brook – Class B-1.

1965, c. 426. § 2.

2. All waters tributary to the Dead River between Long Falls Dam and its junction with the Kennebec River at the Forks – Class B-1.

1955, c. 426, § 2.

3. Dead River, South Branch, segments and tributaries thereof, not otherwise defined above the normal highwater mark of the reservoir created by the Long Falls Dam – Class B-1.

1955, c. 426, § 2.

4. Kennebec River and tributaries below Moosehead Lake (including East and West Outlets), the sections of Dead River (main stem) below Long Falls Dam, to Wyman Dam in Moscow – Class B-1.

5. Moose River and its tributaries above the outlet of Big Wood Pond in Jackman – Class A.

1954, c. 79, § 15.

6. Moose River, all tributaries, main stem excluded, entering between the outlet of Wood Pond at Jackman and the mouth of Moose River at Moosehead Lake in Rockwood – Class B-1.

7. Moose River, Jackman Plantation, between Big Wood Pond and Long Pond – Class C.

8. Moose River, from entrance to Long Pond to entrance to Moosehead Lake – Class B-1.

9. Moosehead Lake, all tributaries above normal highwater with the exception of Roach River below First Roach Pond Dam and the unnamed streams entering East and West Coves of Moosehead Lake respectively through the Village of Greenville – Class B-1.

1955, c. 426, § 2.

10. North Branch of Dead River and its tributaries above its confluence with Flagstaff Lake – Class A.

1955, c. 426, § 2.

11. Roach River, main stem, First Roach Pond Dam to outlet – Class B-2.

1955, c. 426, § 2.

12. Stratton Brook and its tributaries above the Stratton-Kingfield highway – Class B-1.

1955, c. 426, § 2.

13. Unnamed stream and its tributaries entering Moosehead Lake at East Cove through Greenville Village – Class B-2.

1955, c. 426, § 2; 1977, c. 373, § 23.

14. Unnamed stream and its tributaries entering Moosehead Lake at West Cove through Greenville Junction – Class B-2.

1955, c. 426, § 2; 1977, c. 373, § 14.

Meduxnekeag River Basin

1955, c. 426, § 4

1. All segments and branches of the Meduxnekeag River, not otherwise defined above the international boundary – Class B-1.

2. All segments and tributaries of Prestile Stream, not otherwise defined, above the international boundary – Class C.

1965, c. 42, § 1; 1967, c. 18, § 1.

3. Big Brook tributary, main stem, from the bridge at the Bangor & Aroostook Railroad to the outlet of the stream at the Meduxnekeag River – Class B-2.

1977, c. 373, § 24.

4. Meduxnekeag River, main stem, from outlet of pond at New Limerick downstream to a bridge at the road just upstream of Houlton's compact area leading to gravel pits – Class B-2.

5. Meduxnekeag River, main stem, from bridge at gravel pit entrance just upstream of the compact area in Houlton to the international boundary – Class B-2.

1977, c. 373, § 24.

6. North Branch of the Meduxnekeag River and its tributaries above the Monticello-TCR2 boundary – Class A.

1954, c. 79, § 15.

7. North Branch of the Meduxnekeag River, main stem, from the bridge at U.S. Highway No. 1 to the international boundary – Class B-2.

1957, c. 322, § 2; 1977, c. 373, § 25.

8. Pearce Brook tributary in Houlton – Class B-2.

1967, c. 304, § 11; 1977, c. 373, § 26.

9. Prestile Stream, main stem, from the bridge at Westfield to the international boundary in the Town of Bridgewater – Class C.

1965, c. 42, § 1; 1967, c. 18, § 1.

10. South Branch of the Meduxnekeag River, main stem, from the dam at Hodgdon to the outlet into the main river – Class B-2.

1957, c. 322, § 2; 1977, c. 373, § 25.

11. Whitney Brook and its tributaries above the confluence with Prestile Stream – Class B-2.

1957, c. 322, § 2.

Mousam River Basin

1957, c. 322, § 8

1. All portions of Mousam River drainage not otherwise specified or classified – Class B-1.

2. Hay Brook – Class C.

3. Mousam River, main stem, and tributaries, West Branch from dam at Emery's Mills to northerly boundary of compact area of Sanford about 0.5 mile above Mill Street in the Springvale section – Class B-2.

4. Mousam River, main stem, West Branch, from northerly boundary of compact area of Sanford about 0.5 mile above Mill Street in Springvale section to its junction with the East Branch – Class C.

1971, c. 106, § 1.

Time Schedule

A. A municipality, sewer district, person, firm, corporation or other legal entity shall not be deemed in violation of this subsection at any time or times prior to October 1, 1974 with respect to those classifications if by such time or times he or it with respect to any project necessary to achieve compliance with applicable classification shall have completed all steps required to then be completed by the following schedule.

(1) Preliminary plans and engineers estimates shall be completed and submitted to the Board of Environmental Protection on or before March 1, 1972.

(2) Arrangements for administration and financing shall be completed on or before March 1, 1972. This period, in the case of municipalities, shall encompass all financing including obtaining of state and federal grants.

(3) Detailed engineering and final plan formulation shall be completed on or before October 1, 1972.

(4) Review of final plans with the Board of Environmental Protection shall be completed and construction commenced on or before June 1, 1973.

(5) Construction shall be completed and in operation on or before October 1, 1976.

1971, c. 618.

This reclassification shall not be deemed to exempt any municipality, sewer district, person, firm, corporation or other legal entity from complying with the water quality standards of the last previous classification, as such standards existed on December 31, 1970, and enforcement action may be maintained or noncompliance therewith.

1971, c. 106, § 2.

5. Mousam River, main stem and tributaries entering from west from junction of East and West Branches to tidewater – Class B-2.

6. Mousam River, Middle Branch, from bridge near Yeaton Hill to junction with West Branch – Class B-2.

1967, c. 180, § 3.

7. Mousam River, East Branch, main stem, through Waterboro Village and tributary entering at downstream edge of Waterboro Village – Class C.

Penobscot River Basin

1957, c. 322, § 3

East Branch Penobscot River Drainage System

1. East Branch of the Penobscot River and its tributaries above the outlet of Mattagamon Lake – Class A.

1954, c. 79, § 15.

2. Penobscot River, East Branch, segments and tributaries thereof, between its outlet and the dam at Grand Lake Mattagamon – Class B-1.

1955, c. 420, § 2.

Main Stem, that portion of the Penobscot River between the confluence of East and West Branches south to a line drawn due east from Fort Point on Cape Jellison.

1. The main stem of the West Branch of the Penobscot River from the outlets of Ferguson and Quakish Lakes in Millinocket to the highway bridge just above the junction of the East and West Branches in Medway which carries Route 116 across the West Branch – Class D.

2. The main stem of the Penobscot River and the West Branch from the Route 116 highway bridge in Medway to a line extended in an east-west direction from the outlet of Reed Brook in the Village of Hampden Highlands to the Penobscot River – Class C.

Whereas the segment of the Penobscot River between the junction of the East and West Branches thereof and Weldon Dam is now heavily loaded with cumulative deposits of bacterial cellular and other materials exerting a significant oxygen demand over and above that created by the fermentation of current daily loads and whereas some time will elapse before this source of oxygen demand is stabilized, no abatement action shall be taken by state regulatory agencies, if at that time current daily upstream loadings have been reduced to a value which could reasonable be expected to result in conditions which will not foster cumulative deposits and will be compatible with the specifications or the segment classification, with the added and special provisions that the dissolved oxygen level shall not fall below 7.0 p.p.m. at the Old Town-Milford bridge or at the Stillwater bridge.

3. The tidal estuary of the Penobscot River from a line extended in an east-west direction across the estuary from the mouth of Reed Brook in the Village of Hampden Highlands to a line extended in a westerly direction across from the southernmost tip of Verona Island to the westerly bank of the Penobscot Estuary and from the southernmost tip of Verona Island to the easterly bank of the Penobscot Estuary at the Bucksport-Penobscot town boundary – Class SC.

1965, c. 179, § 1.

Mattawamkeag River Drainage System

1955, c. 426, § 2

1. All segments and tributaries of the Mattawamkeag River Drainage, not otherwise defined, above its outlet to the Penobscot – Class B-1.

2. Baskahegan Stream, main stem, from its outlet to the Mattawamkeag River to the narrows in Crooked Brook Flowage approximately one mile above the village of Danforth – Class C.

3. Cold Brook, a tributary of the Mattawamkeag River, East Branch, entering at Smyrna Mills, the main stem thereof from the confluence with Huntley Mill Pond Brook to its outlet to the East Branch of the Mattawamkeag River – Class B-2.

4. Fish Stream, main stem only, from its confluence with the West Branch of the Mattawamkeag River to the entrance of the Crystal Brook tributary – Class C.

5. Fish Stream, main stem only, from its confluence with Crystal Streams tributary to a point ¼ mile upstream of the Route 11 Bridge in Patten – Class C.

1967, c. 304, § 12.

6. Huntley Mill Pond in Merrill, the main stem from the outlet of Huntley Mill Pond to the confluence with Cold Brook – Class B-2.

7. Mattakeunk Stream, main stem, from the outlet of Dwinal Pond to Mattakeunk Pond – Class C.

8. Mattawamkeag River, main stem, outlet to the junction of the East and West Branches in the Town of Haynesville – Class B-2.

9. Mattawamkeag River, East Branch, between the junction of the East and West Branches in Haynesville and the entrance of the Cold Brook tributary near Smyrna Mills Village – Class B-2.

10. Mattawamkeag River, West Branch, main stem, from the junction of the East and West Branches of the river to, and including, the thoroughfare between the upper and lower Mattawamkeag Lakes – Class B-2.

11. Mattawamkeag River, West Branch, main stem, from its outlet to upper Mattawamkeag Lake to a point 100 feet upstream of the railroad bridge at Island Falls – Class C.

1967, c. 304, § 12.

12. Molunkus Stream, main stem, from its outlet to the Mattawamkeag at Kingman to a point ¼ mile above the highway bridge at Sherman Mills – Class B-2.

13. Webb Brook and its tributaries in the Town of Patten – Class C.

1967, c. 304, § 12

Penobscot River, Minor Tributaries, from the confluence of the East and West Branches to a line drawn due east from Fort Point in the Town of Stockton Springs, not including the Piscataquis and Mattawamkeag Rivers Drainage Systems.

1. All tributaries, direct and indirect, and segments thereof, of the Penobscot River from the confluence of the East and West Branches of the Penobscot, with the exception of the Piscataquis and Mattawamkeag Rivers Drainage Systems, to and including Pushaw Stream on the west shore of the Penobscot River and to and including Blackman Stream on the east shore of the Penobscot River, unless otherwise specified or classified – Class B-1.

1971, c. 273.

2. All minor tributaries on the west shore of the Penobscot River between Pushaw Stream and the Hampden-Winterport line, not otherwise designated – Class C.

3. All minor tributaries on east shore of the Penobscot River between Blackman Stream and the Orrington-Bucksport line, not otherwise designated – Class C.

4. All streams, segments and tributaries thereof, not otherwise defined, entering tidewater between the head of tide on Marsh Stream (Frankfort) and Fort Point (Stockton Springs) – Class C.

5. All minor tributaries, segments, direct and indirect, not otherwise defined, entering tidewater from the head of tide on the Orland River south to a line drawn due east from Fort Point (Stockton Springs) – Class B-1.

6. Cambolasee Stream, Lincoln, from the Route 2 crossing to the Penobscot River – Class C.

7. Great Works Stream and its tributaries above the highway bridge on Route 178 in the Town of Bradley – Class A.

8. Halfmoon Pond (Searsport), and its tributaries above the pond outlet – Class B-1.

9. Kenduskeag Stream and its tributaries above the Bullseye Bridge (Bangor) – Class B-2.

10. Kenduskeag Stream and tributaries below the Bullseye Bridge (Bangor) – Class C.

1967, c. 304, § 13.

11. Marsh River (Prospect), segment and tributaries thereof, not otherwise defined, above tidewater – Class B-2.

12. Marsh Stream (Frankfort, etc.), segments and tributaries, not otherwise defined, above tidewater – Class B-2.

13. Marsh Stream, main stem, from a point 0.4 mile above the bridge at Brooks Village to the inlet of Basin Pond – Class B-2.

1969, c. 286.

14. Marsh Stream, main stem, from its junction with the North Branch of Marsh Stream to the bridge at West Winterport – Class B-2.

12. Piscataquis River, main stem, from Abbot-Guilford town line to the junction with Pleasant River – Class C.

1967, c. 304, §14, 1979, c. 495.

13. Repealed. 1979, c. 495, §4.

14. Piscataquis River, main stem, from junction with Pleasant River to Schoodic Stream confluence – Class B-2.

15. Piscataquis River, main stem, from Abbot-Guilford town line to mouth of Kingsbury Stream – Class B-2.

16. Pleasant River, main stem, from its mouth to the end of Maple Street in Brownville Junction – Class C.

17. Sebec River, main stem, from its mouth to the dam at Main Street in Milo – Class C.

18. Sebec River and its tributaries above the outlet of Monson Stream – Class A.

1954, c. 79, §15.

19. Repealed. 1979, c. 495, §6.

1967, c. 304, §15; 1971, c. 138, §1; 1979, c. 495, §§ 4 to 6.

West Branch Penobscot River Drainage System

1955, c. 322, § 2

1. All waters, tributaries and segments thereof of the Penobscot River Drainage System, not otherwise specified or classified, upstream of the outlets of Ferguson Lake and Quakish Lake and North Twin Dam – Class B-1.

2. Penobscot River and its tributaries above Seboomook Lake – Class A.

1954, c. 79, § 15.

3. That portion of the main stem of the Penobscot River (West Branch) between the outlet of Ferguson Lake and of Quakish Lake and North Twin Dam at the outlet of North Twin Lake or Elbow Lake, which would include the reservoirs known as Quakish Lake and Ferguson Lake – Class B-2.

4. Tributaries, direct and indirect, and segments thereof, of the West Branch of the Penobscot River from the outlet of Quakish and Ferguson Lakes (Millinocket) to its confluence with the East Branch; with the exception of the segments of Millinocket Stream (Millinocket) between the railroad bridge and the West Branch of the Penobscot River – Class B-1.

5. Segments of Millinocket Stream (Millinocket) between the railroad bridge near the Millinocket-Indian Purchase town boundary and the Penobscot River – Class D.

1965, c. 179, § 2.

Schedule of Completion Applicable to Certain Waters of the Penobscot River Basin

1965, c. 179, § 2

1. The classification set forth as follows shall become effective on October 1, 1965.

A. Subsections 1, 2 and 3 under main stem;

B. Subsection 5 under West Branch Penobscot River Drainage System.

2. A municipality, sewer district, person, firm, corporation or other legal entity shall not be deemed in violation of these sections at any time or times prior to October 1, 1976 with respect to those classifications if by such time or times he or it with respect to any project necessary to achieve compliance with applicable classification shall have completed all steps required to then be completed by the following schedule.

A. Preliminary plans and engineers' estimates involving municipal and other publicly owned projects shall be completed on or before October 1, 1968 and plans for required abatement steps by others shall be submitted and approved not later than October 1, 1969.

B. Arrangements for administration and financing shall be completed on or before October 1, 1971. In the case of municipal projects this period is to include definite scheduling of grants-in-aid.

C. Detailed plans and specifications shall be approved by the Board of Environmental Protection and construction begun prior to June 1, 1973.

D. All requirements are to be completed and in operation on or before October 1, 1976.

1965, c. 179, § 2; 1967, c. 475, § 7; 1972, c. 618.

Presumpscot River Basin (Includes all drainage area above the Presumpscot Falls Dam)

1957, c. 322, § 6

1. All waters, tributaries and segments of the Presumpscot River Basin, not otherwise specified or classified, with the exception of the Presumpscot River, main stem, below the upstream compact limits of Westbrook – Class B-1.

2. Frank Brook, and Pleasant River above its confluence with Frank Brook, together with tributaries thereof – Class B-2.

3. Little River, main stem, (Windham) from canning plant on Route 114 to its confluence with the Presumpscot River – Class C.

4. Outflow from Panther Pond to Sebago Lake – Class B-2.

5. Outlet of Tuttle Pond, Windham – Class B-2.

6. Pleasant River, and tributaries between Frank Brook (Gray) and its entrance to Little Sebago Lake – Class B-2.

7. Presumpscot River, main stem, below Village of South Windham to tidewater – Class C.

1967, c. 446.

7-A. Presumpscot River, main stem, from the outlet of Sebago Lake to the dam at Dundee – Class A.

1972, c. 612.

8. Second westerly tributary of the North Branch of Little River (Windham) – Class B-2.

9. Stevens Brook, Bridgton – Class C.

1967, c. 304, § 16.

10. Tannery Brook, and its tributaries, Gorham – Class B-2.

11. Tributaries, direct and indirect, of Songo Pond (Albany vicinity) – Class B-2.

12. Tributaries of Papoose Pond(Waterford) – Class B-2.

13. Tributaries of Coffee and Dumpling Ponds, Casco, above inlet to Pleasant Lake – Class B-2.

14. Unnamed stream, entering Sebago Lake at North Sebago Village – Class B-2.

Saco River Basin

1957, c. 322, § 7

Main Stem, Saco River

1. All portions of the main stem, Saco River, above tidewater not otherwise specified or classified – Class B-1.

2. Saco River, main stem, from Route 5 (Fryeburg-Lovell road) to a point ¾ mile below the Fryeburg-Lovell road – Class B-2.

3. Saco River, main stem, from junction with Ossipee River to the entrance of Quaker Brook – Class B-1.

1973, c. 401.

4. Saco River, main stem, from entrance of Quaker Brook to the Central Maine Power Co. dams at Bar Mills – Class B-1.

1967, c. 180, § 1; 1973, c. 401.

5. Saco River, main stem, from the Central Maine Power Co. dams at Bar Mills to the Route #4-A highway bridge at Salmon Falls village – Class B-2.

1967, c. 180, § 1; 1973, c. 401.

6. Saco River, main stem, from Union Falls Dam to Thatcher Brook – Class B-2.

1973, c. 401.

7. Saco River, main stem, from Thatcher Brook to tidewater – Class C.

1967, c. 180, § 1; 1973, c. 401.

Tributaries, Saco River

1. All tributaries, direct and indirect, and segments thereof, of the Saco River Drainage, above tidewater, not otherwise specified or classified – Class B-1.

2. Brown Brook, Limerick, main stem, from outlet of Holland Pond to junction with Little Ossipee River – Class C.

3. Goodwins Mills Brook, main stem, from 0.5 mile above crossing of Route 35 at Goodwins Mills to Saco River – Class B-2.

4. Kimball Brook, vicinity North Fryeburg, from point 0.5 mile above Route 113 crossing to Charles Pond – Class C.

5. Little River, from crossing of Route 5 approximately 1.0 mile above Cornish Village to its outlet to the Ossipee River – Class C.

1967, c. 180, § 2.

6. Ossipee River, main stem, from 0.5 mile upstream of Route 25 bridge at Kezar Falls to the entrance of Wadsworth Brook – Class C.

1967, c. 180, § 2.

7. Ossipee River, main stem, from entrance of Wadsworth Brook to junction with Saco River – Class C.

8. Wards Brook (Ward Pond to outlet of brook) – Class C.

St. Croix River Basin

1. All tributaries of the St. Croix River upstream from the dam at Calais, the drainage areas of which are wholly within the State of Maine, and including the West Branch of the St. Croix River and its tributaries which enter through Grand Lake Flowage – Class A.

1954, c. 79, § 15.

2. Waters of the St. Croix River Watershed, within the State of Maine, not otherwise classified, including those of the Main Stem of the St. Croix River and of Monument Brook on the Maine side of the international boundary above the Grand Falls Dam – Class B-2.

1967, c. 156; 1977, c. 373, § 27.

3. Waters of the St. Croix River Watershed, within the State of Maine, not otherwise classified, including those of the Main Stem of the St. Croix River on the Maine side of the international boundary from the Grand Falls Dam to the head of tide – Class C.

1977, c. 373, § 27-A.

St. John River Basin

Allagash River Drainage Area – Class A

1954, c. 79, § 15

Fish River Drainage Area

1955, c. 322, § 5

1. All waters, segments and tributaries of the Fish River Drainage not otherwise specified or classified – Class B-2.

2. Fish River and its tributaries above the highway bridge over the Fish River at the outlet of St. Froid Lake on Highway Route 11 – Class A.

1954, c. 79, § 15.

3. Fish River from the bridge at Fort Kent Mills to the Route 11 Bridge near the foot of St. Froid Lake – Class B-1.

4. Fish River main stem, from bridge at Fort Kent Mills to confluence with St. John River – Class C.

1967, c. 304, § 17.

Main Stem, St. John River

1969, c. 268.

1. St. John River, main stem, above the International Bridge Fort Kent – Class B-1.

2. St. John River, main stem, from the International Bridge Fort Kent to the Frenchville-Madawaska town line – Class B-2.

3. St. John River, main stem, from the Frenchville-Madawaska town line to the Canadian border – Class C.

Tributaries, St. John River

1969, c. 268

1. All waters of the St. John Drainage Basin not otherwise specified or classified which receive drainage from lands entirely within the United States – Class B-1.

2. All tributaries direct and indirect of the St. John River not wholly in the State of Maine, on the Maine side of the international border not otherwise specified or classified – Class B-2.

3. All tributaries of the St. Francis River, the drainage areas of which are wholly within the State of Maine – Class A.

4. All tributaries and branches of the St. John River above the outlet of Allagash River, the drainage areas of which are wholly within the State of Maine, including that portion of the river above the St. John Pond Dam – Class A.

5. All streams and tributaries, unless otherwise specified or classified, entering the St. John River in Fort Kent, Frenchville, Madawaska (including the upper portion of Martin Brook), Grand Isle, Van Buren, St. John Plantation and Hamlin Plantation – Class B-2.

6. Martin Brook, Madawaska, downstream of the bridge on the Back Settlement Road – Class C.

7. Negro Brook, Allagash Plantation, and its tributaries – Class A.

8. Oquisiquit Brook and its tributaries, Mars Hill and Easton, above the Canadian border – Class B-2.

9. Riviere des Chutes, Easton and Mars Hill, above the Canadian border – Class B-1.

10. Thibodeau Brook, Grand Isle, from Route 1 to the St. John River – Class C.

11. Violette Brook Van Buren, below the railroad to confluence with Violette Stream – Class C.

12. Violette Stream, Van Buren, below Champlain Street to the junction with the St. John River – Class C.

13. Prestile Stream – Class B-2.

1965, c. 42, § 2; 1967, c. 304, § 18; 1969, c. 268; 1977, c. 373, § 27-B.

Salmon Falls-Piscataqua River Basin

1. Waters not previously classified of the main stem and direct and indirect tributaries of the Salmon Falls and Piscataqua Rivers, within the State of Maine, above tidewater – Class B-1.

1961, c. 321, § 1.

R.S. 1954, c. 79, § 15; 1955, c. 426, §§ 1, 2, 4; 1957, c. 322, §§ 1-8; c. 412; 1961, c. 330; 1963, c. 274, § 4.

§ 369. Coastal streams

1971, c. 470, § 5.

Cumberland County

1959, c. 133, § 1

Those waters draining directly or indirectly into tidal waters of Cumberland County with the exception of the Presumpscot River Drainage Area upstream from the Presumpscot Falls Dam and the Androscoggin River Basin Drainage Area.

1. All coastal streams, direct and indirect segments thereof, draining to tidewater of Cumberland County, not otherwise specified or classified – Class C.

2. **Brunswick.** Unnamed Stream entering tidewater of New Meadows River at Middle Bay – Class A.

3. **Cape Elizabeth.** Alewife Brook – Class A.

4. **Falmouth.** Mill Creek and tributaries thereof – Class B-2.

5. **Falmouth and Portland.** Unnamed Stream forming a portion of the Portland-Falmouth town line and located on the southwesterly shore of the Presumpscot River estuary – Class D.

6. **Freeport.**

A. Harvey Brook – Class B-1.

B. Frost Gully Brook – Class A.

C. Merrill Brook – Class B-2.

D. Merrill Brook and tributaries below Maine Central Railroad crossing to confluence – Class B-1.

E. Collins Brook and tributaries – Class B-2.

F. Mill Stream and tributaries – Class B-1.

G. Kelsey Brook and tributaries – Class C.

H. Little River and tributaries – Class B-2.

D-H added 1979, c. 495.

6-A. **Gray.**

A. Collier Brook – Class B-2.

1965, c. 153.

7. **Portland.** Stroudwater River from its origin to its confluence with Indian Camp Brook Class B-2.

7-A. **Pownal.** Chandler Brook – Class B-2.

1967, c. 17.

8. **Scarboro.**

A. Finnard Brook – Class B-2.

B. Phillips Brook – Class C.

1967, c. 304, §19.

C. Stuart Brook – Class B-2.

9. **South Portland.** Red Brook from the Rye Pond outlet dam to its origin and tributaries thereof – Class B-2.

10. **Yarmouth.**

A. Pratts Brook – Class B-1.

B. Royal River, main stem, from its origin to the head of tidewater (dam) above Main Street, Yarmouth – Class B-2.

1979, c. 495.

Hancock County

1955, c. 426, §§ 1-7

Those waters draining directly or indirectly into tidal waters of Hancock County with the exception of those tributary to the Penobscot River Estuary north of a line drawn due east from Fort Point in Stockton Springs.

1. All coastal streams, direct and indirect segments thereof, draining to tidewaters of Hancock county, not otherwise specified or classified, with the exception of those tributary to the Penobscot River Estuary north of a line drawn due east from Fort Point in Stockton Springs – Class B-1.

2. **Blue Hill.**

A. Carleton Stream, main stem, between First Pond and Second Pond – Class C.

1977, c. 373, § 28.

B. Carleton Stream, main stem, from the outlet of First Pond to tidewater at Salt Pond – Class C.

1977, c. 373, § 28.

C. Unnamed stream at edge of Blue Hill Village entering tidewater near "Big Rock" – Class C.

1967, c. 304, § 20.

D. Unnamed stream flowing from near "Old Cemetery" to the Town Wharf – Class C.

1967, c. 304, § 20.

E. Mill Brook Stream from a point just above the sewer of the consolidated school to its outlet at tidewater – Class B-1.

1977, c. 373, § 28.

F. Unnamed Stream about 100 yards east of Mill Brook Stream – Class C.

1967, c. 304, § 20.

3. **Brooksville.**

A. Outlet of Walker Pond, from the dam at Lymeburner's Mill to tidewater – Class B-2.

B. Shepardson Brook (or Mill Brook), main stem, from Route 176 to its outlet at tidewater – Class C.

4. **Ellsworth.**

A. Card Brook, main stem, from the Farm Pond about 250 yards west of U.S. Highway No. 1 to tidewater – Class B-2.

1963, c. 23.

B. Gilpatrick Brook, main stem, from bridge at U.S. Highway No. 1 to its outlet into the Union River – Class B-2.

C. Union River, main stem, from head of Graham Lake to bridge at U.S. Highway No. 1 at Ellsworth Falls – Class B-2.

D. Union River, main stem, from bridge at U.S. Highway No. 1 at Ellsworth Falls to tidewater – Class C.

E. Unnamed Stream south of Laurel Street in Ellsworth – Class C.

5. **Franklin.** Unnamed Stream flowing near r ilroad station in Franklin Village to Hop Bay – Class C.

6. **Gouldsboro.** All coastal streams, direct and indirect segments, discharging to tidewater on the easterly mainland of Gouldsboro – Class C.

7. **Lamoine.** Spring Brook below washer at Grindle's gravel pit – Class C.

8. **Penobscot.**

A. Clements Brook, main stem, from tidewater to a point 100 feet upstream of Route 166 – Class B-2.

B. Tributary of Winslow Stream entering from the south of South Penobscot Village from its confluence with Winslow Stream to the crossing of Route 177 – Class B-2.

C. Winslow Stream, main stem, from tidewater to dam at the sawmill of S. C. Condon – Class C.

1967, c. 304, § 21.

9. **Sedgwick.**

A. Sargent Brook at Sargentville Village, main stem, from tidewater to a point 300 feet upstream of the highway – Class C.

B. Three Unnamed Streams entering tidewater immediately north of Sedgwick Village – Class C.

C. Unnamed Stream entering tidewater at the head of Salt Pond near North Sedgwick – Class B-2.

10. **Trenton.** Stony Brook from Route 3 crossing to tidewater – Class C.

11. **Waltham.** Webb Brook, main stem, from dam immediately downstream of bridge on Route 179 to its outlet to Graham Lake – Class B-2.

12. **Winter Harbor.** Coastal streams between the southerly point of Schoodic Peninsula to the Winter Harbor-Gouldsboro town line – Class C.

Knox County

1955, c. 426, § 1

St. George River Drainage System

1. All segments and tributaries direct and indirect of the St. George River Drainage System, above tidewater, not otherwise defined or classified – Class C.

2. All segments and tributaries direct and indirect of the St. George River above the outlet of St. George Lake in Liberty – Class B-1.

3. Castner Brook below Hillcrest Poultry Plant – Class C.

4. Crawford Pond Outlet and Crawford Pond tributaries – Class B-1.

5. Fuller Brook and its tributaries – Class B-1.

6. North and South Pond tributaries and outlet to the St. George River – Class B-1.

Other Coastal Streams of Knox County

1. **Camden.**

A. All coastal stream, direct and indirect segments thereof, draining to tidewater in the Town of Camden, not otherwise specified or classified – Class B-1.

B. Megunticook River, main stem, below a point 300 feet above the dam at the Mount Battie Mill – Class B-2.

1977, c. 373, § 29.

2. **Cushing.**

A. All coastal stream, direct and indirect segments thereof, draining to tidewater in the Town of Cushing – Class B-1.

3. **Friendship.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Friendship unless otherwise specified or classified – Class B-1.

B. Goose River, main stem, tidewater to dam at the Herbert Tibbetts' sawmill – Class C.

C. Goose River, main stem, from Tibbetts' sawmill dam to the outlet of Havener Pond – Class B-2.

4. **Owls Head.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Owls Head – Class C.

5. **Rockland.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the City of Rockland – Class C.

6. **Rockport.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Rockport, unless otherwise described or classified – Class C.

B. Goose River and its tributaries below the highway bridge near Simonton Corners – Class B-2.

C. Goose River and its tributaries above the highway bridge near Simonton Corners – Class B-1.

D. Lily Pond outlet – Class B-2.

7. **St. George.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of St. George, unless otherwise described or classified – Class C.

B. Unnamed Stream and its tributaries above tidewater, entering tidewater at the northwesterly corner of Tenant's Harbor – Class B-1.

C. Unnamed Stream and its tributaries, above tidewater, entering tidewater at the head of Long Cove – Class B-1.

8. **South Thomaston.**

A. All coastal streams, direct and indirect segments thereof, drainage to tidewater in the Town of South Thomaston – Class C.

9. **Thomaston.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Thomaston, unless otherwise described or classified – Class B-1.

B. Mill River, main stem, from tidewater to a point ½ mile above tidewater – Class C.

C. Oyster River, main stem, from tidewater to a point 200 feet upstream of Packard's Mill – Class C.

D. Oyster River, main stem, from a point 200 feet upstream of Packard's Mill to the junction with the tributary of which is the outlet of Rocky Pond – Class B-2.

E. Tributary of Oyster River, main stem, coming from Rocky Pond between the Route 17 bridge at West Rockport and the junction with Oyster River – Class B-2.

F. Unnamed Stream flowing from Mace's Pond to Chickawaukee Pond – Class B-2.

10. **Warren.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewaters of the St. George River Estuary unless otherwise specified or classified – Class B-1.

B. Oyster River. See: Thomaston above.

C. Unnamed Stream and its tributaries to St. George River tidewater near Warren-Cushing boundary upstream of a point 500 feet above South Warren-North Cushing Road. Class B-2.

D. Unnamed Stream to St. George River tidewater near Warren-Cushing boundary between a point 500 feet above the South Warren-North Cushing road to tidewater – Class C.

E. Unnamed Stream and its tributaries above tidewater which enters tidewater of the St. George River ½ mile below the South Warren bridge – Class B-2.

11. Other coastal streams. All coastal streams, direct and indirect segments thereof, draining to the tidal waters of Knox County, not otherwise specified or classified – Class B-1.

1973, c. 423, § 4.

Lincoln County

1955, c. 426, § 1

Those waters draining directly or indirectly into tidal waters of Lincoln County.

Damariscotta River Drainage

1. All segments and tributaries of the Damariscotta River, not otherwise defined, above tidewater – Class B-1.

2. Damariscotta River, main stem, from the outlet of Damariscotta Lake to tidewater at Salt Bay – Class B-2.

3. Inlet of Damariscotta Lake at Jefferson Village, from the outlet of the mill pond above Jefferson Village to the lake – Class B-2.

Medomak River Drainage

1. All segments and tributaries of the Medomak River Drainage, not otherwise defined or classified, above tidewater – Class B-2.

2. Repealed. 1965, c. 425, § 22.

3. Tributaries of Little Medomak Brook, principally in the Town of Washington – Class B-1.

Sheepscot River Drainage

1. All segments and tributaries of the Sheepscot River Drainage above tidewater not otherwise defined or classified – Class B-1.

2. Sheepscot River, main stem, from tidewater to junction of East and West Branches – Class B-2.

3. Sheepscot River, West Branch main stem, from outlet of Branch Pond to junction of the East and West Branches – Class B-2.

4. Turner Pond outlet in Somerville Plantation from Turner Pond to Long Pond – Class B-2.

Other Coastal Streams of Lincoln County

1. **Alna.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Alna, not otherwise specified or classified, with the exception of the Sheepscot River Drainage above tidewater – Class B-1.

B. Ben Brook, main stem, downstream of the second road crossing above its mouth – Class B-2.

C. Unnamed Stream and its tributaries entering tidewater of the Sheepscot River at a point approximately one mile due east of the Alna Cemetery – Class B-2.

D. Unnamed Stream at Head Tide Village entering the Sheepscot River about 0.15 mile below the Route 218 crossing – Class B-2.

2. **Boothbay.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Boothbay, not otherwise specified or classified – Class B-2.

B. Adams Pond – Class B-1.

3. **Boothbay Harbor.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Boothbay Harbor, not otherwise specified or classified – Class B-2.

B. Meadow Brook and its tributaries entering Lewis Cove – Class B-1.

C. Unnamed Brook and its tributaries entering the most easterly cove of Campbell Pond – Class B-1.

4. **Bremen.**

A. All coastal streams, direct and indirect segments thereof, draining to tidewater in the Town of Bremen – Class B-1.

5. **Bristol.**

A. All coastal streams entering tidewater between the Bristol-Damariscotta town line and the Bristol-South Bristol town line – Class B-2.

B. All coastal streams entering tidewater between the Bristol-South Bristol town line and the head of tide on the Pemaquid River, not otherwise specified or classified – Class B-2.

C. All coastal streams entering tidewater between the head of tide on the Pemaquid River and the Bristol-Bremen town line, not otherwise specified or classified – Class B-1.

D. Pemaquid River, segments and tributaries thereof, not otherwise defined, above tidewater – Class B-1.

E. Pemaquid River, main stem, entrance to Boyd Pond to tidewater – Class B-2.

F. Pemaquid River, main stem, from dam upstream of Bristol Village to the entrance of Boyd Pond – Class C.

G. Unnamed Stream entering a cove in the tidewater of Pemaquid River immediately west of Pemaquid Village – Class B-1.

H. Unnamed Stream, above tidewater, entering Buck Cove in the Town of Bristol – Class B-2.

6. **Damariscotta.**

A. All coastal streams entering tidewaters of the Damariscotta River – Class B-2.

7. **Dresden.** See: Section 368, Kennebec River.

8. **Edgecomb.**

A. All coastal streams, segments and tributaries thereof, draining to tidewater in the Town of Edgecomb, not otherwise specified or classified – Class B-1.

B. All coastal streams, segments and tributaries thereof, draining to tidewater between (not including) the outlet of Lily Pond to the Edgecomb-Boothbay town line – Class B-2.

C. All coastal streams, segments and tributaries thereof, draining to tidewater between the Edgecomb-Boothbay line on the Damariscotta River and Bennett Neck – Class B-2.

9. **Newcastle.**

A. All coastal streams draining to tidewaters of the Damariscotta River in the Town of Newcastle – Class B-1.

B. All coastal streams draining to tidewaters of the Sheepscot River Estuary in the Town of Newcastle – Class B-1.

10. **Nobleboro.**

A. All coastal streams draining to tidewaters of the Damariscotta River in the Town of Nobleboro – Class B-2.

11. **South Bristol.**

A. All coastal streams and segments thereof draining to tidewaters in the Town of South Bristol, unless otherwise specified or classified – Class B-2.

B. Unnamed Stream entering tidewaters about ½ mile above Prentiss Cove at the Bristol-South Bristol boundary – Class B-1.

12. **Southport.**

A. All coastal streams and segments thereof draining to tidewaters in the Town of Southport – Class B-1.

13. **Waldoboro.**

A. All coastal streams and segments thereof draining to tidewaters in the Town of Waldoboro, except as otherwise specified or classified and with the exception of the Medomak River and its tributaries above head of tide – Class B-1.

1963, c. 54, § 2.

B. Goose River. See: Knox County Coastal Streams.

14. Westport.

A. All coastal streams and segments thereof draining to tidewaters in the town of Westport – Class C.

1967, c. 304, § 22.

15. Wiscasset.

A. All coastal streams and segments thereof draining to tidewaters in the Town of Wiscasset, not otherwise specified or classified – Class B-1.

B. Unnamed Stream and tributaries entering tidewater by way of Chewonke Creek – Class B-2.

C. Unnamed Stream and tributaries in Wiscasset entering the tidal estuary which lies immediately west of Bailey Point – Class B-2.

D. Ward Brook and tributaries – Class B-2.

16. Other coastal streams. All coastal streams, direct and indirect segments thereof, draining to the tidal waters of Lincoln County, not otherwise specified or classified – Class B-1.

1973, c. 423, § 5.

Sagadahoc County

Those streams above tidewater which drain to tidal waters of Sagadahoc County, directly or indirectly, not including that portion of Merrymeeting Bay north and west of the Chops at Bath or those streams draining to the Androscoggin River Estuary – Class C.

1967, c. 304, § 23.

Waldo County

1955, c. 426, §§ 1-7

Those streams above tidewater which drain to tidal waters of Waldo County between the Waldo-Knox County line to Fort Point in Stockton Springs.

1. Coastal streams, segments and tributaries thereof, not otherwise described, above tidewater, entering tidewater between the Knox-Waldo County line and the head of tide on the Little River at the Northport-Belfast boundary – Class B-1.

2. Coastal streams, segments and tributaries thereof, not otherwise defined, above tidewater, entering tidewater between the head of tide on Goose River and Fort Point in Stockton Springs – Class C.

3. Ducktrap River, segments and tributaries thereof, not otherwise described, above tidewater – Class B-1.

4. Goose River (Belfast), main stem, below the upstream crossing of Route 101 – Class C.

5. Goose River (Belfast), segments and tributaries thereof, not otherwise defined, above tidewater – Class B-2.

6. Little River, Northport-Belfast, segments and tributaries thereof, not otherwise defined, above tidewater – Class B-1.

7. Mill Brook and its tributaries in Searsport upstream of a bridge site on an abandoned road about 1½ miles northerly of the village at Searsport which includes McClures Pond and Cain Pond – Class B-1.

8. Mixer Pond (Morrill and Knox) tributaries – Class B-1.

9. Passagassawaukeag River, segments and tributaries thereof, not otherwise defined, above tidewater – Class B-2.

10. Passagassawaukeag River Drainage above the outlet of Ellis Pond, to include Ellis Pond, Halfmoon Pond, Passagassawaukeag Lake and their respective tributaries – Class B-1.

11. Sanborn Pond and Dutton Pond tributaries in Morrill and Brooks – Class B-1.

12. Shaw Brook and its tributaries in Northport – Class B-2.

13. Unnamed Stream entering tidewater at Lincolnville Beach – Class B-2.

14. Unnamed Stream and its tributaries entering tidewater at the northwest corner of Long Cove – Class B-1.

15. Wescott Stream, entering tidewater in Belfast, segments and tributaries thereof not otherwise defined, above tidewater – Class B-1.

16. Other coastal streams of Waldo County. All coastal streams, direct and indirect segments thereof, draining to the tidal waters of Waldo County, not otherwise specified or classified – Class B-1.

1973, c. 423, § 6.

Washington County

1955, c. 426, § 7

Those streams above tidewater which drain to tidal waters of Washington County, directly or indirectly, including those which drain to the tidal waters of the St. Croix River.

1. All coastal streams, segments and tributaries thereof, not otherwise defined, above tidewater, entering the tidal waters of Washington County from the Washington-Hancock County line to and including those to the tidal waters of the St. Croix River – Class B-1.

2. Boyden Stream, main stem, from the outlet of Boyden Pond to the first road crossing below Boyden Pond – Class B-2.

3. Chandler River and its tributaries above the Highway Bridge on Route 1 – Class A.

4. Dennys River and its tributaries above the Highway Bridge on Route 1 in the Town of Dennysville – Class A.

5. Dennys River, main stem, from tidewater to the Bridge at U.S. Highway No. 1 at Dennysville – Class B-2.

6. Dyke Brook, East Branch, main stem in Columbia from tidewater to the crossing of the Maine Central Railroad – Class C.

7. East Machias River and its tributaries above the Highway Bridge on Route 191 – Class A.

8. East Machias River, main stem, from head of tide to a point 2,000 feet upstream of the dam of the Bangor Hydro-Electric Co. – Class C.

9. Machias River and its tributaries above the mill pond at Whitneyville – Class A.

10. Machias River, main stem, from the dam creating the mill pond in Whitneyville to the site of the low dam opposite the ends of West Street and Hardwood Street in Machias – Class B-2.

11. Machias River, main stem, between the site of the low dam approximately opposite the ends of West Street and Hardwood Street in Machias to the head of tide – Class C.

12. Middle River, main stem, between the 2nd upstream crossing of Route 192 and tidewater – Class B-2.

13. Narragaugus River, East and West Branches and their tributaries, above the confluence of the 2 streams – Class A.

14. Narraguagus River, main stem, between tidewater and the bridge of the Maine Central Railroad – Class B-2.

15. Orange River and its tributaries above the highway bridge on Route 1 – Class A.

16. Orange River, main stem, between tidewater and the highway bridge at U.S. Highway No. 1 in Whiting – Class B-2.

17. Pennamaquan River, main stem, between the crossing of the Eastport Branch of the Maine Central Railroad and tidewater – Class B-2.

18. Pleasant River, main stem, from tidewater to a point 1,000 feet above tidewater – Class B-2.

19. Tributary of Tunk Stream, the outlet of Round Pond, from Round Pond to the confluence with the main stem of Tunk Stream – Class B-2.

20. Tunk Stream, main stem, from the bridge at Unionville to tidewater – Class B-2.

21. Unnamed Stream entering northerly end of Brooks Cove in Robbinston – Class C.

22. Unnamed Stream immediately north of Schoolhouse Lane in Robbinston – Class C.

23. Unnamed Stream at easterly edge of Columbia Falls Village from tidewater to Maine Central Railroad near Pleasant River Canning Company plant – Class C.

24. Unnamed Stream entering tidewater portion of St. Croix River at Calais crossing North Street between Beech and Union Streets – Class C.

1971, c. 138, § 2.

25. Unnamed Stream passing through Harrington Village, the segment thereof, between tidewater and a point immediately upstream of the school sewer – Class C.

26. Unnamed Stream flowing through Dennysville Village immediately west of school building – Class B-2.

27. Whitten Parrin Stream in T7, S.D. and Steuben – Class C.

28. Wiggins Brook at South Trescott, main stem, between Route 191 and tidewater – Class C.

York County

1957, c. 322, § 8.

Those streams above tidewater which drain to tidal waters of York County with the exception of those streams draining to the inland waters of the Piscataqua-Salmon Falls River Drainage, the Presumpscot River Drainage, the Mousam River Drainage and the Saco River Drainage.

1. All coastal streams draining directly or indirectly to the tidal waters of the Salmon Falls-Piscataqua River north of Sisters Point in Kittery – Class B-1.

2. All coastal streams above tidewater between Roaring Rock Point (York) and the head of tide on Branch River (Wells) except as otherwise specified or classified – Class C.

3. All coastal streams and their tributaries not otherwise specified between Walker Point (Kennebunkport) and Fletchers Neck in Biddeford – Class C.

4. All coastal streams above head of tide and tributaries thereof not otherwise lesignated or classified entering tidewater from Fletchers Neck, Biddeford, to the York-Cumberland County line – Class B-2.

5. Biddeford-Unnamed streams and tributaries, the main stem of which crosses Route 9 two-tenths of a mile southerly of the intersection of Route 9 and Guinea Road – Class C.

6. Branch River (Brook), Wells – Class B-1. Goosefare Brook (Saco), from its origin ot

7. Goosefare Brook (Saco), from its origin to head of tide – Class C.

8. Josias River Tributary and branches thereof, entering from the north approximately 2½ miles above tidewater – Class B-2.

9. Kennebunk River and tributaries not otherwise classified including streams entering tidewater portion of Kennebunk River – Class B-2.

10. Milliken Brook (Saco) – Class C.

11. Webhannet River and tributaries – Class B-2.

12. West Brook (Biddeford) and tributaries above head of tide – Class C.

13. Coastal streams and direct or indirect tributaries thereof above head of tide, not otherwise designated or classified, which enter the tidal waters of York County – Class B-2.

1970, c. 538, § 1.

R.S. 1954, c. 79, §15; 1955, c. 426, §§1-3, 5, 7; 1957, c. 322, §8; 1959, c. 183, §§1, 2; 1963, c. 23; c. 54, §1; c. 420, §2; 1979, c. 495, §§7, 8.

§ 370. Tidal waters

Cumberland County

1963, c. 274, § 3

All tidal waters of Cumberland County with the exception of those in or bordering on the tidal estuary of the Androscoggin River and Merrymeeting Bay.

1. **Brunswick.**

A. Tidal waters of the Town of Brunswick except those in or bordering on the tidal estuary of the Androscoggin – Class SB-1.

2. **Cape Elizabeth.**

A. All tidal waters of Cape Elizabeth not otherwise specified – Class SB-2.

B. From a point where longitude 70°-16'-40" (just north of Higgins Beach) crosses the Scarboro mainland to a point where longitude 70°-16'-14" (about one mile south of the mouth of the Spurwink River estuary) crosses the Cape Elizabeth mainland, including the Spurwink River estuary to head of tide and tidal tributaries thereof – Class SC.

C. Waters surrounding Richmond Island – Class SB-1.

D. From the southernmost point of land at Mackenney Point to the easternmost point of land on the Cape Elizabeth mainland – Class SA.

E. From a point directly west of Chimney Rock to the Cape Elizabeth-South Portland line – Class SC.

3. **Cumberland.**

A. From Cumberland-Falmouth town line to Cumberland-Yarmouth town line – Class SB-2.

B. Waters of Great Chebeague Island not specifically designated – Class SA.

C. Waters of Great Chebeague Island from the northernmost point of land southeast to latitude 43°-45' (approximately ½ mile of shoreline) – Class SB-1.

D. Waters of Great Chebeague Island in Coleman Cove east of longitude 70°-07'-37" – Class SB-2.

E. Waters surrounding islands in the Town of Cumberland not specifically designated – Class SA.

4. **Falmouth.**

A. Presumpscot River estuary from head of tide to Route 9 crossing – Class SC.

1967, c. 447, § 2.

B. Presumpscot River estuary from the Route 9 crossing to the Route 1 crossing – Class SC.

C. From the Route 1 crossing northeast to a point where longitude 70°-13'-40" (about one mile north of Mackworth Point) crosses the Falmouth mainland – Class SC.

D. Waters surrounding Mackworth Island – Class SC.

E. All other waters of Falmouth not otherwise designated – Class SB-2.

5. **Freeport.**

A. All waters in the Town of Freeport unless otherwise specified – Class SB-1.

B. Harraseeket Harbor upstream from an imaginary easterly line drawn from Stockbridge Point to Moore Point, including Harraseeket River to the confluence with Frost Gully Brook below the Mast Landing Road – Class SB-2.

B added 1979, c. 495.

6. **Harpswell.**

A. All waters in the Town of Harpswell – Class SB-1.

7. **Portland.**

A. All tidal waters within the City of Portland not otherwise specified – Class SC.

B. All tidal waters east of longitude 70°-10' not otherwise specified – Class SA.

C. Northerly shoreline of Fore River and Portland Harbor from the Vaughn Bridge crossing to the most easterly point of land on the Portland mainland – Class SC.

1967, c. 447, §1.

D. All waters west of Grand Trunk Bridge which includes Back Cove – Class SC.

1967, c. 447, § 1.

E. Presumpscot River estuary from head of tide to Route 9 bridge – Class SC.

1967, c. 447, § 1.

F. Waters of Peaks Island from the most northerly point of land on the island to a point where latitude 43°-39'-52" crosses the easterly shoreline (approximately one mile of shoreline) – Class SB-2.

G. Waters of Peaks Island from a point where latitude 43°-39'-52" crosses the easterly shore line to the southernmost point of land on the island – Class SB-1.

H. Waters on the easterly shore of Little Diamond Island from the southernmost point of land to the northernmost point of land on the island – Class SB-2.

I. From the most westerly point of land on Long Island to the most northerly land formation on the island – Class SB-2.

J. From the most northerly land formation on Long Island to the most westerly point of land on the island in a southeasterly direction with the exception of Harbor Grace north of latitude 43°-41'-21" – Class SB-1.

K. Waters of Harbor Grace north of latitude 43°-41'-21" on Long Island – Class SB-2.

L. Waters on the easterly shore line of Cushing Island from the northernmost point of land to the southernmost point of land on the island – Class SB-2.

8. **Scarboro.**

A. All tidal waters in the Town of Scarboro not otherwise designated – Class SB-2.

B. Nonesuch River estuary from head of tide to the B. & M. (Dover line) railroad crossing – Class SC.

C. Little River estuary from head of tide to its confluence with the Scarboro River and tidal tributaries thereof – Class SC.

D. Dunstan River estuary from head of tide to its confluence with the Scarboro River and tidal tributaries thereof – Class SB-1.

1965, c. 84.

E. Libby River estuary from head of tide to a point where longitude 70°-19' (about 1½ miles below the Route 207 crossing) crosses the estuary, and tidal tributaries thereof – Class SB-1.

F. From a point directly west of the most northerly point of land on Shooting Rock Island to a point on the mainland directly north of Cool Rock – Class SA.

1973, c. 267.

G. From a point where longitude 70°-16'-40" crosses the Scarboro mainland to a point where longitude 70°-16'-14" (a point about a mile south of the confluence of the Spurwick River estuary) crosses the Cape Elizabeth mainland including the Spurwink River estuary to head of tide and tidal tributaries thereof – Class SC.

8-A. **South Portland.**

A. All tidal waters in the City of South Portland – Class SC.

1965, c. 425, § 23.

9. **Yarmouth.**

A. All tidal waters of the Town of Yarmouth not otherwise designated – Class SB-2.

B. Waters of Little John Island from the northernmost point of land on the island southeast to a point where longitude 70°-07'-32" intersects the shore line – Class SB-1.

Hancock County

1963, c. 320

1. **Bar Harbor.**

A. Tidewater from a point 500 yards south of Bear Brook to the Mount Dessert-Bar Harbor town line, with the exception of Otter Cove north of latitude 44°-18.75' - Class SA.

B. Tidewaters within the Town of Bar Harbor not specifically mentioned or described – Class SB-1.

1968, c. 516, § 1.

2. **Blue Hill.**

A. Tidewater from Sand Point and southerly a distance of 500 yards – Class SB-2.

B. Tidewaters of Salt Pond – Class SB-1.

C. Tidewater from the most southerly bridge crossing at Salt Pond at the "Nub" northerly and easterly including all bays and estuaries to the Blue Hill-Surry town line – Class SB-1.

D. Tidewater within the Town of Blue Hill not previously mentioned or described – Class SA.

3. **Brooklin.**

A. Tidewaters of Herrick Bay north of a line drawn due east from a point of land at latitude 44°-16'-18" – Class SB-2.

B. Tidewater from the tidal portion of the Benjamin River and including this river, to latitude 44°-15.5' at Center Harbor – Class SB-1.

C. Tidewaters o˜ Salt Bay – Class SB-1.

D. Tidewater within the Town of Brooklin not otherwise mentioned or described – Class SA.

4. **Brocksville.**

A. Tidewater from Blake Point at longitude 68°-48' to a point of land south of Horseshoe Cove at latitude 44°-19.25' – Class SB-1.

1968, c. 516, § 2.

B. Tidewaters forming the estuary known as Bagaduce River in the Town of Brooksville and east of a point of land at approximately N. 44°-24' by W. 68°-46.3' (just south of Lord's Cove) – Class SA.

1963, c. 516, § 2.

C. Orcutt Harbor north of latitude 44°-20.75' – Class SB-1.

1968, c. 516, § 2.

D. Easterly shoreline of Bucks Harbor from latitude 44°-20'-10" south of longitude 68°-44.5' – Class SB-2.

E. Westerly shoreline of Bucks Harbor south of latitude 44°-20'-10" easterly to longitude 68°-44.5' at Norembega – Class SB-2.

F. Tidewater of Bucks Harbor north of latitude 44°-20'-10" – Class SB-1.

1968, c. 516, § 2.

G. Tidewater along the shoreline at Norembega from longitude 68°-44.5' southeast to longitude 68°-43.25' – Class SB-1.

1968, c. 516, § 2.

H. Tidewater from longitude 68°-42.25' near Herricks Village to Sedgwick-Brookville town line – Class SB-1.

I. Tidewater within this town along its southerly shoreline from Blake Point to the Sedgwick town line not previously mentioned or described – Class SA.

J. Tidal waters from a point of land just south of Lord's Cove at approximately N. 44°-24' by W. 68°-46.3' on the Bagaduce Estuary around Cape Rosier to Blake's Point – Class SB-1.

5. **Castine.**

1965, c. 179, § 3; 1968, c. 516, § 2.

A. Tidewaters in the Town of Castine between a point on Dice Head due south of the lighthouse to the point of land at approximately N. 44°-24', W. 68°-47' – Class SB-2.

1963, c. 274, §3.

B. Tidewaters of the estuary known as Bagaduce River bordering on Castine east of a point of land at approximately N. 44°-24' – Class SA.

C. Tidal waters of Castine bordering the Penobscot River Estuary between the Penobscot-Castine boundary and a point on Dice Head due south of the lighthouse – Class SB-1.

1965, c. 179, §4. Chapter revised 1979, c. 495.

6. **Deer Isle.**

A. Tidewater bordering the settled area of Eggemoggin between longitude 68°-44' and latitude 44°-18.25' – Class SB-1.

B. Tidewater of Blastrow Cove in Little Deer Isle – Class SB-2.

1969, c. 121, § 1.

C. Tidewaters on the westerly shoreline south of latitude 44°-14.25' to the Deer Isle-Stonington town line, including Northwest Harbor, Pressey Cove and Sheephead Island – Class SB-1.

D. Tidewater from the Stonington-Deer Isle town line at the Holt Pond outlet to the northeasterly point of land at latitude 44°-13.25' at Greenlaw Cove and including Stinson Neck – Class SB-1.

E. Tidewaters of Town of Deer Isle not otherwise mentioned or described – Class SA.

7. **Ellsworth.**

A. All tidal waters within the City of Ellsworth – Class SB-1.

1968, c. 516, § 3.

8. **Franklin.**

A. All tidal waters within the Town of Franklin – Class SB-1.

9. **Gouldsboro.**

A. All tidewaters within the Town of Gouldsboro – Class SB-1.

1968, c. 516, § 4.

10. **Hancock.**

A. Tidewaters of Hancock north and westerly of a line drawn due west from Pecks Point in waters known as Kilkenney Cove, Skillins River and Youngs Bay – Class SB-2.

1967, c. 153, § 1.

B. Tidewaters of the Town of Hancock not otherwise specified or described – Class SB-1.

1967, c. 153, § 1; 1968, c. 516, § 5.

11. **Lamoine.**

A. Tidewaters from the Hancock-Lamoine town line at Kilkenney Cove south to a line drawn due west from Pecks Point in the Town of Hancock – Class SB-2.

1967, c. 153, § 2.

B. Tidewaters wtihin the Town of Lamoine not otherwise specified or classified – Class SB-1.

1967, c. 153, § 2.

12. **Mount Desert.**

A. Tidewater from Otter Cove south of latitude 44°-18.75' to Ingraham Point – Class SA.

B. Tidewaters within the Town of Mt. Desert not otherwise specified or classified – Class SB-1.

1968, c. 516, § 6.

13. Penobscot.

A. Tidewaters of the estuary known as Bagaduce River bordering on Penobscot – Class SA.

B. Tidal waters of Penobscot bordering the Penobscot River estuary – Class SB-1.

1965, c. 179, § 5.

14. Sedgwick.

A. Tidewaters of the estuary known as Bagaduce River bordering on Sedgwick – Class SA.

B. Remaining tidewaters within the Town of Sedgwick not otherwise specified or classified – Class SB-1.

15. Sorrento.

A. All tidewaters within the Town of Sorrento – Class SB-1.

16. Southwest Harbor.

A. All tidewaters within the Town of Southwest Harbor – Class SB-1.

1968, c. 516, § 7.

17. Stonington.

A. Tidewater from the Moose Island Bridge to Ames Pond outlet including waters of Moose Island – Class SB-2.

1969, c. 121, § 2.

B. Tidewaters within the Town of Stonington not otherwise specified or classified – Class SB-1.

1969, c. 121, § 2.

18. Sullivan.

A. All tidal waters within the Town of Sullivan – Class SB-1.

19. Surry.

A. All tidal waters within the Town of Surry – Class SB-1.

20. Swans Island.

A. All tidal waters within the Town of Swans Island – Class SB-1.

1968, c. 516, § 8.

21. Tremont.

A. All tidal waters within the Town of Tremont – Class SB-1.

1968, c. 516, § 8.

22. Trenton.

A. All tidal waters within the Town of Trenton – Class SB-1.

1968, c. 516, § 8.

23. Winter Harbor.

A. All tidal waters within the Town of Winter Harbor – Class SB-1.

1968, c. 516, § 8.

24. Cranberry Isles.

A. All tidal waters within the Town of Cranberry Isles – Class SB-1.

1967, c. 475, § 8; 1968, c. 516, § 9.

25. Long Island Plantation.

A. All tidal waters within Long Island Plantation – Class SB-1.

1968, c. 516, § 9.

26. **Exceptions.**

A. A municipality, sewer district, person, firm, corporation or other legal entity shall not be deemed subject to penalty under this subchapter at any time prior to October 1, 1976 with respect to any of said classification in Hancock County if by such time he or it, with regard to facilities designed to achieve compliance with the applicable classification shall have completed all the steps required to be then completed by the following schedule:

1) Preliminary plans and engineer's estimates shall be completed and submitted to the Board of Environmental Protection on or before October 1, 1969.

2) Arrangements for administration and financing shall be completed on or before October 1, 1971.

3) Detailed engineering and final plan formulation shall be completed on or before January 1, 1972.

4) Detailed plans and specifications shall be approved by the Board of Environmental Protection and construction begun prior to June 1, 1973.

5) Construction shall be completed and in operation on or before October 1, 1976.

1968, c. 516, § 10; 1972, c. 618.

Knox County

1963, c. 274, § 3

General classification – Tidewaters of Knox County not otherwise specifically designated – Class SA (includes: Cushing, Warren, Thomaston).

1. **Camden.**

A. Tidewater bordering Camden from Northeast Point to Ogier Point except that assigned to Class "C" – Class SB-1.

B. Tidewater bordering Camden from Metcalf Point to Eaton Point – Class SC.

2. **Cushing.**

A. Tidewaters bordering Cushing – Class SA.

3. **Friendship.**

A. Tidewaters of Friendship Harbor north of a line drawn from the point of land opposite the northerly tip of Garrison Island to Jameson Point – Class SC.

4. **North Haven.**

A. Shoreline of North Haven for ½ mile east of the point of land on the eastern side of Brown's Cove – Class SB-2.

5. **Owls Head.**

A. Tidewaters from the point of land immediately southwest of Cresent Beach to the Owls Head-Rockland town line – Class SC.

6. **Rockland.**

A. All tidewaters in the City of Rockland – Class SC.

7. **Rockport.**

A. Rockport Harbor north of a line extended due east from end of Sea Street, near Harkness Brook – Class SC.

B. Tidewater from Rockland Town Line to the next point of land to the north – Class SC.

C. Clam Cove in Rockport from Brewster Point to Pine Hill – Class SB-2.

D. Rockport Harbor north of a line due west of Beauchamp Point except that portion assigned to Class "C" – Class SB-1.

8. **St. George.**

A. Tidewaters between a point 100 yards south of the cannery at Port Clyde and the point of land west of Fish Cove – Class SC.

B. Tennants Harbor west of a North-South line at the harbor entrance (approximately longitude 69°-12′ W.) – Class SC.

C. Small Cove just northeast of Tenants Harbor, north of a line drawn due west from point of land forming east side of cove – Class SB-1.

D. Tidewaters between Marshall Point and Hooper Point not assigned to Class "C" – Class SB-1.

9. **South Thomaston.**

A. Northerly cove of Seal Harbor near Sprucehead – Class SB-1.

B. Shoreline St. George River south of Hospital Point – Class SB-1.

C. Weskeag River north of a line due west from Hayden Point – Class SB-1.

10. **Thomaston.**

A. All tidal waters bordering Thomaston – Class SA.

11. **Vinalhaven.**

A. Tidewaters of Carvers Harbor and Sand Cove from the point on the south side of Sand Cove to the bridge to Lane Island – Class SB-2.

12. **Warren.**

A. Tidewaters of Oyster River – Class SB-1.

B. Tidal waters of Warren not otherwise specified – Class SA.

Lincoln County

1963, c. 274, § 3

1. **Alna.**

A. All tidal waters within the Town of Alna – Class SB-1.

1963, c. 320.

2. **Boothbay.**

A. All tidewaters within the Town of Boothbay – Class SB-1.

3. **Boothbay Harbor.**

A. Tidal waters bordering the Town of Boothbay Harbor northerly of a line drawn due east from the point of land off Commercial Street nearest McFarland Island – Class SB-2.

B. Tidal waters not otherwise classified within the Town of Boothbay Harbor – Class SB-1.

4. **Bremen.**

A. All tidewaters within the Town of Bremen – Class SA.

5. **Bristol.**

A. All tidewaters not otherwise described or classified within the Town of Bristol – Class SA.

B. Pemaquid Harbor and New Harbor, including back cove in Bristol from Fish Point to a point 100 yards east of Gilbert's Wharf – Class SC.

C. Tidewaters in the Town of Bristol from Fish Point to the point of land east of Johns River, except that segment assigned to Class SC, and Round Pond Harbor inside the closest points on north and south – Class SB-2.

D. Tidewater of Long Cove north of a line drawn due west from the point of land extending southward on the east side of the cove – Class SB-1.

6. **Damariscotta.**

A. All tidal waters not otherwise described or classified – Class SB-1.

B. Tidewaters from latitude 44°-2.7' (near present Route # 1 Bridge) south to latitude 44°-1.6' (south of Day Cove) – Class SB-2.

1967, c. 304, § 24.

7. **Edgecomb.**

A. All tidal waters bordering the easterly shoreline of Edgecomb – Class SA.

B. All tidal waters bordering the westerly shoreline of the Town of Edgecomb – Class SB-1.

1963, c. 320.

8. **Newcastle.**

A. Tidal waters not otherwise classified or described within the Town of Newcastle on its easterly shoreline – Class SA.

B. Tidewater from head of tide at Damariscotta Mills in Newcastle south to the Railroad Bridge – Class SB-2.

1967, c. 304, § 25.

C. All tidal waters bordering the westerly shoreline of the Town of Newcastle – Class SB-1.

1963, c. 320.

D. Tidewaters from the Railroad Bridge at Damariscotta Mills south to a point at latitude 44°-2.7' (near present Route #1 Bridge) – Class SB-1.

1967, c. 304, § 25.

E. Tidewaters from a point at latitude 44°-2.7' (near present Route #1 Bridge) to a point of land at latitude 44°-1.6' (about ½ mile above Little Point) – Class SB-2.

1967, c. 304, § 25.

F. Tidewaters of the Damariscotta River from a point of land at latitude 44°-1.6' (about ½ mile above Little Point) south of Little Point – Class SB-1.

1967, c. 304, § 25.

9. **Nobleboro.**

A. Head of tide at Damariscotta Mills in Nobleboro to Railroad Bridge – Class SB-2.

1967, c. 304, § 26.

B. Tidewaters in Nobleboro not otherwise classified or described – Class SB-1.

10. **South Bristol.**

A. All tidewaters within the Town of South Bristol not otherwise classified or described – Class SA.

B. Tidewaters south of a line drawn due east from Jones Point except waters around Inner Herron Island and Thrumpcap Island – Class SC.

11. **Southport.**

A. All tidal waters bordering on the Town of Southport – Class SB-1.

1963, c. 320.

12. **Waldoboro.**

A. All tidewaters within the Town of Waldoboro not otherwise classified or described – Class SA.

B. Tidewaters north of a line drawn from Hoffses Pt. to Waltz Pt. – Class SB-1.

13. **Westport.**

A. All tidal waters within the Town of Westport – Class SB-1.

1963, c. 320.

14. **Wiscasset.**

A. All tidal waters within the Town of Wiscasset – Class SB-1.

1963, c. 320.

Sagadahoc County

1. General classification.

A. All tidal waters of Sagadahoc County not otherwise classified or described, with the exception of Merrymeeting Bay north and west of the Chops and the Kennebec River, tidal estuary, from the Chops, so called, southerly to a line drawn between the most easterly pont of land at the southerly end of Popham Beach in Phippsburg and the southernmost extension of Bay Point in Georgetown – Class SB-1.

1963, c. 274, § 3.

2. Other category.

A. Tidal waters of the Sasanoa River bordering the Town of Arrowsic, between the Kennebec River and Upper Hell Gate – Class SB-2.

1961, c. 273.

B. Tidal waters bordering the Town of Woolwich between the junction of the so-called Sasanoa River and the Kennebec River and Upper Hell Gate on the Sasanoa River – Class SB-2.

1961, c. 273.

Waldo County

1963, c. 274, § 3

1. **Belfast.**

A. Tidewaters from the Northport-Belfast town line to "The Battery" in Belfast – Class SB-1.

B. Tidewaters between "The Battery" and a point opposite the swimming pool at the city park – Class SB-2.

C. Tidewaters between a point opposite the swimming pool at Belfast city park and the mouth of Goose River, except for portions otherwise classified or described – Class SC.

1967, c. 155.

D. The portion of the tidal estuary of the Passagassawamkeag River upstream at the site of a bridge about one mile upstream of the Route #1 Bridge at Belfast – Class SB-2.

E. Tidewaters between Goose River and the Searsport-Belfast town line – Class SB-1.

2. **Frankfort.**

3. **Islesboro.**

A. Tidewaters within the Town of Islesboro not otherwise designated or classified – Class SA.

B. Tidewaters from Marshall Pt. to Coombs Pt. – Class SB-1.

C. Dark Harbor inside the tidal dam – Class SB-1.

D. Segment of coast between Grindle Pt. and the point of land to the east of Broad Cove – Class SB-1.

4. **Lincolnville.**

A. Tidewaters within the Town of Lincolnville not otherwise described or classified – Class SA.

B. Tidewater creek or estuary of small stream which rises near Carver's Corner – Class SC.

C. Tidewaters between the Islesboro Ferry wharf, Lincolnville, and a point 1,000 feet north of the tidewater creek at Lincolnville Beach, or estuary of small stream, which rises near Carver's Corner except for the waters of this tidal creek – Class SB-2.

D. Tidewater of the mouth of Ducktrap River from the head of tide to a point approximately 1,000 feet southeasterly of Route 1 – Class SB-1.

5. **Northport.**

A. All tidewaters within the Town of Northport not otherwise described or classified – Class SA.

B. Tidewaters between Saturday Cove and the Northport-Belfast town line – Class SB-1.

6. **Searsport.**

A. All tidewaters within the Town of Searsport not otherwise described or classified – Class SA.

B. Tidewater from Belfast-Searsport town line and the point of land in Searsport Harbor which is formed by the landing or wharf at the end of Steamboat Avenue – Class SB-1.

C. From the wharf at the end of Steamboat Avenue in Searsport to a point opposite the site of the Searsport Railroad Station – Class SB-2.

D. Tidewaters between a point opposite the site of the Searsport Railroad Station and a point 100 yards east of the wharf at Summers Fertilizer Company – Class SB-1.

7. **Stockton Springs.**

A. Tidewater from Ft. Point westerly to the Stockton Springs-Searsport town line – Class SB-1.

B. From a point on the westerly bank of the Penobscot River Estuary at a point where a line drawn in a westerly direction through the southernmost point of Verona Island intersects this bank southerly to Fort Point on Cape Jellison – Class SB-1.

1965 c. 179 § 6.

8. **Effective date.**

A. The classifications set forth in subsection 7 shall become effective on October 1, 1965, A municipality, sewer district, person, firm, corporation or other legal entity shall not be deemed in violation of these sections at any time or times prior to October 1, 1976 with respect to those classifications if by such time or times he or it with respect to any project necessary to achieve compliance with the applicable classification shall have completed all steps required to then be completed by the following schedule:

(1) Preliminary plans and engineers' estimates involving municipal and other publically owned projects shall be completed on or before October 1, 1968 and plans for required abatement steps by others shall be submitted and approved not later than October 1, 1969.

(2) Arrangements for administration and financing shall be completed on or before October 1, 1971. In the case of municipal projects this period is to include definite scheduling of grants-in-aids.

(3) Detailed plans and specifications shall be approved by the Board of Environmental Protection and construction begun prior to June 1, 1973.

(4) All requirements are to be completed and in operation on or before October 1, 1976.

1965, c. 179, § 8; 1967, c. 475, § 9; 1972, c. 618.

Washington County

1963, c. 274, § 3

1. **Addison.**

A. All tidewaters of Addison not otherwise described or classified – Class SA.

B. Tidewaters between a line extending due east from Whites Pt. to the east side shore and the Columbia Falls-Addison town boundary – Class SB-1.

C. Tidewaters in Addison north of a line across the estuary of Indian River 100 yards below the Route 187 Bridge at Indian River Village – Class SB-2.

2. **Beals.**

A. Tidewaters of Beals not otherwise classified – Class SA.

B. Tidewaters around the northern end of Beals Island between Indian Pt. and the point of land on Beals Island nearest French House Island – Class SB-2.

3. **Calais.**

A. Tidewaters from the Calais-Robbinston town line to a point of land immediately upstream of Devils Head in Calais – Class SB-2.

B. Tidewaters of the St. Croix River estuary from the point of land immediately upstream of Devils Head in Calais to the head of tide also in Calais – Class SC.

4. **Cherryfield.**

A. Tidewaters of Narraguagus River estuary – Class SC.

5. **Columbia.**

A. Tidewaters of West Brook estuary above the Columbia-Addison town boundary – Class SB-2.

6. **Columbia Falls.**

A. Tidewater portions of the Pleasant River above the Columbia Falls-Addison town boundary – Class SC.

7. **Cutler.**

A. Tidewaters within the Town of Cutler not otherwise classified – Class SA.

B. Tidewaters of Cutler Harbor inside a line running northeast from the point of land approximately at N. 44°-39.3' and W. 67°-12.4' – Class SB-2.

C. Tidewaters of Money Cove inside the tidal falls – Class SB-1.

8. **Dennysville.**

A. Tidewaters within the Town of Dennysville not otherwise classified – Class SB-1.

B. Tidewaters of Dennys Bay and River west of Hinckley Pt. – Class SC.

9. **East Machias.**

A. All tidewaters within the Town of East Machias – Class SC.

10. **Eastport.**

A. Tidewaters of Eastport not otherwise classified or described – Class SA.

B. Tidewaters of Bar Harbor in Eastport from the fill between northwesterly point of Moose Island and Carlow Island and the old highway bridge from the mainland to Moose Island – Class SB-2.

C. Tidewaters of Carryingplace Cove, east of a line drawn from the point of land at approximately N. 44°-55.3', W. 67°-01.7' to the point of land at approximately N. 44°-55.3', W. 67°-01.7' – Class SB-1.

D. Tidewaters of Prince Cove west of a line extending from Estes Head to the most southerly extension of the point of land on which Country Road, so called, is located – Class SB-2.

E. Tidal waters not otherwise classified between Shackford Head and the point of land near Dog Island in Eastport – Class SC.

11. **Edmunds.**

A. Tidewaters of Edmunds not otherwise classified or described – Class SA.

B. Orange River estuary and Whiting Bay from a line drawn across the bay in a northwesterly and southwesterly direction through Wilbur Pt. and the easterly boundary of Whiting – Class SB-1.

C. Tidewaters of Dennys River in Edmunds west of a line drawn due south from Hinckley Point in Dennysville – Class SC.

D. Tidewater of Dennys River Estuary and Bay east of a line drawn due south from Hinckley Pt. in Dennysville to a point of land at approximately N. 44°-54.5' W. 67°-11.7' – Class SB-1.

12. **Harrington.**

A. All tidewaters of Harrington not otherwise described or classified – Class SA.

B. Tidewaters of Mill River and Cole Creek Estuary northwesterly of Oak Pt. in Harrington – Class SB-1.

C. Tidewaters bordering Harrington west and south of a line across the Harrington River at a point 1,000 feet down-river of the canning factory in Harrington – Class SC.

D. Tidewaters west and north of a line across the Harrington River drawn due east from Oliver Lord Pt., except those west and north of a line across the Harrington River at a point 1,000 feet downriver of the canning factory at Harrington – Class SB-2.

13. Jonesboro.

A. All tidewaters in Jonesboro not otherwise described or classified – Class SA.

B. Tidewaters of the Chandler River in Jonesboro upstream of a line drawn normal to the stream at a point 2/10 mile below the Route #1 Bridge at Jonesboro Village – Class SC.

C. Tidewaters along the Chandler River in Jonesboro between a line normal to the stream at a point 2/10 mile below the Route #1 Bridge at Jonesboro Village and a line drawn from Carlton Point to Deep Hole Point – Class SB-1.

14. Jonesport.

A. All tidewaters of Jonesport not otherwise described or classified – Class SA.

B. Tidewaters in Jonesport north of a line across the estuary of Indian River 100 yards below the Route 187 Bridge at Indian River Village – Class SB-2.

C. Tidewaters between Hopkins Pt. and Indian Pt. – Class SC.

15. Lubec.

A. All tidewaters of Lubec not otherwise described or classified – Class SA.

B. Tidewaters of Bailey's Mistake west of a line drawn due north from Balch Head in the Town of Trescott – Class SB-1.

C. Tidewaters between a point 1,000 yards westerly of Leadurney Pt. and a point 100 yards south of the creek entering tidewater approximately 2/10 mile south of Woodward Pt. – Class SB-1.

D. Tidewaters between Leadurney Pt. and a point 1,000 yards westerly along the shore – Class SB-2.

E. Tidewaters between Leadurney Pt. and a point of land approximately N. 44°-51.2' and W. 67°-00.3' – Class SC.

F. Tidewaters between the site of the North Lubec Ferry landing and a point of land at approximately 44°-51.2' and W. 67°-00.3' – Class SB-1.

16. Machias.

A. All tidewaters within the Town of Machias not otherwise specified or classified – Class SC.

B. All tidewaters of Little Kennebec Bay – Class SA.

17. Machiasport.

A. All tidewaters not otherwise described or classified – Class SA.

B. Tidewaters of Machias and East Machias Rivers north of a line drawn from Ft. O'Brien Pt. to Randall Pt. in Machiasport – Class SC.

18. Millbridge.

A. All tidewaters of Millbridge not otherwise described or classified – Class SA.

B. Tidewaters north and west of a line from Fish Pt. to the point of land approximately N. 44°-31.8' by W. 67°-52.5' – Class SC.

C. Tidewaters of Wyman Cove from Mitchell Pt. to a wharf location approximately 0.4 mile northerly from Mitchell Pt. – Class SB-2.

D. Tidewaters north and west of a line from Timmy Pt. to Fickett Pt., except those defined as Class "SC" – Class SB-1.

E. Tidewaters of the Mill River and Cole Creek estuary southwesterly, westerly and northerly of Blasket Pt. – Class SB-1.

19. Pembroke.

A. Tidewaters of Pembroke not otherwise described or classified – Class SA.

B. Tidewater estuaries of Cobscook River and Wilson Stream in Pembroke lying north and west of a line drawn from the point of land at approximately N. 44°-54.5' W. 67°-11.7' in Edmunds, due northeasterly to the Pembroke shore except those portions in Dennysville described as lying west of a line drawn due south from Hinckley Pt. – Class SB-1.

C. Tidewaters of Pennamaquam River and Meadow Brook estuaries in Pembroke north and west of a line drawn due east and west through a point of land at N. 44°-56.5', W. 67°-10' – Class SC.

D. All waters of Hersey Cove and tidewaters of the Pennamaquam River north and west of a line drawn due south from the headland forming the easterly side of the entrance to Hersey Cove to Leighton Neck – Class SB-1.

20. Perry.

A. All tidewaters of Perry not otherwise described or classified – Class SA.

B. Tidewaters of small cove, the first cove westerly of Eastport Branch of Maine Central Railroad, southwest of Pleasant Point school at Pleasant Pt. – Class SB-1.

C. Tidewaters of Little River above the Route #1 Bridge – Class SB-2.

21. Robbinston.

A. Tidewaters from Liberty Pt. north to Calais-Robbinston town line – Class SB-2.

B. Tidewaters from Liberty Pt. south to Robbinston-Perry town line – Class SB-1.

22. Roque Bluffs.

A. All tidewaters within the Town of Roque Bluffs – Class SA.

23. Steuben.

A. All tidewaters within the Town of Steuben – Class SA.

24. Trescott.

A. All tidewaters not otherwise described or classified within the Town of Trescott – Class SA.

B. Tidewaters of Bailey's Mistake in the Town of Trescott west of a line drawn due north from Balch Head in Trescott – Class SB-1.

C. Tidewaters of Whiting Bay, between a line drawn across the bay northwesterly and southeasterly through Wilbur Pt. and the easterly boundary of Whiting – Class SB-1.

25. Whiting.

A. Tidewaters of Whiting not otherwise described or classified – Class SA.

B. Tidewaters southwesterly of the easterly boundary of Whiting – Class SB-2.

C. Tidewaters of Holmes Bay for a distance of 100 yards around the canning factory – Class SB-2.

York County

1963, c. 274, § 3

1. Biddeford.

A. Tidewaters of Biddeford not otherwise defined or classified – Class SB-2.

B. Estuary of Little River north of latitude 43°-24'-04" to head of tide, including tidal tributaries thereof – Class SC.

C. From the southernmost point of land on the Biddeford mainland to a point where latitude 43°-25'-07" crosses the mainland – Class SA.

D. From a point where 43°-25'-33" crosses the mainland to a point where latitude 43°-26'-05" crosses the mainland – Class SB-1.

E. From a point where latitude 43°-26'-05" crosses the mainland to a point directly north of the Coast Guard Station at Fletchers Neck – Class SA.

F. From the most easterly point of land on the Biddeford mainland to a point directly west of the most northerly point of land on Basket Island, including tidewaters of "The Pool" – Class SB-1.

1967, c. 154, § 1.

G. Tidewaters from a point directly west of the most northerly point of land on Basket Island to head of tide on the Saco River estuary – Class SC.

1967, c. 154, § 1.

2. **Eliot.**

A. Tidewaters within the Town of Eliot – Class SB-1.

3. **Kennebunk.**

A. Tidal waters of Kennebunk not otherwise classified or described – Class SB-2.

B. Estuary of Mousam River from head of tide to Route 9 bridge crossing and tidal tributaries thereof – Class SC.

C. Kennebunk River estuary from head of tide to the Route 9 crossing and tidal tributaries thereof – Class SC.

4. **Kennebunkport.**

A. Tidewaters of Kennebunkport not otherwise classified or described – Class SB-2.

B. Kennebunk River estuary from head of tide to the Route 9 crossing and tidal tributaries thereof – Class SC.

C. Tidewater from a point where longitude 70°-27'-37" crosses the Kennebunkport mainland to a point where longitude 70°-26'-48" crosses the mainland of Kennebunkport – Class SA.

D. Tidewater from a point directly west of the most northerly point of Vaughn Island to a point directly west of the most northerly point of land on Redin Island – Class SC.

E. Estuaries of Smith Brook and Batson River north of latitude 43°-23'-22" and tidal tributaries thereof – Class SC.

F. Tidewater from the mainland of Kennebunkport at latitude 43°-23'-22" north to a point where longitude 70°-24'-33" crosses the mainland – Class SB-1.

G. Tidewater from a point where longitude 70°-24'-33" crosses the mainland north to a point where longitude 70°-24'-05" crosses the mainland – Class SA.

H. Estuary of Little River north of latitude 43°-24'-04" to head of tide, including tidal tributaries thereof – Class SC.

5. **Kittery.**

A. Tidewaters of Kittery not otherwise specified or classified – Class SB-1.

B. Tidewaters from Sister's Point to Kittery-York town boundary, with the exception of Brave Boat Harbor – Class SA.

6. **Old Orchard Beach.**

A. All tidewaters of Old Orchard Beach – Class SB-2.

7. **Saco.**

A. Tidewaters of Saco not otherwise described or classified – Class SB-2.

B. Saco River estuary from head of tide to the Camp Ellis breakwater – Class SC.

1967, c. 154, § 2.

8. **South Berwick.**

A. All tidewaters of South Berwick – Class SB-1.

9. **Wells.**

A. Tidewaters of Wells not otherwise described or classified – Class SB-2.

B. From Wells-York town line to a point of land at longitude 70°-35'-35" and latitude 43°-14'-44" – Class SC.

C. Tidewater from a point where latitude 43°-16'-15" crosses Moody Beach to a point where latitude 43°-19'-04" crosses Wells Beach – Class SA.

D. Estuary of Webhannet River from head of tide to a point at longitude 70°-34'-32", latitude 43°-17'-48" – Class SC.

E. Estuary of Webhannet River from the most easterly bridge crossing at latitude 43°-18'-15" to its ocean confluence at latitude 43°-19'-14" – Class SB-1.

F. Estuary of Merriland River and tidal tributaries thereof from head of tide to its ocean confluence at latitude 43°-20'-10" – Class SC.

10. **York.**

A. Tidewaters of York not otherwise described or classified – Class SB-2.

B. Tidewaters from Kittery-York town line to point of land known as Argo Pt. – Class SA.

C. Tidal estuary of York River from Route 1 crossing to head of tide, including tidal tributaries thereof – Class SB-2.

1970, c. 538, § 2.

D. Tidewaters from East Pt. to the northernmost point of land at Concordville at longitude 70°-36'-11" – Class SB-1.

E. Estuary of Cape Neddick River from head of tide to point where longitude 70°-36'-46" crosses – Class SB-2.

1970, c. 538, § 2.

F. Tidewaters from Weare Pt. to a point where longitude 70°-36'-46" crosses the mainland of York – Class SB-1.

G. Tidewater from a point where longitude 70°-35' crosses the mainland of York to York-Wells town line – Class SA.

R.S. 1954, c. 79, §15; 1957, c. 322, §9; 1959, c. 183, §3; 1961, cc. 273, 284; 1963, c. 54, §2; c. 157; c. 274, §3; cc. 316, 320; 1979, c. 495, §§ 9, 10.

§371. Repealed.

1955, c. 426, §8; 1957, c. 322, §§2, 10; 1963, c. 420, §3; 1967, c. 342, §1; 1971, c. 335; 1973, c. 29; 1977, c. 373, §30.

§371-A. Classification of great ponds

1. **Great ponds classified.** All great ponds within the State of Maine shall be classified as not less than Class GP-A, except as otherwise provided in this section. The board, upon application by any interested person, may hold a hearing in accordance with the classification procedure and if it shall find it is for the best interests of the public that such waters or any part thereof would be otherwise classified, it shall do so in accordance with the classification procedure of this subchapter.

2. **Existing discharges.** Existing licensed discharges to Class GP-A great ponds will be allowed to continue until practical alternatives exist, but no new discharges will be permitted to Class GP-A great ponds after the effective date of this section.

3. **Exemption.** Aquatic chemical applications approved by the Board of Environmental Protection shall be exempt from the "no discharge" provision.

1979, c. 281 & 495.

4. **Class GP-B.** The following great ponds shall be classified Class GP-B:

A. Annabessacook Lake, Monmouth and Winthrop Townships, Kennebec County;

B. Repealed. 1979, c. 281 & 495.

C. Cobbosseecontee Lake, Winthrop, Monmouth, West Gardiner and Litchfield Townships, Kennebec County;

D. Douglas Pond, Pittsfield Township, Somerset County;

E. Estes Lake, Sanford and Alfred Townships, Franklin County;

F. Repealed. 1979, c. 281 & 495.

G. Little Cobbosseecontee Lake, Winthrop Township, Kennebec County;

H. Lovejoy Pond, Albion Township, Kennebec County;

H-1. Monson Pond, Fort Fairfield and Easton Townships, Aroostook County;

H-1 added 1979, c. 281 & 495.

I. Nubble Pond, Raymond Township, Cumberland County;

J. Pattee Pond, Winslow Township, Kennebec County;

K. Pleasant Pond, Litchfield and Gardiner Townships, Kennebec County;

L. Repealed. 1979, c. 281 & 495.

M. Sabattus Pond, Sabattus, Green and Wales Townships, Androscoggin County;

N. Salmon Lake, Belgrade and Oakland Townships, Kennebec County;

O. Sebasticook Lake, Newport Township, Penobscot County;

P. Spaulding Pond, Lebanon Township, York County;

P-1. Togus Pond, Augusta Township, Kennebec County; and

Q. Webber Pond, Vassalboro Township, Kennebec County.

P-1 added 1979, c. 281 & 495.

1977, c. 373, §31; 1979, c. 281, §2; 1979, c. 495, §§11 to 15.

DEPARTMENT OF ENVIRONMENTAL PROTECTION
BUREAU OF WATER QUALITY CONTROL
REGULATIONS

These regulations are current as of the date printed on the cover of this booklet. There may have been changes after this booklet was printed. The reader is urged to contact the Bureau of Water Quality Control 207-289-2591 or the Citizens' Environmental Assistance Service 1-800-452-1942 if there are any questions.

581-1 – 581.7 WATER QUALITY EVALUATIONS
EFFECTIVE DATE NOVEMBER 29, 1973; AMENDED DATE MARCH 14, 1977

581.1 Assimilative Capacity–Rivers and Streams

For the purpose of computing whether a discharge will violate the classification of any river or stream, the assimilative capacity of such river or stream shall be computed using the minimum seven day low flow which occurs once in ten years. Waste discharges shall be appropriately reduced when flows fall below the seven day ten year low flow if the board determines that such reduction is necessary to maintain such applicable classification.

581.2 Minimum Flow–Regulated Rivers and Streams

For regulated rivers and streams, the Department may establish a minimum flow necessary to maintain water quality standards. This flow will be based upon achieving the assigned classification, criteria and protection of the uses of the stream. The Department will cooperate with appropriate Federal, State and private interests in the development and maintenance of stream flow requirements.

581.3 Assimilative Capacity–Great Ponds

The hydraulic residence time will be used to compute the assimilative capacity of great ponds. Hydraulic residence time will be computed by dividing lake volume by the product of watershed area and the precipitation runoff coefficient.

581.4 Reserved

581.5 Zone of Passage

All discharges of pollutants shall, at a minimum, provide for a zone of passage for free-swimming and drifting organisms. Such zone of passage shall not be less than ¾ of the cross-sectional area at any point in the receiving body of water. Such zone of passage may be reduced whenever the applicant for a discharge can demonstrate that (a) because of physical phenomena in the receiving body of water such minimum zone cannot be maintained and (b) such minimum zone of passage is not necessary to protect organisms in the receiving body of water from substantial adverse effects.

581.6 Great Ponds Trophic State Index (TSI)

For the purposes of determining trophic state of great ponds the following trophic state index will be used.

Zero on this scale indicates poor water quality and 100 indicates excellent water quality. The TSI is defined as 40 + 33 ($\log_{10}$ minimum Secchi disk transparency in meters). Average chlorophyll *a* and spring total phosphorus may also be related to TSI.

581.7 Stream Species Diversity Index

The generic diversity of the bottom fauna of waters classified B-1 and B-2 shall not be less than 2.2 as measured by the Shannon-Weiner diversity index.

582.1 – 582.8 TEMPERATURE
EFFECTIVE DATE NOVEMBER 29, 1973

582.1 Freshwater Thermal Discharges

No discharge of pollutants shall cause the ambient temperature of any freshwater body, as measured outside a mixing zone, to be raised more than 5°F or more than 3°F in the epilimnion of any lake or pond. In no event shall any discharge cause the temperature of any freshwater body to exceed 84°F at any point outside a mixing zone established by the board, nor shall such discharge cause the temperature of any waters which presently are designed as trout or salmon waters to exceed 68°F at any point outside a mixing zone established by the board.

582.2-4 Reserved

582.5 Tidal Water Thermal Discharges

No discharge of pollutants shall cause the monthly mean of the daily maximum ambient temperatures in any tidal body of water, as measured outside the mixing zone, to be raised more than 4°F nor more than 1.5°F from June 1 to September 1. In no event shall any discharge cause the temperature of any tidal waters to exceed 85°F at any point outside a mixing zone established by the board.

582.6-8 Reserved

583.1 – 583.3 NUTRIENT CONCENTRATION
EFFECTIVE DATE NOVEMBER 29, 1973; AMENDED DATE MARCH 14, 1977

583.1 Phosphorus

There shall be no additional discharge of phosphorus to any lake or pond or tributary thereto which discharge does not employ the best available technology for phosphorus removal.

583.2

Existing discharges of phosphorus to any lake, pond or tributary thereto shall, on or before October 1, 1976, be treated to remove phosphorus to the maximum extent technically feasible.

583.3 Phosphorus Concentrations in Tributaries to Great Ponds

Notwithstanding Sections 583.1 and 583.2, the ambient concentration for total phosphorus in all tributaries to Great Ponds shall not exceed 50 micrograms per liter (50 ug/l).

584 – 584.1 WATER QUALITY CRITERIA
EFFECTIVE DATE MARCH 14, 1977

584 Water Quality Criteria

The criteria listed below will apply only to Seciton 363, standards of classification of fresh waters, class B-1 and B-2. The numbers represent maximum acceptable concentration limits in the receiving waters. All numbers are expressed in micrograms per liter (ug/l).

584.1 Metals

	Metal	*Concentration*
A.	Chromium	50 ug/l

590.1 – VARIANCES FROM VARIOUS REGULATIONS
EFFECTIVE DATE NOVEMBER 29, 1973; AMENDED DATE MARCH 14, 1977

590.1 Variances

The board may, in any license or Order issued by it, impose on any discharge limitations more stringent than those required by Regulations 580, 581, 582, 583 and 584 whenever the physical or chemical properties or biological phenomena in the receiving body of water so require in order to maintain the statutory classification. The board may authorize a variance from any of the limits established hereby whenever the applicant demonstrates that (a) because of physical or natural conditions in the receiving body of water such limits cannot be attained and (b) maintenance of such limits are not necessary to protect organisms in the receiving water from substantial adverse effects and (c) the proposed discharge will assure the protection and propagation of a balanced and indigenous population of fish, shellfish and wildlife in and on the receiving body of water.

591.1 – EXCEPTIONS FOR VARIOUS REGULATIONS
EFFECTIVE DATE NOVEMBER 29, 1973; AMENDED DATE MARCH 14, 1977

591.1

No provision of Regulations 580, 581, 582, 583, 584 and 590 shall be deemed to change, alter, affect or supersede the terms or conditions of any Order or license heretofore issued by the Board of Environmental Protection.

DVWK
German Association for Water Resources and
Land Improvement
Gluckstraße 2 · D-5300 Bonn 1 · Phone: 0228/631446

STRUCTURE
TASKS
ACTIVITIES

Table of Contents:

(Bonn, January 1984)

The German Association for Water Resources and Land Improvement (DVWK) is a technical-scientific organization which promotes water resources development and agricultural engineering under consideration of all environmental aspects.

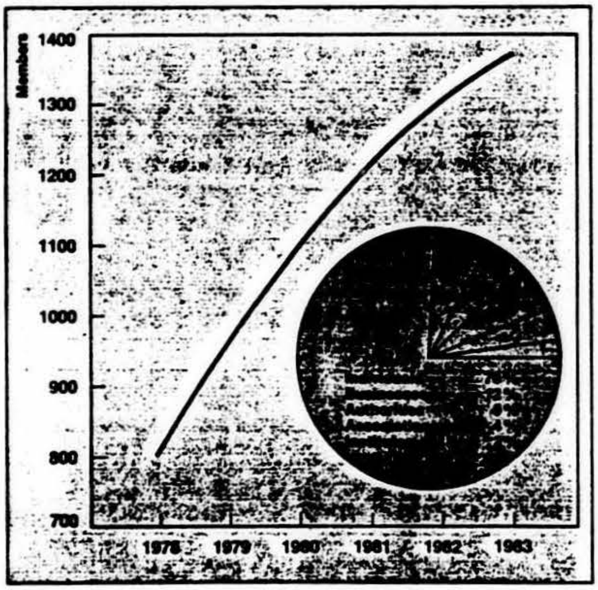
Membership

- is open to anyone who is prepared to support the DVWK objectives

Members

- are federal, state and local authorities of the Federal Republic of Germany, as well as businesses and organizations which apply the results of the Association's activities.

Membership since 1978



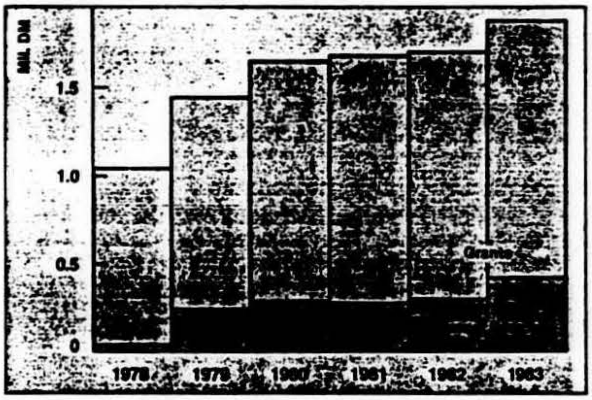
DVWK Members

- support the overall technical effort by voluntary participation in the committees and working groups
- regularly receive the DVWK-Nachrichten (Newsletter) and various information
- receive in some instances extensive rebates for advanced training programs and conferences sponsored by the Association
- pay a lower subscription rate for the Association's periodicals WASSERWIRTSCHAFT (Water Resources) and WASSER UND BODEN (Water and Soil)
- establish valuable contacts with colleagues and technical institutions

Finances

The Association's work is supported and financed to a great extent by the Federal Government and the State Authorities. Their grants enable the transfer of working results to the professional public.
Further funds are derived from membership fees, general income and special projects.

Association's Finances



Tasks + Objectives

Interdisciplinary exchange of thoughts and experiences enables the integration of experts from universities, authorities, engineering firms and industry. The DVWK considers itself to be a mediator between theory and practice, research and application. Its community of experts is primarily active in the following areas:

State of the Art

The DVWK evaluates the latest research results, combines them with the most recent practical experiences and provides application oriented suggestions.

Recommendations

The Association publishes the "DVWK-Regelwerk" (Technical Rules). Its "Regeln und Merkblätter zur Wasserwirtschaft" (Standards and Guidelines for Water Management), which contain recommended methods and procedures, are developed with the aid of the experts inside and outside the Association.

Standardization

The DVWK participates in the developement of standards by sending experts to the "Water Practice Standards Committee" (NAW) of the German Institute for Standardization (DIN). This involves hydrology, hydraulics, agricultural engineering and landscaping.

Collaboration

The DVWK sets its experts' competence at the disposal of other technical-scientific working groups and expects the same of these organizations.

International Activities

The DVWK participates in the symposia and working groups of international organizations; seeks consultation and technical contributions; gives organizational and financial support to German participants; spreads information on international technical activities and their results.

Research

The DVWK initiates research projects resulting from its working groups' activities and establishes contacts with research foundations and institutions. It has the right to recommend reviewers for projects sponsored by the German Research Society (DFG).

Advanced Education

The DVWK organizes application oriented seminars and short courses. It also offers a four semester advanced study in Hydrology and Water Resources Management.

Meetings

The DVWK supports and organizes technical meetings, symposia and workshops. Its biennial experts' meeting and its irrigation symposium are particularly renown.

Publications

The DVWK has six publication series which reflect the results of its working groups. The Association's activities are reported bimonthly in the "DVWK-Nachrichten" (Newsletter). The Association's periodicals are WASSER UND BODEN (Water and Soil) and WASSERWIRTSCHAFT (Water Resources).

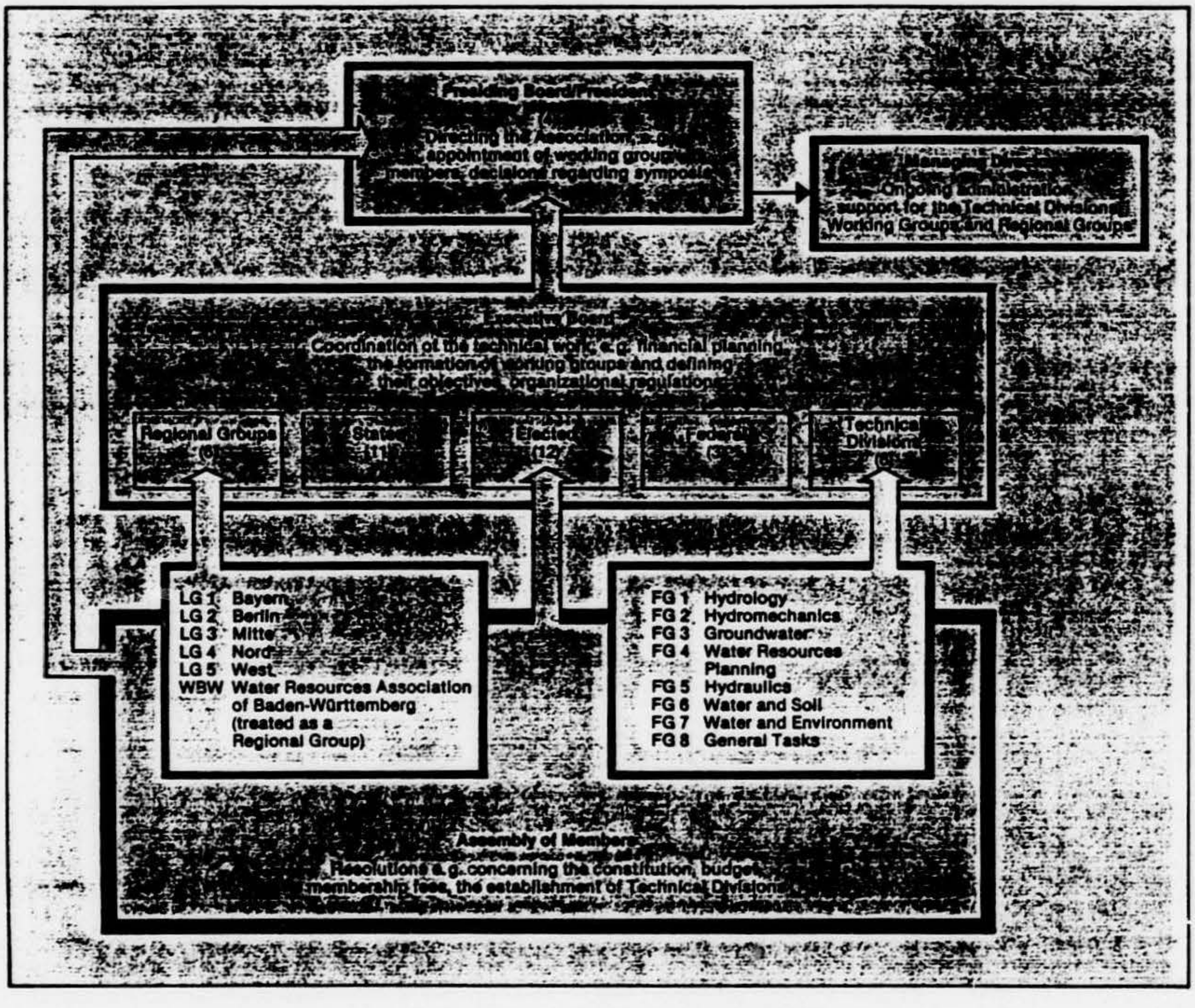
Public Relations

The DVWK presents a podium for public discussion of actual water resources problems, organizes press conferences and publishes press releases.

Consulting

The DVWK consults for authorities, members and professional colleagues, seeks qualified experts in various disciplines and makes relevant informative material available.

Organization of the Association



Technical Activities

Working Groups

The DVWK has established working groups (FA) in which appointed experts from administrative agencies, universities, associations and industry work together to achieve the goals set by the Executive Board. Each working group is composed of up to eight voluntary members who meet two to three times yearly at different locations for one or multiple day meetings. Here they discuss their drafts of technical-scientific opinions prepared during their free time and produce a final report following appropriate consultation and extensive evaluation.

Technical Divisions

The working groups are each categorized into one of eight technical divisions (FG) representing a basic overall discipline. These promote the exchange of ideas and collaboration in a particular technical field. Each DVWK member can declare in writing to the respective technical division chairman his membership to one or more technical divisions. The responsibilities of the Technical Division Board include both the coordination of working groups and the planning and undertaking of symposia. This board consists of the technical division chairman and the leaders of the associated working groups.

Technical Division 1 Hydrology	Technical Division 2 Hydromechanics	Technical Division 3 Groundwater	Technical Division 4 Water Resources Planning	Technical Division 5 Hydraulics	Technical Division 6 Water and Soil	Technical Division 7 Water and Environment	Technical Division 8 General Tasks
FA 1.1 Precipitation FA 1.2 Snow Hydrology FA 1.3 Low Discharge FA 1.5 Runoff Models FA 1.6 Water Level and Discharge Forecast FA 1.8 Forests and Water FA 1.9 Anthropogenetic Influcences on Surface Discharges FA 1.10 Water Resources Investigations in Semi-Arid Regions	FA 2.1 Pipe and Channel Hydraulics FA 2.2 Hydraulic Modelling and Measurement FA 2.3 Dilution Processes FA 2.4 Groundwater Transport Processes FA 2.6 Sediment Transport	FA 3.1 Locating Groundwater FA 3.2 Groundwater Use FA 3.3 Groundwater Hydraulics and Models FA 3.4 Groundwater Biology FA 3.5 Groundwater Chemistry FA 3.6 Groundwater Measurement	FA 4.6 Planning of Regional Water Resource Systems FA 4.7 Operation Research for Water Resource Systems FA 4.8 Water Resources Economics	FA 5.1 River Dikes FA 5.2 Reservoirs FA 5.3 Weirs	FA 6.1 Site and Soil FA 6.2 Drainage FA 6.3 Soil Use and Nutrient Washout FA 6.4 Irrigation FA 6.5 Soil Erosion FA 6.6 Rural Roads	FA 7.1 River Training FA 7.2 Use and Conservation of Agricultural Lands FA 7.3 Lakes and Earth Embankments FA 7.4 Contaminant Loads FA 7.5 Obtaining and Evaluating Water Quality Data FA 7.6 Influence of Man on River Quality	FA 8.1 Law and Taxes FA 8.2 Advanced Education and Training FA 8.3 Publications and Public Relations FA 8.4 International Collaboration FA 8.5 Data Processing in Water Resources

Technical Activities

Division HYDROLOGY

The understanding of the interactive processes within the water cycle is a prerequisite for an effective planning and implementation of water management measures. Hydrological investigations are, therefore, integrally coupled with water management. The Hydrology Division is primarily concerned with precipitation -- runoff phenomena; parameters and boundary conditions which control these processes; short and long term water levels and discharge forecasts; and the consequences of anthropogenetic measures.

Tasks:

- Procedures for analysing precipitation data
- Determination of rainfall intensity distribution
- Determination of runoff from snow covered catchment areas
- Statistical techniques in the analysis of low discharges
- Procedures for applying precipitation-runoff models to small catchment areas
- Transport of substances into forests by rainfall
- The effects of human intervention on the runoff situation
- Determination of hydrological design data in arid regions

Division HYDROMECHANICS

Hydromechanics provides a multi-sided tool for hydraulic, water resources and agricultural engineering. The Hydromechanics Division deals with the application of theoretical models and experimental data, the use of hydraulic and numerical models and practical implementation of measurement techniques. The basic topics include the improvement and standardization of design criteria for hydraulic structures; the theory and simulation of transport processes; and the role of hydraulic modelling in engineering applications. Questions of water quality receive rising importance.

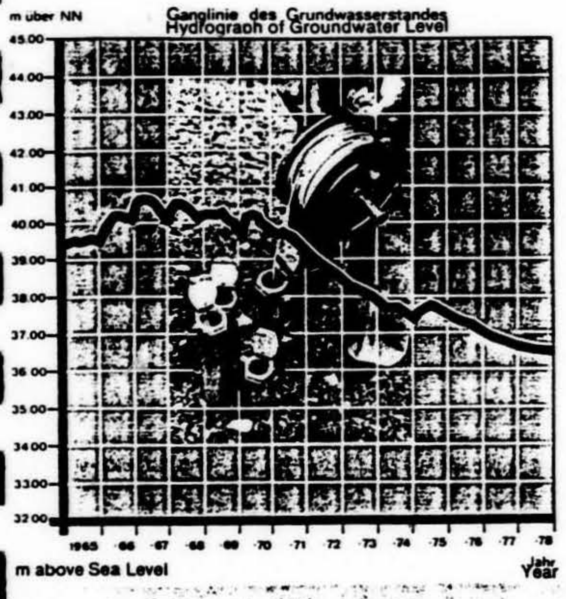
Tasks:

- Numerical hydrodynamic models
- Application and limits of hydraulic models
- Roughness coefficients for natural and artificial channels
- Transport of heats and substances in waters

Sedimentation in Reservoirs

Technical Activities

Division GROUNDWATER



Groundwater quality is influenced by physical, chemical and (micro) biological processes. The Groundwater Division is particularly concerned with the anthropogenetic threat to groundwater quality and the necessary criteria for reasonable groundwater management. Of primary importance is the development of concepts for improving groundwater quality and guidelines for in part conflicting riparian rights and restoration.

Tasks:

- Groundwater replenishment and availability
- Reconnaissance of deep lying groundwater tables
- Hydrochemical groundwater classification
- Improvement of groundwater management
- Transport and dilution of contaminants
- Infiltration of contaminants into groundwater
- Evaluation and removal of anthropogenetic groundwater pollution

Groundwater Measurement Station

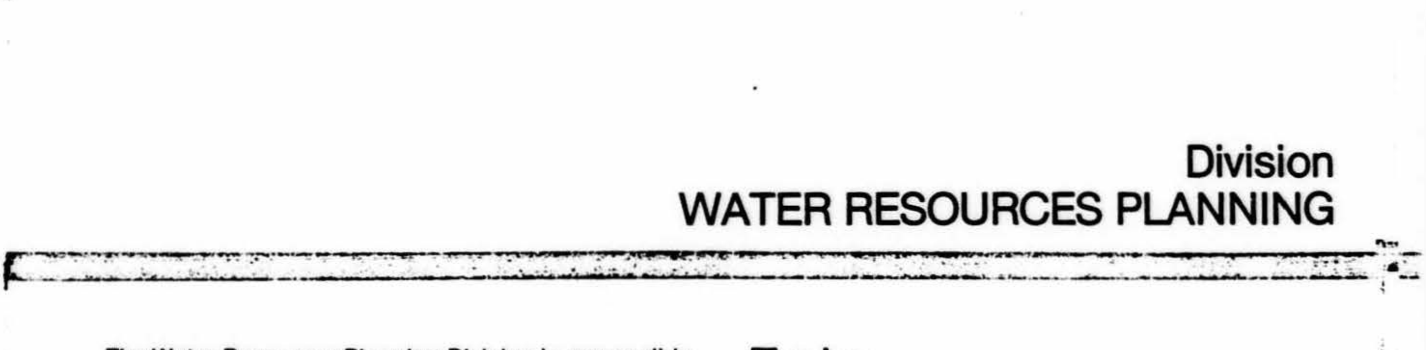
Division WATER RESOURCES PLANNING

The Water Resources Planning Division is responsible for revising the water resources planning and implementation concepts developed during the past two decades, in such a manner, as to extremely simplify their application. Regional planning, economics and cybernetics are essential elements of this effort.

Tasks:

- Project evaluation in water resources management
- Operation research and simulation techniques
- System's operation and regulation
- Structure of water resources regional planning

Water Resources Regional Planning for the Isar (map extract BayLfU)

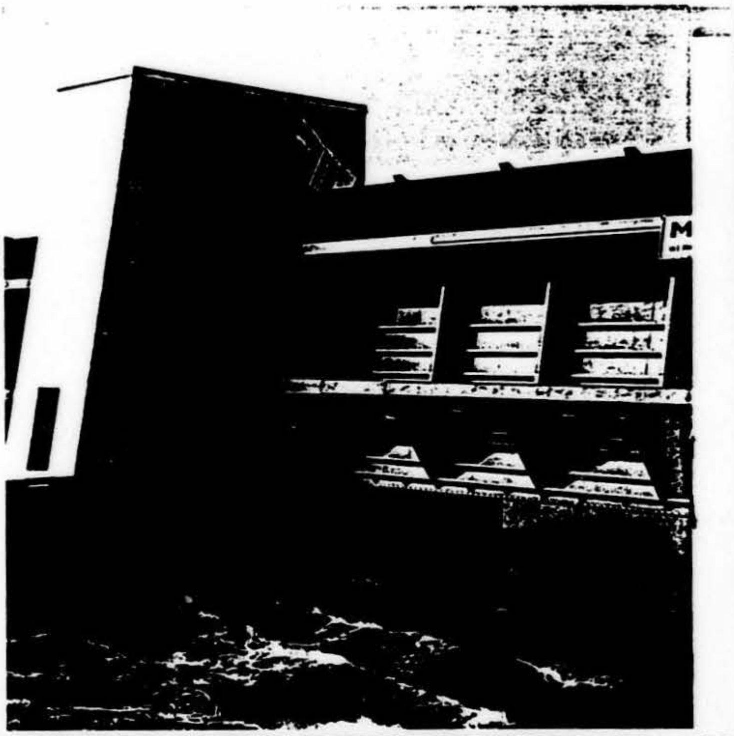


Division
HYDRAULICS

The Hydraulics Division treats questions related to the design and construction of hydraulic structures. This also includes questions of structure safety, considering as well the choice of appropriate measurement and control systems; the goal oriented implementation of relevant measures; analysis and evaluation techniques.

Tasks:

- Planning, construction and inspection of river dikes
- Documentation of reservoirs in the Federal Republic of Germany
- Measurement equipment and control structures for dams
- Design of intake and discharge structures
- Safety risks at weirs due to hydrodynamic phenomena
- Modernization of old weirs



Weir in Geisling

Division
WATER AND SOIL

Present land management measures cause long term and in part negative changes in soil quality and its water balance. Revision of both land and water resource management policies are necessary in order to restore land fertility and improve the groundwater budget. The Water and Soil Division must, therefore, not only seek ways to guarantee agricultural productivity but also address the problems of erosion and nutrient washout.

Tasks:

- Investigation on site to determine indicators for soil improvement measures
- Reduction of nutrient washout from farmland
- Techniques for effective use on irrigation water
- Recommendations for determining sprinkling rates
- Influence of heavy rains on soil erosion
- Guidelines for rural road construction

Mud Formation Following Heavy Rains

Division
WATER AND ENVIRONMENT

Rivers and Lakes are one of the most strained and endangered components of our environment. Man generates unsuitable contaminants, impurities and a disruption of the natural ecological cycie. The Waters and Environment Division seeks solutions for limiting the misuse of surface water, develops criteria for setting water quality goals for rivers and lakes; establishes appropriate techniques for collecting and evaluating water quality data; and defines ecological constraints to be considered for the development and maintenance of flowing and stagnant surface water.

Tasks:

- Impact of land use on the water balance
- Mapping surface water pollution
- Statistical analysis of water quality data
- River training under ecological aspects
- Improvement and restoration of lakes
- Decision aids related to data acquisition for water quality control

Ecologically Trained River

Division GENERAL TASKS

The General Tasks Division is responsible for overall organization and coordination. The most important areas are consultation in legal matters; planning of advanced professional training; the preparation of publications; and activities related to international cooperation. The Working Group for Data Processing in Water Resources, established in 1983, is also part of this division. Its primary goal is the transfer of experience and information relating to questions of electronic data processing. It provides interdisciplinary support in this field for the DVWK Working Groups and Federal Government and State Institutions.

The Association offers a wide range of possibilities for advanced education and training courses.

Seminars (1—2 days)

A limited number of participants are presented lectures and examples of practical problem solutions and are exposed to modern water resource and land management techniques. Over 100 seminar topics have been offered as a response to requests from ministries, water resource agencies, DVWK regional groups and other associations.

Short Courses (3—5 days)

Short courses on the subjects technical hydraulics, hydrology, groundwater, river training and irrigation are offered in cooperation with universities or federal and state authorities. They do not only afford an opportunity for the transfer of knowledge in a particular topic, but also serve as a podium for the exchange of experiences and information with experts familiar with the state of the art.

Advanced Studies

Established on October 1, 1982, the 4-semester correspondence course Advanced Studies in Hydrology and Water Resources is oriented towards professionals who are put into the position to evaluate and solve applied problems using the latest scientific information.

Publications

The **DVWK-Regeln zur Wasserwirtschaft** (Water Management Standards) include technical procedures and measures which have been found to be of practical use and are considered by experts to be indisputable technical solutions. The DVWK recommends their application as generally accepted technical standards.

Procedures and construction techniques as well as other technical aspects considered to be the state of the art are treated in the **DVWK-Merkblättern zur Wasserwirtschaft** (Water Management Guidelines). Information found in DVWK guidelines can subsequently be issued as DVWK Regeln, if the guidelines have been proven in practice and been accepted by renown experts in the field.

The Working Groups' summary reports, background information for DVWK standards and guidelines, as well as presented papers are published in the **DVWK-Schriften** (Journals).

The **DVWK-Bulletin** is a foreign language series in which the work of mainly German authors and the results of international conferences involving the DVWK are reported in English or French. Some editions are published in cooperation with international water resources and land management organizations.

The annual report, membership lists and conference calendars are issued in the **DVWK-Mitteilungen** (Standard Publications). Additionally, the working groups have a forum for reporting workshop or colloquia results and publishing working or discussion papers which were prepared in a limited edition for a small professional group.

Seminar notes and, when appropriate, lecture manuscripts are published in the **DVWK-Fortbildung** (Advanced Education) series. Advanced study material, as well as a course guide and correspondence notes for the study "Hydrology and Water Resources" are also issued in this series.

The bimonthly **DVWK-Nachrichten** (Newsletter) is sent to members free of charge. Activities of the technical divisions and regional groups, as well as information on conferences, courses, and the Association's publications are reported here.

Regional Groups

The DVWK members have formed regional groups (LG) in the various states of the Federal Republic of Germany with the exception of Baden-Württemberg. There, the "Water Resources Association of Baden-Württemberg" is acting on behalf of the DVWK for the DVWK members.

The Regional Groups' responsibilities include:

- Advising and informing members
- Conducting conferences, assisting in the exchange of information with practicing professionals
- Visits and excursions, including foreign study trips
- Aid in solving regional water resource and land management problems

Each regional group usually conducts its membership meetings biennially. The regional groups of LG Bayern and LG Mitte publish their own membership newsletters.

Regional Groups (Member States)

Collaboration

The DVWK collaborates with numerous water resources associations and related societies on both the national and international levels. The German National Committee of the International Commission on Irrigation and Drainage is incorporated organizationally within the DVWK; its secretariat is located in the Association's headquarters.
Some of the organizations with which collaboration occurs are:

National

ATV	—	German Association on Water Pollution Control
DGEG	—	German National Society for Soil Mechanics and Foundation Engineering
DLG	—	German Agricultural Society
DVGW	—	German Gas and Water Association
FHDGG	—	Hydrology Division of the German Geological Society
MEG	—	Max-Eyth-Society
VDEW	—	Association of German Electric Utilities
VDI	—	Association of German Engineers

International

ÖWWV	—	Austrian Water Resources Association
SWV	—	Swiss Association for Water Economy
CIGR	—	International Commission of Agricultural Engineering
IAH	—	International Association of Hydrogeologists
IAHR	—	International Association for Hydraulic Research
IAHS	—	International Association of Hydrological Sciences
ICID	—	International Commission on Irrigation and Drainage
ICOLD	—	International Commission on Large Dams
IWRA	—	International Water Resources Association
UNESCO	—	United Nations Educational, Scientific and Cultural Organization

Chronology

DVWW German Water Resources Association		KWK Water and Land Management Board
"Hagener Meeting". Draft of a Prussian Water Regulation	1891	
"Water Resources Association" in Prussia	1908	
Merger with "German Water Power Association" to "German Water Resources and Power Association"	1920	
	1921	"Land Management Committee" formed as a working group coordinated by the German Industrial Standards Committee (presently DIN — German Institute for Standardization)
	1924	"German Land Management Committee" (DAfK). Continuation of the above function
Renamed as the "German Empire Water Resources Association"	1933	
	1939	With 13 subcommittees
Disbandment	1945	Disbandment
	1946	Establishment of the "Land Management Committee" (AfK) in East Berlin. Continuation of the DAfK responsibilities
Starting in 1948 — Formation of regional associations	1948	
	1949	Establishment of the "West German Land Management Committee" (AfKW) in Bonn. Formation of 8 subcommittees
	1952	Renamed as the "Land Management Board" (KfK)
"German Water Resources Association" (DVWW) as umbrella organization for the state associations	1960	
	1972	Starting 1972: Definition of KfK tasks and responsibilities to those of "German Association on Water Pollution Control" (ATV) and the "German Gas and Water Society" (DVGW). Orientation towards water resources management
Open to individual members. Reorganization of the regional associations into regional groups. The establishment of advanced education courses in hydrology and river training. Collaboration with international organizations	1973	
	1974	"Water and Land Management Board" (KWK). Some of the activities include advanced training seminars, technical publications, joint standards with DVWW. Collaboration with the "International Commission on Irrigation and Drainage" (ICID) and other international organizations

DVWK

1978
Merger of the DVWW and KWK societies into the "German Association for Water Resources and Land Improvement (DVWK)

1979
Disbandment of the "German Irrigation Management Society" (DGBW). Assumption of its responsibilities by DVWK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Surges from ice jam releases: a case study

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Surges from ice jam releases: a case study

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Accounts by witnesses of spring ice breakup in rivers often mention violent ice runs with extreme water speeds and rapidly rising water levels. Such events are believed to follow the release of major ice jams. To gain preliminary understanding of this problem, an attempt is made to reconstruct a partially documented ice jam release reported recently by others. The equations of the ice–water flow that occurs after the release of an ice jam are formulated. It is shown that the problem may be approximately treated as a one-dimensional, unsteady, water-only flow of total depth identical to that of the ice–water flow, and average velocity. The retarding effect of the frequently encountered intact ice cover below the jam is considered implicitly, that is, by adjusting the friction factor so as to make the predicted and observed downstream stages equal. The effects of jam length are considered next by assuming longer jams of the same maximum water depth. The duration of the surging velocities increases with jam length and so does the peak stage. Less than 2 h after the jam release the surge was arrested and a new jam formed, causing further stage increases. Present capabilities of modelling the reformation process are discussed and the major unknowns identified.

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Introduction

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

There are several practical problems that are related to surges from ice jam releases, such as short and long term forecasting of peak water levels near a populated area located downstream of a major jamming site; possible bed scour and bank erosion due to relatively brief but intense ice runs; and peak stages during reformation of a released jam. Such dynamic aspects of ice jamming are poorly understood at present, especially in quantitative terms. The writers are aware of only two pertinent investigations: an application of an open-water unsteady flow model to assess surge effects on bed scour (Mercer and Cooper 1977) and a theoretical investigation of surging and new jamming (Henderson and Gerard 1981).

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

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

Unsteady ice–water flow

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

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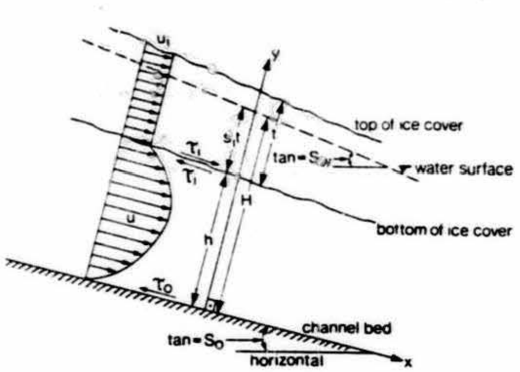

FIG. 1. Definition sketch.

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

Continuity equations

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

$$[1] \quad (1 - \epsilon)\frac{\partial t}{\partial T} + \frac{\partial q_i}{\partial x} = 0$$

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

$$[2] \quad q_i = (1 - \epsilon)u_i t$$

Substituting [2] in [1] gives:

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

Consideration of the mass conservation of water gives:

$$[4] \quad \frac{\partial h}{\partial T} + \epsilon s_i \frac{\partial t}{\partial T} + \frac{\partial q_w}{\partial x} = 0$$

in which q_w = water discharge, given by

$$[5] \quad q_w = q' + \epsilon u_i s_i t \; .$$

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

$$[6] \quad q' = \int_0^h u \, dy = Vh$$

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

$$[7] \quad \frac{\partial h}{\partial T} + \frac{\partial q'}{\partial x} = 0$$

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

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

$$[8] \quad \rho_w \frac{\partial H}{\partial T} + \frac{\partial \rho_w q}{\partial x} = 0$$

in which H = overall water depth, given by:

$$[9] \quad H = h + s_i t$$

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

$$[10] \quad \rho_w q = \rho_i q_i + \rho_w q_w$$

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

Momentum equations

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

$$[11] \quad \rho_w\left(\frac{\partial u}{\partial T} + u\frac{\partial u}{\partial x} + \upsilon\frac{\partial u}{\partial y}\right) = \rho_w g S_0 - \frac{\partial p}{\partial x} + \frac{\partial \tau}{\partial y}$$

in which u, υ = velocity components in the x and y directions respectively; g = magnitude of the acceleration due to gravity = 9.8 m/s^2; S_0 = channel bed slope; p = pressure, assumed approximately equal to the hydrostatic pressure; and τ = shear stress parallel to the x-axis, acting on a plane normal to the y-axis. The differential equation of continuity reads:

$$[12] \quad \frac{\partial u}{\partial x} + \frac{\partial \upsilon}{\partial y} = 0$$

By virtue of [12], the bracketed term on the left-hand side of [11] may also be written as $(\partial u/\partial T) + (\partial u^2/\partial x) + (\partial u\upsilon/\partial y)$. Making this substitution and integrating both sides of [11] from $y = 0$ to $y = h$, gives:

$$[13] \quad \rho_w\left\{\frac{\partial}{\partial T}\int_0^h u \, dy + (u)_h\frac{\partial h}{\partial T} + \frac{\partial}{\partial x}\int_0^h u^2 \, dy - (u^2)_h\frac{\partial h}{\partial x} + (u)_h(\upsilon)_h\right\} = \rho_w g S_0 h - \frac{\partial}{\partial x}\int_0^h p \, dy + (p)_h\frac{\partial h}{\partial x} - (\tau_i + \tau_0)$$

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

in which τ_0 = bed shear stress and τ_i = shear stress on the bottom of the cover, considered positive if it tends to retard the water layer and accelerate the cover, as sketched in Fig. 1. It is noted that $(u)_h = u_i$ and $p = \rho_w g(H - y)$. To determine $(v)_h$, [12] may be integrated from $y = 0$ to $y = h$; this gives:

$$[14] \quad (v)_h = u_i \frac{\partial h}{\partial x} - \frac{\partial}{\partial x} \int_0^h u \, dy$$

Using [6], [7], [9], and [14], [13] may be simplified to:

$$[15] \quad \rho_w \left(\frac{\partial q'}{\partial T} + \frac{\partial m'}{\partial x} \right) = \rho_w g h S_w - (\tau_i + \tau_0)$$

in which S_w = slope of the water surface and

$$[16] \quad m' = \int_0^h u^2 \, dy$$

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

$$[17] \quad (dm_i)a_i = g(dm_i)S_w + \tau_i \, dx$$

But

$$[18] \quad a_i = \frac{du_i}{dT} = \frac{\partial u_i}{\partial T} + u_i \frac{\partial u_i}{\partial x}$$

and

$$[19] \quad dm_i = \rho_i(1 - \epsilon) t \, dx + \rho_w \epsilon s_i t \, dx = \rho_w s_i t \, dx$$

Substituting [18] and [19] in [17] and rearranging, gives:

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

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

$$[21] \quad \rho_w h \left(\frac{\partial V}{\partial T} + V \frac{\partial V}{\partial x} \right) = \rho_w g h S_w - (\tau_i + \tau_0)$$

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

$$[22] \quad \rho_w H \left(\frac{\partial V}{\partial T} + V \frac{\partial V}{\partial x} \right) = \rho_w g H S_w - \tau_0$$

which is the same as the momentum equation for flow

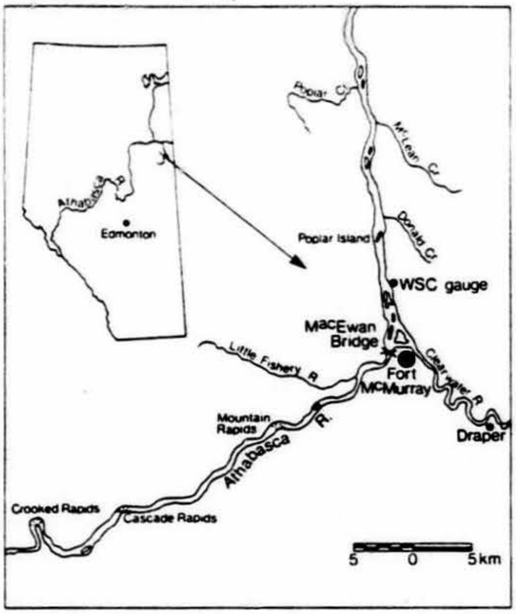

FIG. 2. Location map of study area (after Doyle and Andres (1979), with changes).

of depth H and average velocity V. Further, with $u_i = V$, it can be shown that q (defined by [10]) becomes equal to VH and [8] reduces to

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

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

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

Fort McMurray case study

Figure 2 is a plan of the Athabasca River in the

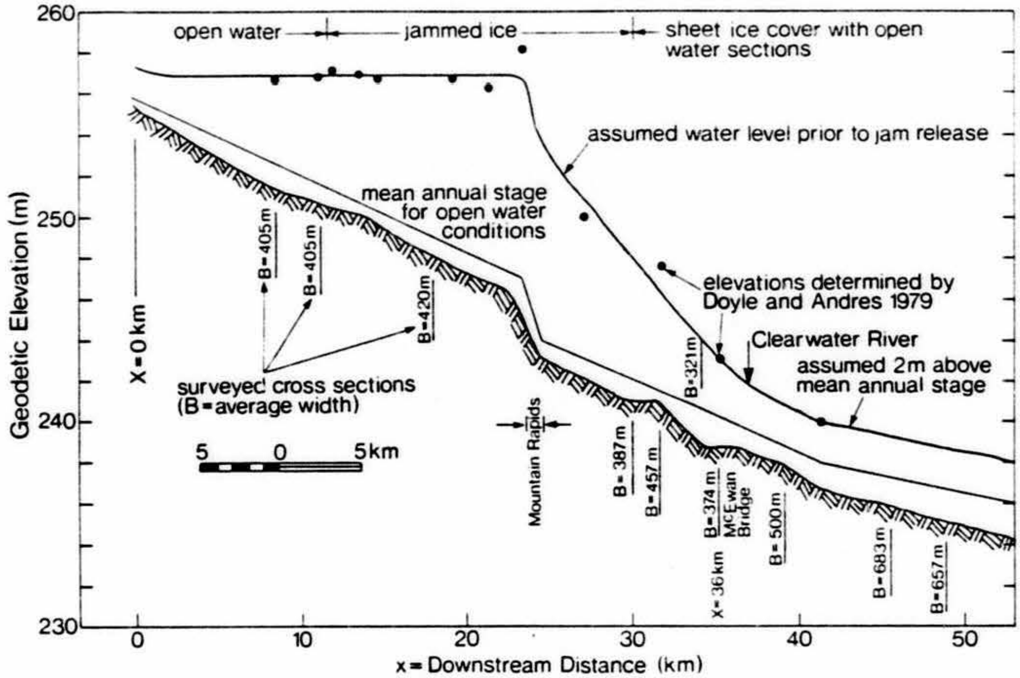

FIG. 3. Initial flow conditions and channel geometry (T = 0 is taken at 1950 h, April 28, 1979).

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

To solve the governing differential equations, a numerical algorithm was used that has been developed by Krishnappan and Snider (1977) for one-dimensional unsteady flow with variable channel width. Though this algorithm is capable of dealing with cross sections of arbitrary shape, it was deemed sufficient for the present purpose to assume rectangular sections, as follows. First, a cross section was approximated by a trapezoid of depth equal to the distance of the water surface from the average channel bed level. This trapezoid was then approximated by a rectangle of the same depth and of width equal to the average width of the trapezoid. The channel width and depth between successive surveyed cross sections were determined by linear interpolation.

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

In addition to the initial conditions, Krishnappan and Snider's algorithm requires the depth or flow rate at the upstream and downstream boundaries of the study reach plus an estimate of the friction factor or of the ratio V/V_* (V_* = shear velocity), which is assumed independent of x and T. Note that $V/V_* = C/\sqrt{g}$ with C being the Chezy resistance coefficient. The boundary conditions were specified simply by choosing the boundaries sufficiently far upstream of the jam and downstream of MacEwan Bridge to ensure that surge effects do not reach these locations during the computation period. The parameter V/V_* was left free, i.e., it was selected by trial and error so as to give optimum agreement between predicted and observed stages at McEwan Bridge. Though this parameter is known for open-water conditions ($V/V_* = 16$) and should probably apply to

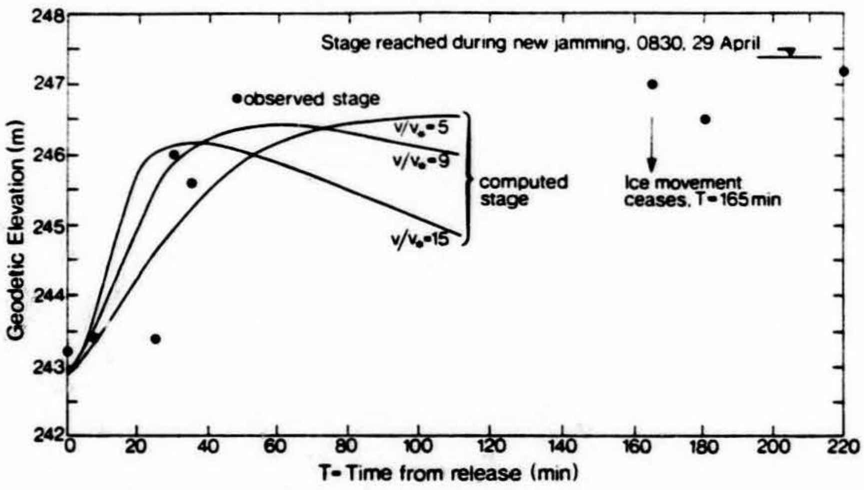

FIG. 4. Computed versus observed stage–time variations at MacEwan Bridge (T = 0 is taken at 1950 h, April 28, 1979; observed stages are approximate).

unimpeded ice–water flow, the same value may not be appropriate for the present case study; downstream of the ice jam, the river was not open but covered by 1-m thick sheet ice with occasional open-water sections. What the friction factor should be in this reach is unknown and certainly it would be expected to change with time and distance as the surge moves in and dislodges the sheet ice cover. Because this effect is too complex, it was considered reasonable to use an "average" constant value; clearly, this value should be less than the open-water value.

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

Figure 4 shows stage–time variations as computed for different values of V/V_* along with available observations. The best agreement between computation and observation seems to be obtained when $V/V_* = 9.0$. Note that all computed curves have a peak and decline slowly afterwards, as might have been expected since the simulated surge resembles, to a degree, the dam break problem. However, the observations show the stage to remain fairly steady after $T \simeq 50$ min. This is probably due to new jamming that occurred somewhere downstream of MacEwan Bridge. According to Doyle and Andres, ice movement at the bridge ceased at T = 165 min (2235 h, April 28) and a major jam was observed in the morning of April 29 with its toe 14 km below, and head 11 km above, MacEwan Bridge. Assuming that the new jam was initiated at the location indicated above,[2] it was estimated that, with $V/V_* = 9.0$, the time of initiation would have been $T \simeq 70$ min. For $T > 70$ min, effects of the new jamming would be experienced at MacEwan Bridge.

The downstream variation of the peak stage computed with $V/V_* = 9.0$ is shown in Fig. 5. For $x \geq 24$ km, the peak stage exceeds the initial stage, whereas for $x \leq 24$ km, the peak stage coincides with the initial stage. Clearly, in the latter reach the water level drops continuously from the very start of the surge.

Figure 6 shows the variation of V with time at MacEwan Bridge as computed for $V/V_* = 9.0$. At T = 35 min the computed value of V is 2.2 m/s while the surface velocity was estimated by site observers to be between 2 and 3 m/s. Considering that surface velocities are typically 15% larger than average values, the agreement between prediction and observation appears satisfactory. This finding provides a measure of confidence to the present approach since poor velocity estimates despite the matching of water stages would be a strong negative indication.

Considering again Fig. 3, it may be noticed that the stage profile of the ice jam does not include any section parallel to the normal river slope. This implies that the jam did not attain equilibrium in the sense adopted by Uzuner and Kennedy (1976) as a result of its limited length. Had the supply of ice been larger, an equilibrium section would have formed; this would have caused a longer jam than the one that actually formed.

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

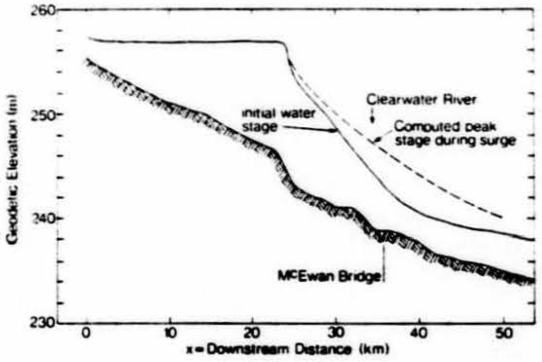

FIG. 5. Downstream variation of peak surge stage as computed with $V/V_* = 9$.

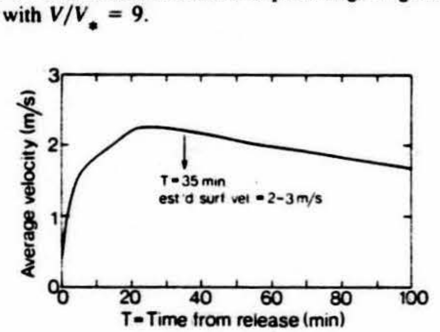

FIG. 6. Computed velocity–time variation at MacEwan Bridge ($V/V_* = 9$).

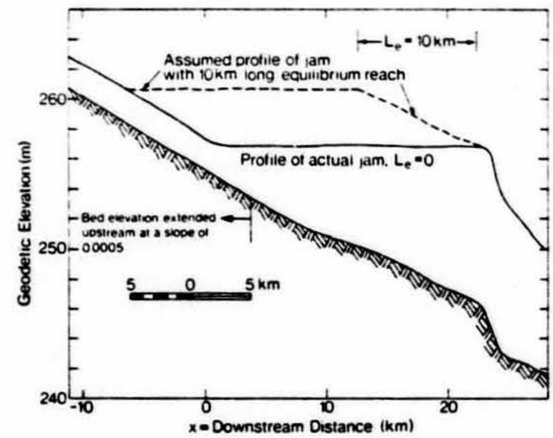

FIG. 7. Illustration of initial hypothetical jam with an equilibrium reach.

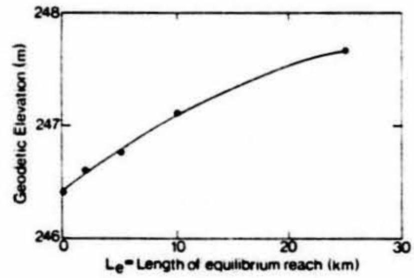

FIG. 8. Effect of jam length on peak surge stage at MacEwan Bridge (computed with $V/V_* = 9$).

though not necessarily associated with greater overall water depth, H (see also Fig. 1). Considering that such an occurrence is not inconceivable, it is of interest to examine the effects of a hypothetical jam with the same maximum H as that of the actual jam but with larger length. Figure 7 shows the assumed initial profile of the hypothetical jam: a constant water depth, equal to the maximum overall depth associated with the actual jam, is assumed to occur in a reach of length L_e, and a horizontal water surface transition is drawn between this reach and the uniform-flow, open-water reach upstream. Figure 8 shows the resulting peak stage at MacEwan Bridge plotted versus L_e using $V/V_* = 9.0$; for $L_e = 25$ km, this peak would have been 1.3 m higher than the one that actually occurred. The main effect of L_e on V is associated with the duration of surging velocities. For $L_e = 0$, Fig. 6 indicates a maximum of 2.3 m/s for V, whereas velocities in excess of 2 m/s lasted for about 45 min. For $L_e = 25$ km, the maximum value of V was calculated as 2.35 m/s but velocities larger than 2 m/s persisted for 110 min.

Discussion

From the foregoing analysis, it appears tht a one-dimensional, unsteady, open-water flow model can be applied to the ice–water flow that results from the release of an ice jam using appropriate definitions of the mass and momentum fluxes. Realistic predictions can be made with this approach provided a suitable value is selected for the coefficient V/V_*. At this time, it is not known how this coefficient is to be predicted because of complications arising from the frequent existence of solid ice sheets below an ice jam. For the present study, the best value of V/V_* was found equal to 9, which is between the open-water value ($\simeq 16$) and the apparent value (5) for flow under a 1-m-thick ice cover. The apparent value of V/V_* is defined as the ratio of the apparent V (equal to q/H) to the apparent V_* (equal to $(gHS_f)^{1/2}$); S_f = energy slope). The apparent V/V_* applies when the cover is stationary but has to increase when the cover is set in motion. Additional case studies would help to develop a method for predicting suitable values of V/V_*.

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

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

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

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

(i) surge characteristics;

(ii) unsteady flow equations under a stationary fragmented ice cover (new ice jam);

(iii) mechanisms of upstream propagation and vertical growth (thickening) of an ice jam;

(iv) the downstream boundary condition, that is, discharge or depth variation with time at the jam toe;

(v) stability of the jam toe.

Item (i) can be dealt with using the approach presented herein and item (ii) has already been discussed in an earlier section where continuity and momentum equations were developed (see also similar equations derived by Uzuner and Kennedy (1976) for flow under a stationary cover). For a situation where an ice jam lengthens in the upstream direction, two flow models must be applied simultaneously: a model of ice–water flow for the region upstream of the jam head and a model for flow under a stationary cover for the region downstream of the jam head. The location of the boundary between these regions depends on time in a manner dictated partly by item (iii) and partly by the incoming ice discharge which is related to item (i). Item (iii) can be formulated so as to be consistent with generally accepted theoretical developments to date (see for example Kennedy 1958; Pariset *et al.* 1966; Uzuner and Kennedy 1976). Some attempts to formulate mathematically the propagation and thickening of an ice jam have already been made (Uzuner and Kennedy 1976; Mercer and Cooper 1977) but the resulting models have not as yet been tested against laboratory or field data.

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

When a jam toe is grounded, the downstream boundary condition may be formulated in terms of a seepage type of equation which relates the discharge to the water depths upstream and downstream of the grounded portion. If it is assumed that, at the time of formation of the toe, the flow is stopped completely, i.e., the discharge becomes zero momentarily, the upstream depth will subsequently increase and the downstream depth will

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

decrease: this will establish a hydraulic gradient which, in turn, will cause the discharge to increase. This concept could be formulated mathematically and incorporated in an overall model of jam formation; however, there is an additional consideration that requires investigation. As the hydraulic head across the jam toe increases, the seepage force also increases, whereas the ability of the jam to resist this force may decrease if increased water stages cause partial flotation of the grounded ice. Therefore there must be a limit of stability beyond which the jam would fail and move downstream but it is not known how a pertinent criterion should be expressed quantitatively. It would thus appear that research on the mechanics of grounded jams is necessary before a complete model of ice jam formation can be produced.

Summary and conclusions

The results of a preliminary investigation into the mechanics of surges due to ice jam releases have been reported in the previous sections. The investigation was prompted by a recent report (Doyle and Andres 1979) that includes a partially documented case of ice jam release.

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

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

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

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

Acknowledgments

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J. [illegible], Chief.
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Western Reservoir and Stream Habitat Improvements Handbook

Guide to the Performance of Fish and Wildlife Habitat and Population Improvement Measures Accompanying Water Resource Development

by

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Environmental Guidelines

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New species suited to future conditions.
- Structures to provide access to habitats for migratory species.
- Extension of the area available to certain species.
- Promotion of certain species to the detriment of others.

Water Quality

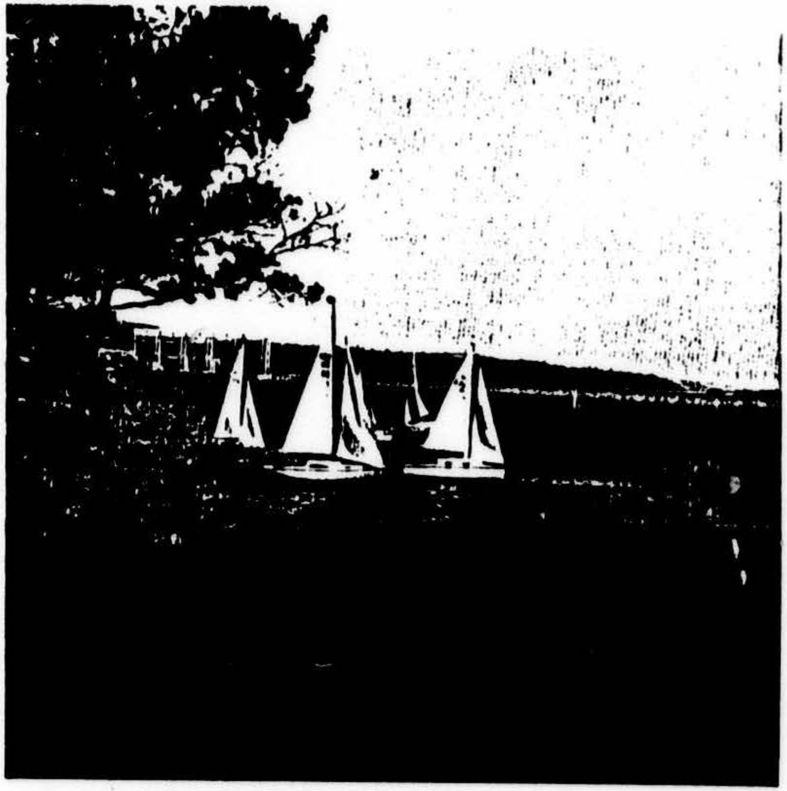
The major parameters defining water quality include temperature, turbidity, dissolved oxygen and nitrogen, and nutrients. Aquatic life depends on the inter-relationship of all these factors.
To minimize the effects of a hydro-electric project on the physical, chemical, and biological quality of the environment, the planning and design of the structures should take into account:
- Present and potential uses of the water upstream and downstream.
- Effects of water storage on dissolved oxygen and water temperature, and repercussions on aquatic life.
  Downstream effects on nitrogen saturation.
- Effects of organic matter and nutrients carried by flood waters to the reservoir, where sedimentation is accelerated.

Ground Water Levels

Adjacent land where the ground-water level may rise should be studied to determine:
- Effects of higher ground-water levels.
- Present and future uses including potential with regard to forestry, agriculture, mining, tourism, wildlife and plant life.

Reservoir Clearing

Reservoirs can be clear cut, or partially cleared. Decisions on the degree of clearing should be consistent with environmental uses of the reservoir within the operating levels. Consider the following:
- Wildlife habitat, boating, fishing, swimming, water skiing, as well as sightseeing.
- Proximity and accessibility of the reservoir to urban, industrial, agricultural, and wilderness areas.
- Fish and bird reproduction can be enhanced, in some instances, if portions of reservoirs are not cleared.
- The salvage of as much wood material as is economically possible.
- Waterway habitat improvement, including bank stabilization, control of wind and water erosion in the zone of fluctuation and replanting of mudflat areas with water adaptable species.



Erosion, Sedimentation, and Ice

A reservoir presents a potential source of erosion and sedimentation. To the degree it is economically and technically possible, siting and design should ensure stability of the banks, prevent land from sliding or breaking, avoid the silting of the reservoir, and achieve erosion protection in harmony with the natural surroundings. Similarly, the design of structures and remedial or protective devices should take into account the behaviour of ice and frazil ice.

Construction

Construction activities can be carried on in a way which minimizes damage to the environment. Before work begins, a plan to protect and manage the environment should be developed and conveyed to the construction forces.

Excavation, Borrow Pits, and Quarry Areas

Borrow pits, quarries and disposal areas are unsightly.

- Choose disposal areas for surplus excavation with care so that the landscaping of these spoil areas, when abandoned, will be compatible with the natural surroundings.
- Locate pits and quarry sites within the area of the future reservoir if possible. Landscape and stabilize exploited areas outside the limits of the reservoir on completion, including slopes and faces.

Operation

Reservoir

During the filling of the reservoir, protect the water and animal and plant life, taking into account the rights of other users of the river.
Permissible reservoir fluctuations should be established by a study of the energy production and environmental requirements. The best solution that takes these two factors into account should be applied. It may be necessary to apply more severe operating restrictions during migration or spawning. They should be co-ordinated with fish and wildlife management authorities.
Consider the following elements in the operation of a reservoir:

— Stability of river or reservoir banks affected by sharp fluctuations in water level.
— Protection of the rights or property owners by ensuring operation within minimum and maximum operating flows and levels.
— Monitoring variations in ground-water levels caused by operation of a reservoir.
— Appearance of areas affected by variations in levels.
— Surveillance of water quality and sedimentation.
— Protection of flora and fauna, taking into account reproductive cycles.
— Recreational activities such as fishing, pleasure boating, camping and swimming.

Rights of Other Users

The rights of other users upstream and downstream should be protected during operation. Clear operating limits should be defined for various conditions. Other user rights include:

— Floating of logs
— Navigation
— Drainage, irrigation, industrial and residential water supply, and waste water discharges.
— Appearance of the area
— Water quality
— Recreational uses, such as for fishing, swimming and camping
— Commercial fishing

De-Commissioning

De-Commissioning of a hydro-electric plant involves taking it out of service and transferring all rights to the water and land to others, and usually occurs when its useful life is completed and rehabilitation or redevelopment is not economically justified.
The economic analysis should consider the environmental effect of de-commissioning, especially on the uses that have developed over the plant's lifetime. These may include recreation, irrigation, flood control, conservation, water supply, and fish farming. Returning a site to its original state should be considered, although it is seldom practical to restore levels and flows to their natural state when the ecology and human population have adapted to a changed regime.

- Consider the visual quality of the de-commissioned structures and site in the economic analysis.

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Articles

Original Signed by
J. E. SLATER
Original Signé par

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3 Supply Officer	949-3297
Admin Services	949-2912
Assistant	949-2912
Clerk	949-6355
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Clerks	949-3181
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'rocessing Unit	949-2130
'er Centre	949-2130

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Records Officer	949-2158
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e Bldg
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Supvr, Accounts	949-4033
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President, Raymond Richards	566-9653
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Vice-President, Water Shed Management J.F. Danyluk	565-6400
Vice-President, Resource Management D.L. MacLeod	565-6220
Vice-President, Water Supply and Utility Management R.S. Pentland	565-2663
Comptroller, V.C. Fowke	566-9521

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Newfoundland Commission of Public Utilities

Prince Charles Bldg
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THALLIUM

Thallium is a trace metal with chemical properties similar to those of aluminum. Even though thallium has two valence states (Tl^{1+}, Tl^{3+}) in aqueous solution the monovalent ion (Tl^{1+}) is the most stable.

Environmental Range

Thallium is present in only trace amounts in fresh water. Typical thallium concentrations in seawater are less than 0.000 01 mg/L (10 ng/L).

Sources

Thallium is present in pyrite (FeS_2) and is recovered from the roasting of this ore in connection with the production of sulphuric acid. It is also obtained from the smelting of lead and zinc ores.

Thallium forms alloys with other metals and readily amalgamates with mercury. It also forms a wide variety of compounds with varied uses: thallium sulphide is used in photocells, thallium bromide-iodide crystals are used as infrared detectors, thallium with sulphur or selenium and arsenic, and thallium oxide are used to produce special glasses; other thallium compounds are used as dyes, pigments in fireworks, and as depilatories. Since thallium sulphate is odourless and tasteless, giving no warning of its presence, and is an effective cumulative poison, it is used as a rodenticide and as an ant killer.

Water Quality Guidelines

Thallium is a cumulative poison and has sublethal effects such as hair loss and high blood pressure, however, no drinking water or livestock water guidelines have been set.

Although thallium has been shown to inhibit photosynthesis and plant transpiration by interfering with stomatal function, no guidelines have been set for thallium levels in irrigation water.

Thallium acts as a neuro-poison in fish and aquatic invertebrates. Thallium concentrations of 0.1 mg/L constitute a hazard in the marine environment, whereas levels less than 0.05 mg/L present a minimal risk of deleterious effects (Environmental Studies Board, 1973). No freshwater guidelines have been set.

TIN

Tin, a minor constituent of the earth's crust, occurs only in minute quantities in natural water. Stannous hydroxide is soluble in water but can only be present in appreciable concentrations above the pH range of natural water.

Environmental Range

Freshwater concentrations of tin are minute and seawater contains only 0.0008 mg/L tin.

Sources

Tin is released by the weathering of igneous rocks that contain the mineral cassiterite (SnO_2). It is not usually transported in solution but is sorbed to clay minerals or remains as a resistate deposit.

Tin is used principally to coat other metals in order to prevent corrosion or other chemical action; for example, the tin plating of steel is used in the production of containers (tin cans) for the food processing industry. This element is also used in alloys such as pewter. Tin is unsurpassed by any other metal in the multiplicity of its organic applications. Organotin compounds are used as stabilizers for polyvinyl chloride, industrial catalysts, industrial and agricultural biocides, and wood-preserving and anti-fouling agents. Thus, a variety of industries may discharge wastes containing tin into receiving waters.

Water Quality Guidelines

Tin itself is not toxic to man, but it has no known physiological function in the human body. Tin is not taken up by plants. Therefore, no water quality guidelines for tin have been established for drinking waters; for agricultural uses; or for freshwater or marine aquatic environments.

TOTAL DISSOLVED SOLIDS

Total Dissolved Solids (TDS) is an index of the amount of dissolved substances in a water. The presence of such solutes alters the physical and chemical properties of water.

The range of dissolved solids is variable (Table 8).

Table 8. Total Dissolved Solids – Salinity Relationships

Total Dissolved Solids mg/L	Degree of Salinity	
0 – 1 000	Fresh; non-saline	
1 001 – 3 000	Slightly saline	Brackish
3 001 – 10 000	Moderately saline	Brackish
10 001 – 100 000	Saline	
>100 001	Brine	

Sources

The base flow of a waterway acquires mineral constituents in the form of dissolved salts in solution, such as sodium, chloride, magnesium, sulphate, etc. In periods of high surface runoff, overland flow contributes dissolved materials to waters. In addition, significant contributions to the TDS load are anthropogenic in the form of municipal d industrial effluents, agricultural runoff, and aerosol llout.

ater Quality Guidelines

Basic guidelines on the concentration of TDS which ıve been established relate to taste and palatability rather an to detrimental health effects on man and aquatic ota. TDS concentration less than 500 mg/L has been ısignated as an objective level for drinking water (Department of National Health and Welfare, 1969). It has been cognized that concentrations of 1000 mg/L may still be :ceptable for drinking providing none of the individual ssolved constituents exceed their particular guidelines)epartment of National Health and Welfare, 1969). If the ›ncentration exceeds 2000 mg/L definite laxative effects ıve been observed in man.

A similar laxative effect has been shown in livestock. or animals concentrations less than 3000 mg/L have roven to be satisfactory in most circumstances (Environıental Studies Board, 1973).

Industrial users of waters usually prescribe TDS conentrations to be less than 1000 mg/L, but this is quite ariable among individual users and their particular requireıents (U.S. Geological Survey, 1970).

ffects on Use

High concentrations of TDS limit the suitability of a ıater as a drinking source. Industries are sensitive to boiler caling or to accelerated corrosion associated with subtantial amounts of TDS in water. High TDS waters may nterfere with the clarity, colour, and taste of manufactured ›roducts.

TUNGSTEN

Tungsten is a transition metal, chemically similar to hromium, molybdenum and tantalum. Its physical and hemical properties make it suitable for many commercial nd industrial applications. Tungsten occurs naturally in everal tungstate minerals and is obtained commercially rom these ores by reducing tungstic oxide to the metal.

Environmental Range

Most freshwaters contain negligible concentrations of ungsten. Seawater typically exhibits concentrations of .0001 mg/L tungsten.

ources

Tungsten occurs naturally in the minerals wolframite (Fe,Mn)WO_4], huebnerite ($MnWO_4$), ferberite ($FeWO_4$), nd scheelite ($CaWO_4$). Tungsten is very insoluble in water.

Tungsten and its alloys are used extensively for filaments, electric lamps, electron and television tubes, electrical contact points for automobile distributors and for numerous high temperature applications, including space missiles. Tungsten carbide (W_2C) is used in the metalworking, mining, and petroleum industries. Calcium and magnesium tungstates are used in fluorescent lighting; and other salts of tungsten are used in the chemical and tanning industries.

Water Quality Guidelines

No water quality guidelines have been formulated for tungsten.

[redacted]

Turbidity is a measure of the suspended particles such as silt, clay, organic matter, plankton, and microscopic organisms in water which are usually held in suspension by turbulent flow and Brownian movement. Turbidity is measured by comparing the optical interferences of suspended particles to the transmission of light in water in an instrument previously standardized with samples of standard turbidity units.

Environmental Range

It is impractical to assign a range of values to turbidity, however, non-detectable turbidity may be approximated by pure distilled water (0 Jackson Turbidity Units [JTU]). Values of 1000 JTU may be encountered in wastewaters; waters with very high natural turbidity may be in the range of several hundred JTU.

Sources

The amount of solid materials in suspension in water may result from natural erosion, runoff, and algal blooms, although man may contribute to the presence of such materials. The concentration and particle size of these suspended materials may cause significant variation of turbidity values. Turbidity is high during spring runoff.

Water Quality Guidelines

High turbidity reduces photosynthesis of submerged, rooted aquatic vegetation and algae; this reduced plant growth may in turn suppress fish productivity. Turbidity, therefore, can affect aquatic biological communities. Water quality guidelines suggest that discharges resulting from human activity should not alter ambient turbidity levels.

Turbidity, unless related to asbestiform minerals, does not affect the safety of a drinking water, but does alter its consumer acceptability. Although water with a turbidity of 5 JTU or less is acceptable for drinking, a value of less than

MARCH 1973

PPWB
WATER QUALITY
OBJECTIVES

TO BE USED FOR THE PURPOSES OF THE PPWB

[redacted]
305 BRENT BUILDING 2505 11TH AVENUE REGINA, SASKATCHEWAN S4P 0K6

CANADA · ALBERTA · SASKATCHEWAN · MANITOBA

SURFACE WATER QUALITY OBJECTIVES

These objectives represent water quality suitable for most uses either through direct use or prepared for use by an economically practical degree of treatment. They apply to surface waters except in areas of close proximity to outfalls.

There are many instances where the natural water quality of a lake or river does not meet some of the suggested limits. In these cases, the limits obviously will not apply. It should be noted, however, that where the natural existing quality is inferior to desirable criteria, it would be unwise to permit further deterioration by unlimited or uncontrolled introduction of pollutants. Naturally occurring circumstances are not taken into account in these objectives and due consideration must be given where applicable (e.g. spring runoff effect on colour, odour, etc.).

---

1. Bacteriology (Coliform Group)

(a) In waters to be withdrawn for treatment and distribution as a potable supply or used for outdoor recreation other than direct contact, at least 90 percent of the samples (not less than five samples in any consecutive 30-day period) should have a total coliform density of less than 5,000 per 100 ml and a fecal coliform density of less than 1,000 per 100 ml.

(b) In water used for direct contact recreation or vegetable crop irrigation the geometric mean of not less than five samples taken over not more than a 30-day period should not exceed 1,000 per 100 ml total coliforms, nor 200 per 100 ml fecal coliforms, nor exceed these numbers in more than 20 percent of the samples examined during any month, nor exceed 2,400 per 100 ml total coliforms on any day.

2. Dissolved Oxygen

A minimum of five mg/l at any time.

3. Biochemical Oxygen Demand

Dependent on the assimilative capacity of the receiving water, the BOD must not exceed a limit which would create a dissolved oxygen content of less than five mg/l.

4. Suspended Solids

Not to be increased by more than 10 mg/l over background value.

5. pH

To be in the range of 6.5 to 8.5 pH units but not altered by more than 0.5 pH units from background value.

6. Temperature

Not to be increased by more than 3°C above ambient water temperature.

7. Odour

The cold (20°C) threshold odour number not to exceed eight.

8. Colour

Not to be increased more than 30 colour units above natural value.

[redacted]

Not to exceed more than 25 Jackson units over natural turbidity.

WATER QUALITY

OBJECTIVES

January 1975

Second Printing
Feb. 1977

Water Pollution Control Branch
Environment Saskatchewan
5th Floor, 1855 Victoria Ave.
Regina, Saskatchewan S4P 3T1

There are many instances where the natural water quality of a lake or river does not meet some of the suggested limits. In these cases, the limits obviously will not apply. It should be noted, however, that where the natural existing quality is inferior to desirable objectives, it would be unwise to permit further deterioration by unlimited or uncontrolled introduction of pollutants. Naturally occurring circumstances are not taken into account in these "Objectives" and due consideration must be given where applicable (e.g. spring runoff effect on colour, odour, etc.)

Table 1

SURFACE WATER QUALITY OBJECTIVES

These objectives have been prepared in co-operation with the Provinces of Alberta and Manitoba and represent water quality suitable for most uses either through direct use or prepared for use by an economically practical degree of treatment.

Parameter	Objectives
1. Bacteriology (Coliform Group)	(a) In waters to be withdrawn for treatment and distribution as a potable supply or used for outdoor recreation other than direct contact, at least 90 per cent of the samples (not less than five samples in any consecutive 30-day period) should have a total coliform density of less than 5,000 per 100 ml and a fecal coliform density of less than 1,000 per 100 ml. (The Maximum Permissible Limit of total coliform organisms in a single sample shall be determined by the Department based on the type and degree of pollution and other local conditions existing within the watershed.) (b) In waters used for direct contact recreation or vegetable crop irrigation the geometric mean of not less than five samples taken over not more than a 30-day period should not exceed 1,000 per 100 ml total coliforms, nor 200 per 100 ml fecal coliforms, nor exceed these numbers in more than 20 per cent of the samples examined during any month, nor exceed 2,400 per 100 ml total coliforms on any day.
2. Dissolved Oxygen	A minimum of five mg/1 at any time.
3. Biochemical Oxygen Demand	Dependent on the assimilative capacity of the receiving water. The BOD must not exceed a limit which would create a dissolved oxygen content of less than five mg/1.
4. Suspended Solids	Not to be increased by more than 10 mg/1 over background value.
5. pH	To be in the range of 6.5 to 8.5 pH units but not altered by more than 0.5 pH units from background value.
6. Temperature	Not to be increased by more than 3°C above ambient water temperature.
7. Odour	The cold (20°C) threshold odour number not to exceed eight.
8. Colour (Apparent)	Not to be increased more than 30 colour units above natural value.
[redacted]	Not to exceed more than 25 turbidity units over natural turbidity.

[redacted]

SURFACE WATER QUALITY OBJECTIVES

Water Quality Branch

Standards and Approvals Division

January, 1977

TABLE 1

SURFACE WATER QUALITY OBJECTIVES

(see explanatory notes for definition of parameters)

These objectives represent water quality suitable for most uses either through direct use or prepared for use by an economically practical degree of treatment. They apply to surface waters except in areas of close proximity to outfalls.

There are many instances where the natural water quality of a lake or river does not meet some of the suggested limits. In these cases, the limits obviously will not apply. It should be noted, however, that where the natural existing quality is inferior to desirable criteria, it would be unwise to permit further deterioration by unlimited or uncontrolled introduction of pollutants. Naturally occurring circumstances are not taken into account in these objectives and due consideration must be given where applicable (e.g. spring runoff effect on colour, odour, etc.).[8]

---

1. Bacteriology (Coliform Group)

   (a) In waters to be withdrawn for treatment and distribution as a potable supply or used for outdoor recreation other than direct contact, at least 90 per cent of the samples (not less than five samples in any consecutive 30-day period) should have a total coliform density of less than 5,000 per 100 ml and a fecal coliform density of less than 1,000 per 100 ml.

   (b) In water used for direct contact recreation or vegetable crop irrigation the geometric mean of not less than five samples taken over not more than a 30-day period should not exceed 1,000 per 100 ml total coliforms, nor exceed these numbers in more than 20 per cent of the samples examined during any month, nor exceed 2,400 per 100 ml total coliforms on any day.

2. Dissolved Oxygen

   A minimum of five mg/l at any time.

3. Biochemical Oxygen Demand

   Dependent on the assimilative capacity of the receiving water, the BOD must not exceed a limit which would create a dissolved oxygen content of less than five mg/l.

4. Suspended Solids

   Not to be increased by more than 10 mg/l over background value.

5. pH

   To be in the range of 6.5 to 8.5 pH units but not altered by more than 0.5 pH units from background value.

6. Temperature

   Not to be increased by more than 3°C above ambient water temperature.

7. Odour

   The cold (20°C) threshold odour number not to exceed eight.

8. Colour

   Not to be increased more than 30 colour units above natural value.

[illegible]

   Not to exceed more than 25 Jackson units over natural turbidity.

SURFACE WATER QUALITY OBJECTIVES

There are many legitimate uses of water within the province. Some of these uses have been mentioned briefly earlier. The range of acceptable water quality (as measured in terms of significant physical, chemical and biological parameters) varies depending upon the particular requirement.[(6)] The many uses may be ranked according to the range of water quality that can be tolerated for the particular application. Involved in the ranking are other factors such as the availability of treatment technology, economic and social costs to individuals, groups, industry and to the province as a whole, relative water demand for a specific purpose and the susceptibility of fish and wildlife to minor or major changes. It has usually been agreed that water for public water supply, recreation involving direct water contact, wildlife and aquatic life protection require the highest quality of water with the minimum allowable variance.

Alberta, in association with Saskatchewan and Manitoba, has established minimum water quality guidelines which would allow the most sensitive use. By doing so, all other demands involving lesser quality or demands more tolerant to wider variation would also be satisfied.

The surface water quality objectives for Alberta are presented in Table 1. The numerical values established for the various parameters listed in Table 1 represent a goal which should be achieved or surpassed. In certain regions of the province, natural conditions result in levels of certain metals, nutrients or other water quality parameters which exceed those presented as objectives. This does not mean that the objectives should be revised to account for these particular situations. Rather, these individual occurrences will be treated as special cases and reviewed accordingly. The surface water quality objectives apply to all surface waters in Alberta.

SUMMARY TABLE

[redacted] WATER QUALITY OBJECTIVES

PARAMETER	UNITS	1A	1B	1C	1D	2A	2B	2C	3A	3B	3C	4A	4B	4C	5
Sulfate (SO_4)	mg/1	250.	250.	250.					250.			250.	400.	500.	
Sulfide (H_2S)	mg/1					0.002	0.002	0.002				3.	5.		0.02
Temperature	C°					(O)0.5	(O)1.0	(O)2.0							
Thallium (Tl)	mg/1					0.05	0.05	0.05							
Threshold Odour Number		4.	4.	4.					4.						
Total Coliform Organisms	Counts /100 ml	NEG. (Y)	100. (Z)	1000. (B)	5000. (F)	100. (D)	500. (R)	1000. (U)	100. (Z)	5000. (F)					
Total Dissolved Solids	mg/1	500.	500.	1000.					500.			700.	1500.	3000.	
Toxic Algae														(BB)	
Turbidity Valu[illegible]	JTU	5.	5.	25.	[illegible]	10.	25.	25.	5.	[illegible]	[illegible]				
Unspecified Toxic Substances														(CC)	
Uranium (U)	mg/1					(DD)	(DD)	(DD)							
Vanadium (V)	mg/1											0.1	0.1	0.1	
Zinc (Zn)	mg/1	5.	5.	5.		(DD)	(DD)	(DD)	5.			2.0	2.0	25.	

* See Notes Appendix D Annex 4 Pages v and vi

II. 2. Class 2A

The quality of this class of the waters of the province shall be such as to permit the propagation and maintenance of warm and cold water sport or commercial fishes and be suitable for aquatic recreation of all kinds, including bathing, for which the waters may be usable. Limiting concentrations or ranges of substances or characteristics which should not be exceeded in the waters are given below.

Substance or Characteristic	Acceptable Limit or Range
Ammonia (N), unionized	0.02 milligrams per liter
Cadmium	0.01 milligram per liter for waters with hardness greater than 100 milligrams per liter ($CaCO_3$) 0.004 milligrams per liter for waters with hardness lower than 100 milligrams per liter ($CaCO_3$)
Chlorides (Cl)	200 milligrams per liter
Chlorine (Cl_2)	0.01 milligram per liter
Chromium (as Cr + 6)	0.1 milligram per liter
Colour value	30
Copper (Cu)	0.02 milligrams per liter or not greater than 1/10 the 96-hour LC50 value
Cyanides (CN)	0.005 milligrams per liter
Dissolved oxygen	60% or more saturation at the ambient temperature
Faecal Coliform Organisms	The median (50 percentile) based on not less than 5 samples per month should be not greater than 20 MPN per 100 milliliters.
Fluorides (F)	1.5 milligram per liter
Lead (Pb)	0.03 milligrams per liter
Mercury (Hg)	0.0002 milligrams per liter
Nickel (Ni)	0.025 milligrams per liter
pH value	6.5 - 8.5
Polychlorinated biphenyls (PCB)	0.000002 milligrams per liter

II. 2.

Substance or Characteristic	Acceptable Limit or Range
Radioactive materials	Not to exceed the lowest concentrations permitted to be discharged to an uncontrolled environment as prescribed by the appropriate authority having control over their use.
Selenium (Se)	0.01 of the 96-hour LC50 value
Silver (Ag)	0.01 of the 96-hour LC50 value
Sulfides (H_2S)	0.002 milligrams per liter
Temperature	No change greater than 0.5° Celsius beyond natural minimum and maximum temperatures
Thallium (Tl)	0.05 milligrams per liter
Total Coliform Organisms	The median (50 percentile) based on not less than 5 samples per month should be not greater than 100 MPN per 100 milliliters.
[redacted]	
Uranium (U)	0.01 milligram per liter or .01 of the 96-hour LC50 value
Zinc (Zn)	.01 of the 96-hour LC50 value

Class 2B

The quality of this class of the waters of the province shall be such as to permit the propagation and maintenance of cool or warm water sport or commercial fishes and be suitable for aquatic recreation of all kinds, including bathing, for which the waters may be usable. The physical and chemical objectives quoted above for Class 2A shall also apply to these waters except as listed below.

Substance or Characteristic	Acceptable Limit or Range
Dissolved Oxygen	47% or more saturation at the ambient temperature
Faecal Coliform Organisms	The median (50 Percentile) based on not less than 5 samples per month should be not greater than 200 MPN per 100 milliliters
pH value	6.5 - 9.0
Temperature	No change greater than 1.0° Celsius beyond natural minimum and maximum temperatures.

II. 2.

Substance or Characteristic	Acceptable Limit or Range
Total Coliform Organisms	The median (50 percentile) based on not less than 5 samples per month should be not greater than 500 MPN per 100 milliliters.

[redacted]

Class 2C

The quality of this class of the waters of the province shall be such as to permit the propagation and maintenance of rough fish or species commonly inhabiting waters of the vicinity under natural conditions, and be suitable for boating and other forms of aquatic recreation for which the waters may be usable. The physical and chemical objectives quoted above for Class 2A shall also apply to these waters except as listed below.

Substance or Characteristic	Acceptable Limit or Range
Colour value	100
Dissolved Oxygen	35% or more saturation at the ambient temperature
Faecal Coliform Organisms	The median (50 percentile) based on not less than 5 samples per month should be not greater than 400 MPN per 100 milliliters.
pH value	5.0 - 9.3
Temperature	No change greater than 2.0° Celsius beyond natural minimum and maximum temperatures.
Total Coliform Organisms	The median (50 percentile) based on not less than 5 samples per month should be not greater than 1000 MPN per 100 milliliters.

[redacted]

3. INDUSTRIAL CONSUMPTION

Class 3A

The quality of this class of the waters of the province shall be such as to permit their use without chemical treatment, for most industrial purposes, except food processing and related uses, for which a high tative control location plus 10°C (18°F) or the allowed temperature difference, whichever is the lesser temperature. These maximum temperatures are to be measured on a mean daily basis from continuous records.

(c) Taking and Discharging of Cooling Water

Users of cooling water shall meet both the Objectives for temperature outlined above and the "Procedures for the Taking and Discharge of Cooling Water" as outlined in the Implementation Procedures for Policy 3 (page 15).

Total Dissolved Solids — Dissolved Solids must not be added to increase the ambient concentrations by more than 1/3 of the natural concentrations to protect aquatic life. The added solids should not significantly alter the overall ionic balance of the receiving waters.

[illegible] — Suspended matter should not be added to surface water in concentrations that will change the natural Secchi disc reading by more than 10 percent.

HEAVY METALS

The following metal Objectives (except for mercury) are based on the total concentration of an unfiltered water sample. It is recognized that metals may not be toxic in particulate or bound form. It is quite possible for the total concentration in a given surface water to exceed the Objective without damaging any aquatic life. However, in the absence of any standard technique to measure the toxic components, it is assumed that all of the metal is in a toxic form unless specific data show otherwise. Further, an asterisk is placed before three parameters, namely arsenic, chromium and selenium, which have Objectives for aquatic life protection that are less stringent than the criteria for other beneficial uses.

***Arsenic** — Concentrations of Arsenic in an unfiltered sample should not exceed 100 $\mu g/L$ to protect aquatic life.

Beryllium — For the protection of aquatic life, concentrations of beryllium should not exceed 11 $\mu g/L$

in an unfiltered sample of hardness less than 75 mg/L of $CaCO_3$ and not exceed 1100 $\mu g/L$ in an unfiltered sample of hardness greater than 75 mg/L of $CaCO_3$.

Cadmium — Concentrations of cadmium in an unfiltered sample should not exceed 0.2 $\mu g/L$ to protect aquatic life.

***Chromium** — Concentrations of chromium in an unfiltered sample should not exceed 100 $\mu g/L$ to protect aquatic life.

Copper — Concentrations of copper in an unfiltered sample should not exceed 5 $\mu g/L$ to protect aquatic life.

Iron — Concentration of iron in an unfiltered sample should not exceed 300 $\mu g/L$ to protect aquatic life.

Lead — The toxicity of lead is highly dependent on the alkalinity of the water. The toxicity declines as the alkalinity increases. The total lead concentration should not exceed the values given below:

Alkalinity mg/L as $CaCO_3$	Maximum lead Concentration $\mu g/L$
Up to 20	5
20 to 40	10
40 to 80	20
greater than 80	25

Mercury — Concentrations of total mercury in *filtered* water should not exceed 0.2 $\mu g/L$ nor should the concentrations of total mercury in whole fish exceed 0.5 $\mu g/g$ (See Table 2).

Nickel — Concentrations of nickel in an unfiltered sample should not exceed 25 $\mu g/L$ to protect aquatic life.

***Selenium** — Concentrations of Selenium in an unfiltered sample should not exceed 100 $\mu g/L$ to protect aquatic life.

Table 3. (Cont'd)

Parameter		Sources and Objectives (a)				
		[redacted]	[redacted]	Australia 1974	U.S. EPA 1976	I.J.C. Great Lakes 1974-76
PHYSICAL						
pH	units	7.0-9.2	7.0-9.2	7.0-9.2	–	–
Settleable substances		Minimized	Minimized	–	–	Free
Temperature		–	Avoid changes in natural freezing patterns	–	–	–
Toxic growths		Free of heavy algal growths	–	–	–	–
[redacted]		<5% incident light at surface not to be exceeded on 7 consecutive days.	<10% change in compensation point	–	–	–

(a) Canada, 1972 – Guidelines for Water Quality Objectives and Standards. Dept. of Environment, Inland Waters Directorate, Ottawa, Ontario. Technical Bulletin 67.
U.S., NAS/NAE, 1973 – Water Quality Criteria 1972. Environmental Protection Agency, Washington, D.C., Pub. EPA-R3-73-33.
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I.J.C., Great Lakes, 1974-76 – Appendices "A" to Great Lakes Water Quality Board Reports 1974, 1975 and 1976 to the International Joint Commission, 100 Ouellette Avenue, Windsor, Ontario.

(b) Substantially absent–meaning less than detection levels as determined by the best scientific methodology available.

PARAMETER	USE AND AGENCY	CRITERIA, STANDARDS, OBJECTIVES, AND GUIDELINES
[redacted]	*Raw Water*	
	US Water Quality Criteria	
	Warm water	50 Jackson units
	Cold water	10 Jackson units
	Canadian Federal Guidelines	
	Objective:	5 Jackson units
	Michigan State Standard	no quantity to cause injury
	Minnesota State Standard	5 Jackson units
	Ontario Provincial Criteria	
	Desirable:	absent
	Permissible:	absent
	Drinking Water	
	US Drinking Water Standard	Daily maximum = 1 unit
	Canadian Drinking Water Objectives	
	Objective:	<1 Jackson unit
	Acceptable:	5 Jackson units
	Wisconsin State Standard	1 Jackson unit
	Ontario Provincial Drinking Water Objectives	1 Jackson unit
	Recreation	
	Canadian Federal Guidelines	
	Objective:	<5 Jackson units
	Maximum:	50 Jackson units
	[redacted]	
	Minnesota State Standard	10 Jackson units
	Ontario Provincial Criteria Associated with waste inputs	
	Warm water	not to exceed 50 Jackson units
	Cold water	not to exceed 10 Jackson units
	Lakes:	
	Warm water	not to exceed 25 Jackson units
	Cold water or oligotrophic	not to exceed 10 Jackson units
URANYL ION	*Raw Water*	
	US Water Qualtiy Criteria	
	Desirable:	absent
	Permissible:	5 mg/ℓ
	Canadian Federal Guidelines	
	Objective:	<1.0 mg/ℓ
	Acceptable:	5.0 mg/ℓ

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Ice Jams

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Present ice jam knowledge is briefly reviewed and assessed in conjunction with practical needs. It is shown that only upper-bound estimates of ice jamming severity are possible at present. Major research needs are identified.

Introduction

A major consequence of ice formation in rivers is the jamming that occurs during the breakup of the ice cover and, less importantly, during the freeze up period. Due to their large aggregate thickness and hydraulic resistance, jams can cause unusually high water stages with repercussions to flooding, damage to structures, channel erosion and interference with navigation. Ice jams are extremely complex phenomena, accurately described as (16) "... chaotic disorderly untidy affairs." To date, research has concentrated on the relatively simple problem of floating equilibrium jams and a start has been made recently on jam dynamics. However, many aspects of ice jams are either insufficiently or not at all understood, e.g., initiation, evolution and release mechanisms as well as design criteria for control measures. This paper attempts to summarize the existing knowledge and identify gaps which seriously hamper progress.

Types of Ice Jams

An ice jam has been defined (9) as "an accumulation of ice at a given location which, in a river, restricts the flow of water." This definition encompasses several jam types of which the behavior and effects may differ, as outlined next.

Freeze up and breakup jams: The former type is subject to freezing of water between ice fragments thus being better able to resist the applied forces. Because, in addition, breakup discharges are usually much larger than freeze up ones, the breakup jam has a greater damage potential and is thus given emphasis in this paper.

Floating and grounded jams: A jam may be grounded, if it extends to the river bed, or floating, if it permits unobstructed flow beneath its lower boundary. Very little is known about either grounded jams or seepage through fragmented ice accumulations. The latter is invariably neglected when dealing with floating jams.

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Evolving and steady-state jams: Jams are essentially unsteady phenomena as witnessed by their transient nature, i.e., initiation, formation, release. However, under certain circumstances, a jam may attain a steady-state condition. A steady, floating jam may be long enough to contain an "equilibrium" reach, along which the jam thickness and flow depth are approximately uniform. It can be shown that the flow depth is largest in the equilibrium reach.

Behavior of Ice Jams

Initiation: Commonly, jams are initiated by the combined effects of a stationary ice sheet and one or more morphological or man-made features, e.g., constrictions, bends, shallows, slope reductions, bridge piers (4). Two initiation mechanisms have been studied so far: congestion, i.e., the channel capacity to transport ice fragments may be exceeded depending on ice floe characteristics, ice discharge and local flow velocity and geometry (5, 6, 14); and initiation by a transverse floating obstacle such as a stationary ice sheet or a cover formed by congestion. When an ice floe comes to rest against a floating obstacle, it may or may not submerge depending on its own characteristics and the pressure distribution on its wetted boundary. Ashton's simple theory gives good results under most conditions (see Ref. 1 for a review of pertinent studies):

$$V_c / \sqrt{(1 - s_i)gt_i} = 2(1 - \frac{t_i}{H_u}) / \sqrt{5 - 3(1 - \frac{t_i}{H_u})^2} \qquad (1)$$

in which V_c = critical velocity such that a block of thickness t_i submerges when the average upstream velocity exceeds V_c; g = acceleration of gravity; s_i = specific gravity of block material; and H_u = upstream flow depth. If the blocks do not submerge, a jam comprising a single layer of blocks will be initiated. If the blocks submerge, a multi-layered jam may form depending on the ability of the flow to transport the submerging blocks. Using the "no-spill" condition, Pariset et al (12) predicted the thickness, t, of this type of jam, i.e.:

$$t = V^2/2(1-s_i)g \qquad (2)$$

in which V = average flow velocity under the jam. A different theory, based on energy considerations, was advanced by Tatinclaux (15) and generally predicts larger t's than Eq. 2. If the flow depth is comparable to the size of the submerging blocks, grounding may occur (10) but little else is known about this phenomenon.

The foregoing mechanisms are principally founded on laboratory tests and implicitly assume that ice is unbreakable. In nature, however, one often observes violent ice runs arriving at ice sheet edges where they eventually come to a halt after intense local breaking and piling up. It is unlikely that such events can be successfully described in terms of the above mechanisms.

Evolution and equilibrium: Once a stable toe (downstream end of jam) has formed, the jam lengthens upstream and the streamwise forces applied on it increase. These forces give rise to internal stresses which likewise increase but only to a limiting value, owing to side friction. If the internal stresses become large enough, the jam will collapse and thicken so that it is just able to withstand the (adjusted) stresses. This phenomenon has been analyzed by considering the jam a floating granular mass (12, 16). While this theory has had some success with steady jams (see later discussion), prediction of transient jam parameters is not possible because the toe conditions are unknown (16).

The foregoing suggests two possibilities: (a) a jam formed by floe submergence with a thickness given by Eq. 2 or a related theory, known as the "narrow-channel" jam (12); and (b) a jam formed by collapse, known as the "wide-channel" jam. Under equilibrium conditions, the force balance in a "wide" jam is expressed by (12, 16):

$$0.5(\tau_i + w_i)W = C_i t + 0.5\,\mu\, s_i(1 - s_i)\rho g t^2 \qquad (3)$$

in which W = channel width; τ_i = flow shear stress on jam underside; w_i = streamwise component of jam's own weight per unit area = $s_i \rho g t S$; S = channel slope; C_i = jam cohesion; μ = dimensionless coefficient that depends on the internal friction of the jam; and ρ = water density. Comparison of Eqs. 2 and 3 has shown that "narrow" jams should not occur in any but very small streams, at least during breakup (2).

To determine the equilibrium depth, H, of a floating jam, hydraulic resistance considerations for the flow under the jam can be combined with either Eq. 2 or 3. For the more common, "wide" jam (Eq. 3), this procedure gives (2):

$$\eta \equiv H/WS = F(f_o, f_i/f_o, \mu; \xi) \qquad (4)$$

in which f_o = composite friction factor of the flow under the jam = $0.5(f_i + f_b)$; f_i, f_b = ice and bed friction factors, respectively; and ξ = dimensionless discharge $\equiv (q^2/gS)^{1/3}/WS$, with q = Q/W. Equation 4 neglects C_i which should be a fair approximation for breakup jams (12). The main independent variable in Eq. 4 is ξ, so that field data can be plotted in the form of η versus ξ to define the function F, as shown in Fig. 1. It is seen that the equilibrium jam data points provide a satisfactory relationship while the non-equilibrium ones generally fall below this relationship, as expected. This supports the granular mass theory of ice jams but direct confirmation, based on Eq. 3, is not possible because the thickness of breakup jams is not measurable at present. For a detailed discussion of the implications of Fig. 1 and an alternative method of field data interpretation, see Ref. 2.

Release: How, why and when jams release is unknown but it is suspected that toe conditions and thermal ice deterioration play a role. A sudden jam release can result in very high speeds and rapid stage rises. These can be roughly predicted by ignoring the effects of the moving ice and applying the unsteady, open-water equations of motion (3, 8, 11).

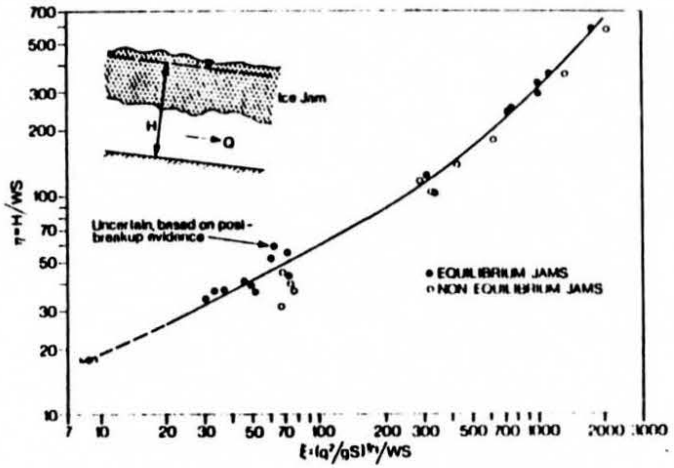

Fig. 1. Dimensionless depth versus dimensionless discharge for floating breakup jams.

Practical Needs and Applications

At this point, it is of interest to consider practical needs and how well the available knowledge addresses them.

Breakup initiation: This event is not directly related to jamming but its study and forecasting are important because it heralds the ice jam period. Moreover, the factors which govern breakup initiation may be relevant to the severity of subsequent jamming (13).

Severity of breakup: At a given site, this could be quantified by the magnitude and duration of high water levels and speeds. In turn, these are related to the magnitude, number and persistence of nearby jams. Present knowledge can only help to identify potential jam sites; it cannot assist in predicting whether, where and when jams will actually form and release. Similarly, only the potential magnitude of ice jams can be estimated by assuming that, for a given discharge, the maximum possible stage is that of an equilibrium floating jam (barring occurrence of severe grounded jams). The latter can be estimated using Fig. 1 or by a more elaborate procedure described in Ref. 2. Whether this potential stage will actually be realized during any one breakup is unknown and depends on many other factors, e.g., proximity of jams that actually form to the site of interest; degree of thermal ice deterioration; and possibility of overbank spreading of water and ice. Examples of applying the equilibrium jam stage approach are given in Ref. 2. Rough estimates of high velocities that may occur during breakup can be made by application of a jam release model (3, 8, 11).

Stage-frequency relationships: The annual peak stage often occurs during breakup at relatively low discharge, hence flood-frequency estimates must take into account the frequency of the maximum breakup stages. The latter can be determined from historical data (7) but such information is often unavailable. In such cases only an upper bound for the breakup stage-frequency relationship can be estimated, using breakup discharge frequencies and the equilibrium jam theory. However, this approach might seriously overestimate the desired frequencies.

Ice jam control: There are two main categories of control measures (4): (a) Ice modification, comprising ice breaking; dusting to promote heat transfer to the ice cover; and artificial increase of water temperature to weaken or suppress formation of ice cover at critical areas. And (b) river modification, mainly consisting of channelization to eliminate undesirable geomorphic features and erection of ice retention or diversion structures. The effectiveness of these methods is uncertain and depends heavily on experience.

Major Unknowns

A brief review of current ice jam knowledge has been presented along with a discussion of how well this knowledge can address practical needs. It was found that only upper-bound estimates of ice jamming severity are possible at present. Several serious gaps in ice jam understanding were identified and these are summarized below.

Jam thickness measurement: The thickness of breakup jams cannot be measured at present. Direct access is hazardous and even where it can be successfully attempted, only a few spot measurements are possible. Development of airborne, remote sensing instrumentation would give a very strong impetus to ice jam research.

Breakability of the ice cover: This aspect has been largely ignored despite its relevance to breakup initiation and ice jam formation and release. Thermal ice deterioration just before and during breakup is pertinent in this regard.

Conditions at ice jam toes: These remain largely unknown even though they govern jam stability and release and have repercussions to ice jam control.

Control methods: In this area, experience is likely to remain our best tool for many years. In the meantime, it would be a good practice to systematically record this experience by monitoring and documenting the performance of control measures that are occasionaly implemented at problem sites.

Conclusions

Present ice jam knowledge is less than satisfactory because it can only furnish upper-bound estimates of ice jamming severity and little guidance for pertinent control methods. Major research needs include development of jam thickness measurement techniques, study of ice breakability and thermal deterioration effects, investigation of toe conditions and systematic recording of ice jam control experience.

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RIVER ICE BREAKUP

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ABSTRACT

A conceptual model of river ice breakup is formulated and used to analyze and compare data from four river gauge sites. Emphasis is on development of generalized short-term forecasting methods which to date have been site-specific. The features to be forecast are the onset and flooding potential of breakup. These are related to the water surface width available for passage of large sheets that form by transverse cracking of the ice cover. Thus it is possible to study the effects of such parameters as ice cover dimensions and channel geometry. Owing to a lack of pertinent data, other parameters such as ice mechanical properties and flow characteristics are only considered indirectly. Using recent observations, a possible mechanism of transverse crack formation is identified. Suggestions for future research and improvement of observational procedures are made and limitations of the model are discussed.

KEYWORDS: breakup; cracks; field data; forecasting; gauge records; ice; ice-clearing; ice sheets; model; onset; river ice; rivers.

RÉSUMÉ

On élabore un modèle conceptuel de la débâcle qui est utilisé pour l'analyse et la comparaison des données provenant de quatre emplacements de jaugeage. On insiste sur la mise au point de méthodes généralisées de prévision à court terme qui, jusqu'ici, ne s'appliquaient qu'à des emplacements particuliers. Les caractéristiques à prévoir sont le moment du déclenchement de le débâcle et les possibilités de crues. Ces caractéristiques sont reliées à la largeur en gueule disponible pour le passage de grandes nappes de glace qui se forment par fragmentation transversale de la couverture glacielle. Il est ainsi possible d'étudier des paramètres comme les dimensions de la couverture glacielle et la géométrie du chenal. En raison d'un manque de données pertinentes, d'autres paramètres comme les propriétés mécaniques de la glace et les caractéristiques de l'écoulement ne sont considéré qu'indirectement. D'après des observations récentes on propose un mécanisme possible de formation de fissures transversales. On présente des suggestions de recherches futures et d'amélioration des méthodes d'observation en plus d'étudier les limites du modèle.

MOTS CLÉS: débâcle, fissures, données de terrain, prévision(s), données de jaugeage, glaces, dégagement des glaces, nappes de glace, modèle, moment du déclenchement, glace de rivière, rivières.

MANAGEMENT PERSPECTIVE

The onset of river ice breakup in the spring and the forces and natural variables controlling it, is a phenomenon, often observed but not understood. It is important that the controlling factors be understood and recognized because of increased management of rivers and the number of settlements close to rivers. This paper proposes and illustrates how some sense may be made from chaos. It illustrates the need for collaborations to ensure useful analysis from the collection of data. Environmental management now demands qualitative relationships rather than disruptions. This paper is an important signpost.

T. Milne Dick, Chief
Hydraulics Division
May 30, 1983

PERSPECTIVE GESTIONNELLE

Le moment du déclenchement de la débâcle printanière ainsi que les forces et les variables naturelles qui la commandent, sont un phénomène observé mais encore mal expliqué. Il est important que les facteurs déterminants soient compris et reconnus en raison de la régulation accrue des cours d'eau et par suite du nombre des localités riveraines. L'auteur se propose d'illustrer le besoin de collaboration pour assurer l'exécution d'analyses utiles à partir de la collection de données. La gestion du milieu exige maintenant des corrélations qualitatives plutôt que des incohérences. L'étude est un important signe avant-coureur.

T. Milne Dick, chef
Division de l'hydraulique
Le 30 mai 1983

INTRODUCTION

A major consequence of ice cover formation in rivers is the jamming that occurs during the spring breakup of the cover and clearance of the ice from the river. Due to their large thickness and hydraulic resistance, jams can cause unusually high water stages. This has repercussions in many operational and design problems of which spring flooding is the most pressing. However, the present capability for engineering predictions related to breakup and jamming problems is very limited. Only crude estimates of jam stage are possible and only where it can be assumed that a floating equilibrium jam has formed nearby. Clearly, such information is hardly adequate for satisfactory consideration of practical questions such as short term forecasting of the onset and severity of breakup; assessment of flooding frequency; and flood risk mapping.

A conceptual model of the breakup process is developed and discussed herein as a means of addressing short term forecasting problems.

BACKGROUND INFORMATION

Shulyakovskii (1963) and Deslauriers (1968) have given excellent qualitative descriptions of the breakup process in rivers while the former author proposed the following functional relationship for the onset of breakup.

$$[1] \qquad \Sigma q = f(h_i, h_s, \Phi, V, H_o, H', \Sigma q_i)$$

in which Σq = total heat input per unit outer surface of the ice cover that is necessary to initiate breakup; h_i and h_s = ice thickness and snow depth respectively, prior to the beginning of melting; Φ = (a set of) parameters describing stream morphology; V = average flow velocity; H_o = initial water stage; H' = rise in water stage above H_o; and Σq_i = total heat input per unit inner surface of the cover. Equation 1 is useful as a compact statement of the problem, but involves too many parameters to permit empirical assessment. For practical purposes, Eq. 1 is simplified by restricting attention to site-specific studies (Shulyakovskii 1963). This approach provides useful results but can only be applied to sites with good historical records. Since publication of Shulyakovskii's book (1963), the state of the art has not advanced appreciably (see, for example, Murakami 1972 and Galbraith 1981). The forecasting indices used are empirical and often change from site to site. Clearly, substantial improvement can only be achieved by development of a general conceptual model of the breakup process which would lead to the quantitative relationship envisaged in Eq. 1.

A possible starting point is the following quotation (Shulyakovskii 1963):

> "If the ice breakup develops during a rise in the water level, the stage "(H_B)" at which the ice push occurs is determined by the highest position of the ice cover during the winter, i.e., by the maximum winter stage"(H_F)

This suggestion will be discussed later and developed further. For the present, it is noted that use of H_B as an index of breakup initiation in conjunction with H_F, not only appeals to intuition but has also been found satisfactory by the writer on several occasions. The following quotation is also pertinent (Gerard, 1979):

> "...the only quantitative indication of the circumstances required to cause breakup of a reasonably competent ice cover is an increase in water level to near that which existed just after freeze up the previous fall. Beyond this the moment of breakup is difficult to anticipate ..."

In a later work, Shulyakovskii (1972) presented a theoretical model of breakup initiation which sheds more light to the significance of H_B, as outlined next. The main force responsible for stresses in the ice cover is identified as the flow shear stress on the cover's underside[1]. The cover is assumed to be separated from the river banks by a distance dictated by the difference $H - H_F$ (H = prevailing water stage, say gauge height; H_F = maximum stable stage during the preceeding freeze up - see also later discussion). The river is assumed to consist of linear segments intersecting at known angles (Fig. 1). Normal and bending stresses develop as illustrated in Fig. 1. Breakup initiation is defined as the instant when the strength of the cover is exceeded and transverse cracks form. By a simple structural analysis, confined to a plane parallel to the water surface, it is shown that breakup starts when

$$[2] \quad \sigma_i h_i = f_1(H_F, H_B)$$

in which σ_i = representative value of ice strength; H_B = stage at breakup initiation; and f_1 = a function of which the mathematical form depends on channel geometry, flow velocity and friction characteristics. If σ_i and h_i do not change appreciably from year to year, Eq. 2 reduces to

$$[3] \quad H_B = f_2(H_F)$$

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[1] This should be increased by the streamwise component of the cover's own weight per unit area. Total driving force per unit area = τ.

which is in agreement with earlier findings of the same author (1963).

A MODEL OF RIVER ICE BREAKUP

A conceptual model of breakup is developed in this section based on existing understanding while introducing a few new conditions.

Significance of H_F

The significance of the maximum stable freeze up stage, H_F, is illustrated in Fig. 2. It is this stage at which the width of the cover forms and can thus be approximated by the corresponding channel width, W_F. To eliminate very brief maxima for which there is little time for freezing, H_F is defined as a daily average value. So long as H remains less than H_F, the cover is supported by the channel banks. Under this boundary condition, the driving forces can be shown to produce very small stresses, not sufficient to break the cover. When warm weather and runoff start, a sufficiently high wave may travel downstream so that $H>H_F$ upstream of A (Fig. 2c) and $H<H_F$ downstream. Upstream of A, the cover may be considered a beam cantilevered at A. With the passage of time, A moves downstream and the stresses in the cover increase leading to formation of transverse cracks.

The foregoing suggests that a stage in excess of H_F is a necessary condition for breakup. This is valid so long as the cover remains competent in thickness, width and strength during the pre-breakup period. Though this occurs often in nature, there are instances of warm weather accompanied with insignificant runoff. The cover then deteriorates by thermal effects until it can either be broken by the available driving forces or slowly disintegrate in place. Deterioration may consist of full or partial loss of strength; and reduction in ice thickness and width. This is the "overmature" breakup type (Deslauriers, 1968), known to have little, if any, damage potential. Another complication is the commonly observed formation of side cracks, usually preceeding that of transverse ones. Side cracks are caused by uplift pressures which develop to accommodate the increased discharge. Side cracking is, to a degree, predictable and reduces the effective width of the cover. For simplicity, this effect will be temporarily ignored until certain basic relationships are demonstrated.

Description of the breakup process

Returning to the main discussion where it was seen how the first transverse cracks may be expected to form, it is noted that new cracks will appear as the flood wave advances. Eventually, a given river reach will be covered by large separate ice sheets. However, general breakup does not follow from this phase because the sheets may be too large to advance for any significant distance; they may be simply re-aligned into a "loose" but stable arrangement, as shown in Fig. 3 (see also later discussion). As the stage continues to increase, the water surface width increases until some of the sheets can "clear" the bends or other obstacles and move for a substantial distance. These sheets pick up speed and impact with stationary ones or with the channel boundaries which causes fragmentation. Small jams begin to form, causing additional stage increases, new dislodgements and so on, until the entire reach is cleared of ice. Based on this discussion, it is felt reasonable to define breakup initiation at a given site as the instant when a sustained movement of the cover takes place. This definition has the additional advantage of dealing with an easily observed event relative to transverse crack formation.

As breakup progresses, the destruction of the ice cover is accelerated by an increasing number of impacts and by thermal effects. The reach of interest will be cleared of ice when the sheet that holds the last ice jam is finally dislodged.

Dimensional analysis

The foregoing can be quantified as follows. Let ℓ_i be a length representative of the longitudinal dimensions of the separate ice sheets illustrated in Fig. 3b. Breakup starts when the water surface width, W_B, is such that it "just" permits a sufficient number of ice sheets to clear the various obstructions. (Clearly, ℓ_i will have a statistical distribution in a given reach rather than be a constant. The concept may be made more precise by stipulating that W_B is such that a fixed, though unknown, percentage of ice sheets are able to move. Then ℓ_i will be the length corresponding to this percentage). One could now write:

$$[4] \quad W_B = f_3(W_i, \ell_i; L_1, \ldots, L_K; \theta_1, \ldots, \theta_n)$$

in which W_i = ice cover width; and L_K, θ_n = lengths and angles that define river plan geometry. By dimensional reasoning, Eq. 4 can be reduced to

$$[5] \quad W_B/W_i = f_4(\ell_i/W_i; \ldots L_K/W_i; \ldots \theta_n)$$

Figure 4 shows two examples for which Eq. 5 can be quantified. A curved sheet of average radius, R, and central angle, θ, will "clear" a straight reach when (Fig. 4b)

$$[6] \quad \frac{W_B}{W_i} = 1 + \left(\frac{R}{W_i} - \frac{1}{2}\right)\left(1 - \cos\frac{\theta}{2}\right)$$

It is noted that $\theta = \ell_i/R$ and Eq. 6 applies for $\theta<\pi$. The length ℓ_i can be expressed as

$$[7] \quad \ell_i = f_5(\tau, \sigma_i, W_i, h_i, \ldots L_K; \ldots \theta_n)$$

in which τ = driving force per unit area. Eq. 7 implicitly assumes that the mechanism of transverse crack formation is as shown in Fig. 1. If another mechanism is assumed (e.g., approach of a steep water wave), some additional ice and water properties will be needed on the RHS of Eq. 7, as will be discussed later. However, this consideration does not alter the essence of what follows. Equation 7 can be non-dimensionalized and substituted in Eq. 5 to obtain

$$[8] \quad W_B/W_i = f_6(h_i/W_i, \sigma_i/\tau; \ldots L_K/W_i; \ldots \theta_n)$$

Based on previous discussion, it can be assumed that $W_i \simeq W_F$, provided the cover has not been subjected to significant side cracking and melting. Because W_F varies little from year to year (freeze up flows), the parameters L_K/W_i could be considered river constants as a first approximation. Moreover, in most natural streams, W varies as a power of Y (=average depth) so that W_B/W_i can be replaced by the more practical parameter Y_B/Y_F. With these assumptions, Eq. 8 reduces to

$$[9] \qquad Y_B/Y_F = f_7(h_i/W_F, \sigma_i/\tau; \text{ dimensionless river constants})$$

From the physical understanding described so far, it is reasonable to expect that the function f_7 increases with both h_i/W_F and σ_i/τ. The dimensionless river constants serve to account for the channel plan geometry. For example, the L_K's may be used to identify such dimensions as meander length and amplitude, radii of curvature of bends, lengths and widths of islands, etc., while the θ_n's identify typical bend angles.

The second major problem associated with breakup is how to forecast its severity which can be partly quantified by H_m, the maximum breakup stage. It was mentioned earlier that a reach will be cleared of ice when the ice sheet that is least amenable to dislodgement, is eventually lifted to a level at which it can advance. Letting Y_c be the average flow depth at this level, one could write, as before

$$[10] \qquad Y_c/Y_F \leq f_8(h_i/W_F, \sigma_i/\tau; \quad \text{dimensionless river constants})$$

The inequality symbol has been used because the last ice sheet to move will very likely be subjected to reductions in competence and dimensions during the breakup period. The depth, Y_c, can be expressed in terms of the corresponding discharge intensity, q_c, via a resistance formula and substituted in Eq. 10 to obtain

$$[11] \qquad \frac{(q_c^2/gS)^{1/3}}{Y_F} + \frac{1.59}{f_c^{1/3}} \left(\frac{s_i h_i}{Y_F}\right) \leq \frac{1.59}{f_c^{1/3}} \quad f_8\left(\frac{h_i}{W_F}, \ldots\right)$$

in which g = acceleration of gravity; S = channel slope; s_i = specific gravity of ice; and f_c = composite friction factor of the flow under the ice sheet. Equation 11 implies that there exists an "ice clearing" discharge, of which the upper limit depends on H_F, h_i, σ_i, τ and on channel geomorphology. In all probability, the last ice sheet to move, will be holding back an ice jam whose potential stage can be estimated in terms of q_c and channel hydraulics (Beltaos 1982a; Pariset et al 1966). This places an upper limit on H_m, independent of discharge but dictated by H_F, h_i, σ_i, τ as well as channel geomorphic characteristics.

To test the foregoing results, data from several sources have been used, as described next.

DESCRIPTION OF DATA

Apart from relatively few data obtained by direct observation, the major data sources have been Water Survey of Canada records at hydrometric gauge sites (see Table 1). These include stage records, discharge measurement notes and local observers' reports on ice conditions. Supplementary information consists of meteorological data and channel hydraulics obtained by hydrometric surveys in the gauged reaches. From these data, several parameters have been extracted, as described below.

Maximum stable freeze up stage, H_F: A typical, though not universal, configuration of the daily average stage hydrograph at the start of the ice season is sketched in Fig. 5. While the effective stage (= stage that would have occurred had the flow been unaffected by ice) decreases continuously, the actual stage is seen to first rise, reach a peak and then decrease again. The rise is caused by the upstream advance of the ice cover, formed by jamming of slush pancakes at some point downstream of the gauge. Once the ice cover edge arrives at the gauge site, the stage begins to drop owing to decreasing discharge and thermal smoothing of the underside of the cover.

"Winter" peaks: occasionally, a brief thaw may occur during the winter, resulting in a peak on the stage hydrograph. If this peak does not initiate a breakup, it can be considered a lower limit for H_B at that time.

Stage at initiation of breakup, H_B: When a thaw does lead to breakup, the stage hydrograph shows irregularities that cannot be explained by discharge variations, as illustrated in Fig. 6. A probable value for H_B may be fixed at the first significant spike or slowdown in the rate of stage rise. (It may be recalled here that breakup initiation has been defined as the instant when a sustained movement of the ice cover begins. When a stationary ice sheet is set in motion, the stage would tend to drop as a result of reduced resistance to flow. This effect may be masked, however, by simultaneous rapid increases in discharge.) However, this definition is not always objective or meaningful. Only a probable range of H_B can then be determined by considering (a) the latest time for which it can be confidently assumed that there still was continuous ice cover; and (b) the earliest time for which broken ice effects became evident. Simultaneous consideration of local observers' reports and familiarity with local conditions greatly increase the accuracy of H_B determinations.

Maximum breakup stage, H_m: Determination of H_m is straight forward (Fig. 6).

Discharge, Q: Daily average discharges are estimated by Water Survey of Canada, based on interpolations between measurements and on such evidence as upstream and tributary flows, runoff and weather conditions, etc. Such estimates may involve large errors during breakup, except where discharge measurements have been performed, as is the case for most of the years of record at the Thames R. gauge. For the other gauges in Table 1, discharge values used herein must be viewed as, at best, crude.

Ice thickness, h_i: ice thickness can be approximately determined from discharge measurement notes, subject to certain limitations (Beltaos and Lane 1982). For years without measurements, h_i can be estimated from site-specific correlations between measured values and time since the date of H_F. Such values of h_i are designated herein as "estimated" and involve errors of up to ±30%.

MODEL TESTING - BASIC VARIABLES

In this section, the dimensionless relationships derived earlier are tested in a preliminary manner, i.e. by ignoring the effects of σ_i, τ and side cracking which will be discussed in the next section. Figure 7 shows a trend for H_B to increase with H_F. The effect of h_i is considered in Fig. 8 where a trend for H_B-H_F to increase with h_i is exhibited. Also shown in Fig. 8 are data for the Smoky and Peace Rivers, plotted as ranges due to limited variability. The latter do not fit the Thames River relationship. Figure 9 shows the same data sets plotted in the dimensionless form suggested by Eq. 9. Despite the scatter, the anticipated trend is confirmed. More importantly, the Smoky and Peace River data are now much more consistent with the Thames River data than they were in Fig. 8. The scatter in Fig. 9 can be partly attributed to variations of σ_i/τ which, however, are unknown because neither σ_i nor τ values are available. It is also noted that the the present findings can explain the empirical results of Murakami (1972) (Beltaos 1982b). A similar analysis for two rivers in the United States resulted in a set of data points consistent with those shown in Fig. 9 (D. Calkins, personal communication); the range of 100 h_i/W_F for this set was from 0.4 to 2.0.

To test the predicted existence of the "ice clearing" discharge, the maximum discharge Q_m, attained during the breakup period has been used. To determine the corresponding discharge intensity, q_m, the water surface width downstream of the last ice jam just prior to its release is needed. This is unknown for the present data but setting $q_m \approx Q_m/W_B$ is considered a fair approximation. For the Thames, Peace and Smoky Rivers, h_i is small relative to Y_c, hence the second term on the LHS of Eq. 11 has been neglected. With these assumptions, data for these three rivers are plotted in Fig. 10, in the form suggested by Eq. 11. The data points define an upper envelope that exhibits the anticipated dependency on h_i/W_F. From earlier discussion, it is reasonable to expect that the function f_8 in Eq. 10 takes a value of 1.0 when h_i/W_F vanishes. Inspection of Eq. 11 and Fig. 10 suggests further that the intercept ($\approx$4.0) at

$h_i/W_F = 0$ should be equal to $1.59/f_c{}^{1/3}$. This gives $f_c \approx 0.063$ which is a plausible friction factor value for a channel covered with sheet ice at the time of breakup (corresponding Manning coefficient ≈0.032). Additional support for the ice-clearing discharge concept is provided by Figs. 11 and 12 where H_m is seen to be influenced by both H_F and h_i in the expected manner. The scatter is thought to be due to (a) whether or not the ice clearing discharge is realized in any one breakup event; and (b) as yet unknown effects of thermal and mechanical deterioration. At a given site, a graph such as Fig. 10 could be utilized as follows: Let q_p = peak discharge intensity, forecast for a runoff event expected to cause breakup; and q_c = value obtained from the upper envelope in Fig. 10. If $q_p \leq q_c$, then H_m should not exceed the potential jam stage for $q = q_p$. However, if $q_p > q_c$, H_m should not exceed the jam stage for $q = q_c$.

MODEL TESTING-OTHER EFFECTS

A preliminary comparison of data with the present model has produced encouraging results. However, there remain several questions that need addressing. For example, what is the effect of side cracks? Is the pre-breakup pattern postulated in Fig. 3 realistic? What are the effects of σ_i and τ? Is the model a general one or just one of several different break-up processes? These questions are considered in this section.

Effects of side cracks

A floating cover attached to the channel banks and subjected to uplift pressures may be considered a beam on an elastic foundation (Billfalk 1981). Using the appropriate structural theory (Hetenyi 1946) it is possible to predict the uplift pressure head (ΔH) required to cause side cracking and the locations of the side cracks. Billfalk (1981) performed this calculation for infinitely wide channels and showed good agreement with measurements. The type of support assumed for the ice edges has a large effect on ℓ_s(=distance of side cracks from respective edges). For fixed ends, $\ell_s = 0$. For hinged ends, it was found that

[12] $$\lambda \ell_s = \pi/4 \text{ (infinitely wide channel)}$$

in which λ is defined as

[13] $$\lambda \equiv \sqrt[4]{\gamma/4E_i I}$$

with γ = unit weight of water; E_i = elastic modulus of ice; and I = moment of inertia per unit cover width = $h_i^3/12$.

Billfalk's analysis has been extended to the finite width case (Beltaos and Wong, unpublished data) and the results are shown in Figs. 13 and 14. For hinged ends, the two side cracks merge into a single central crack for $\lambda W \leq 3$, while Eq. 13 applies for $\lambda W \geq 6$. Figure 13 affords a means of applying a correction to W_F in order to determine W_i. To match observed with predicted ℓ_s's for the Thames River, a value of $E_i = 1.4$ GPa was found appropriate. This is considerably less than 6.8 GPa, representing good-quality, freshwater ice (Gold 1971). The difference could be due to thermal deterioration and creep effects; it is much less pronounced when comparing predicted (with 6.8 GPa) and observed ℓ_s's because ℓ_s varies as the fourth-root of E_i. As an illustration, let $h_i = 0.5$ m. Then $\lambda = 0.113\ m^{-1}$ and Fig. 13 shows that, for $W \leq 26.5$ m, there will only be one hinge crack. For $W \geq 53$ m, two hinge cracks will form, each located 7.0 m from the respective ice edge (Eq. 13). For the latter case, Fig. 14 shows that $\gamma \Delta H/\sigma_i(\lambda h_i)^2 = 1.03$; with $\sigma_i = 600$ kPa, this gives $\Delta H = 0.2$ m. [Note: σ_i is now the flexural ice strength.]

To illustrate the effect of the side crack correction, W_B/W_F and W_B/W_i (W_i = corrected width) are respectively plotted versus h_i/W_F and h_i/W_i in Fig. 15 (see Eq. 8). A reduction in scatter seems to be effected by this correction. Figure 16, comprising the Nashwaak River data (see Table 1) provides a more striking illustration. Whereas the uncorrected width plot shows no trend (Fig. 16a), the width correction effects a clear increasing trend (Fig. 16b), similar to that found in Fig. 15. At the same time, it may be observed that the Nashwaak River data exhibit more scatter and a smaller rate of increase with h_i/W_i than the Thames River data. These differences will be discussed later.

Observed transverse crack patterns

Because the possible significance of transverse cracks was only recently understood, only one documentation of their spacing and location can be furnished herein, as shown in Fig. 17. The centre of the reach shown in this figure is located some 25 km downstream of the Thamesville gauge site. Crack locations are approximate because they were viewed from a height of 400 m and drawn on a 1:50,000 map. Nevertheless, Fig. 17 shows a fairly consistent crack spacing, reminiscent of the conditions postulated in Fig. 3b.

For the reach shown in Fig. 17, it is estimated that W_i = 55 m and h_i = 0.35 m. Therefore, 100 $h_i/W_i \simeq 0.64$ which, from Fig. 15, gives $W_B/W_i \simeq 1.46$. The photos of Fig. 17 indicate that the water surface to ice cover - width ratio was less than 1.46 which agrees with the fact that breakup had not yet been initiated.

A frequency analysis[2] indicated that the average ice

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[2] After transferring the observed crack locations to two-fold enlargements of 1:10,000 vertical air photos.

sheet length in Fig. 17 was 300 m while the 16- to 84-percentile values were 225 m and 415 m, respectively. Corresponding values of ℓ_i/W_i are 5.5; and 4.1 to 7.5. These can be shown to be comparable to what is implied by earlier findings, as follows. Using Eq. 6 as a rough guide and putting W_B/W_i = 1.46, one can solve for ℓ_i/W_i, provided R/W_i is known. For the sheets of Fig. 17, it was found that R/W_i = 6.2 (average); and 3.2 to 15.6 (16- to 84 -percentiles). With these values, Eq. 6 gives ℓ_i/W_i = 5.0 (average); and 3.8 to 7.7 (16- to 84 -percentiles).

Mechanism of transverse crack formation

Shulyakovskii's (1972) postulated mechanism has been illustrated in Fig. 1 where it was assumed that the river comprises linear segments of uniform width. A slight improvement that avoids this "linearization" resulted in (see Fig. 18)

$$[14] \quad M = 2\tau W_i a_M$$

in which M = bending moment at C; and a_M = area of segment ABC. A transverse crack will form at C when $6M/h_i W_i^2$ becomes equal to σ_i(=flexural ice strength). By virtue of Eq. 14, this condition leads to

$$[15] \quad a_M = \sigma_i h_i W_i / 12\tau$$

Equations 14 and 15 involve the following assumptions: (a) ice cover curvature effects on the stress distribution at C are negligible which is a good approximation for mild curvature (Flugge 1962); (b) the elementary force $\tau W_i ds$ (Fig. 18) acts on the centreline of the cover which, too, is a good approximation for mild curvature; (c) contributions to the stresses at C by normal forces are negligible which is estimated to apply in most instances; and (d) moments caused by forces that may be transmitted between adjacent sheets, (e.g, at A in Fig. 18) are ignored (see also later discussion).

Equation 15 shows that a_M and thence ℓ_i must vary along a given reach owing to changing planform geometry and ever-present variations in σ_i, h_i, W_i and τ. For the ice sheets shown in Fig.17, the following average values have been estimated: W_i = 55 m, h_i= 0.35 m, τ = 5.0 Pa and a_M = 6400 m^2. Substituting these in Eq. 15 gives σ_i = 20 kPa which is very low relative to 600 kPa, a common flexural strength value for good-quality ice as measured by the well-known cantilever or simply supported beam tests (Frankenstein 1961; Korzhavin 1971; Butyagin 1972). This large discrepancy is moderated by the following considerations:

(a) The value of σ_i has been found to decrease with specimen size. Using empirical results (Butyagin 1972), a reduction factor of at least three was estimated for the flexural strength of the entire ice cover cross-section, relative to that obtained from beam tests. This would bring σ_i up to at least 60 kPa which is still low but close to the lower limit of the range of σ_i's measured near the time of breakup (≈100 kPa - Frankenstein 1961).

(b) Equation 15 ignores stresses caused by forces that may be transferred at existing cracks. It is difficult to assess this effect because it depends on the (unknown) configuration of lateral restraints imposed by the channel boundaries on upstream ice sheets. It is estimated that, in the absence of restraints, this effect could cause a two- to three-fold increase of the calculated σ_i.

(c) Creep effects that reduce the apparent ice strength have been ignored. The writer is not aware of creep data pertaining to the loading configuration at hand. For vertical loadings of the ice cover, creep reduces the apparent ice strength by 50% within a few hours of loading time (Assur 1961; Panfilov 1972).

It thus appears that Shulyakovskii's mechanism of transverse crack formation may apply but more data are needed for a definite conclusion on its validity. Another mechanism that can produce transverse cracks is the passage of water waves under the cover. Such waves can be caused by sudden releases of ice jams or rapid discharge increases. A first attempt to analyze this problem was made by Billfalk (1982) who assumed a linear water surface profile and ignored dynamic effects based on an order-of-magnitude comparison with static ones. This theory predicts crack spacings that are far too small relative to observations (Fig. 17) but the wave breaking theory needs further development before deciding on its applicability. For the present, it is noted that wave breaking would produce ℓ_i's that are largely independent of W_i and τ but dependent on such additional parameters as initial water surface configuration, wave celerity, E_i and γ.

<u>Effects of ice deterioration and driving forces</u>

Inspection of Figs. 15 and 16 suggests the following empirical equation

$$[16] \quad W_B/W_i = 1 + C(100h_i/W_i)$$

According to Eq. 8, C should depend on σ_i/τ and dimensionless river constants. The former parameter was introduced via Shulyakovskii's (1972) mechanism of transverse crack formation. However, it was shown in the previous section that this mechanism cannot as yet be confidently accepted. Hence, σ_i/τ may be more appropriately replaced by several other dimensionless factors reflecting ice and flow properties. In general, C would be expected to decrease with increasing degree of thermal ice deterioration and possibly with increasing τ[3].

Few data on the mechanical properties of ice at the time of breakup are available and the process of thermal deterioration is not well understood at present (Frankenstein 1961; Korzhavin 1971; Butyagin 1972). Bulatov (1972) outlined a method for computing ice strength based on theoretical and experimental correlations with radiation effects; however, this paper is too general to permit application of the proposed method by others. Thus, thermal effects can only be studied at present by introducing empirical indices intended to describe weather conditions. For the Nashwaak River data, preliminary analysis indicated that both accumulated degree - days of thaw and hours of sunshine influenced the onset of breakup. To reduce the number of thermal indices as well as introduce the incoming solar radiation, a single index, Σq, was also tried and showed equal effectiveness as that of the combination of degree-days and sunshine. Σq is a calculated heat input per unit ice cover area, accumulated to the time of breakup initiation (see Shulyakovskii 1963 and Beltaos and Lane 1982 for computational details). For the Nashwaak River data, it was

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[3] The wave breaking mechanism, for example, should be largely independent of τ. At the same time, it is difficult to conceive instances where C would increase with increasing τ.

subsequently found that C decreases with increasing Σq, in accordance with expectation. Pertinent meteorological data were obtained from published records (Atmospheric Environment - "Monthly Records") for a nearby station and are thus considered representative of local weather conditions. "Premature" events had a value of 0.45 for C_0 (= C for $\Sigma q \rightarrow 0$). Analysis of the Thames River data indicated a similar but not as well defined variation of C with Σq. However, there are no nearby sunshine recording stations so that Σq values are uncertain in this case. The value of C_0 was 0.85. For the Smoky and Peace River data, C_0 cannot be determined because no premature events have been observed; all that can be stated about C_0 is that it should not be less than 0.45 and 0.52 respectively. Inspection of Table 1 suggests a trend for C_0 to decrease with increasing river slope. This may be a hint for a similar dependency on τ but the latter also depends on other factors that are generally unknown for the present data.

In conclusion, it may be stated that the present results show the expected stratification with thermal ice deterioration via empirical meteorological indices. While use of any such index can be criticised on the grounds of not adequately representing the physical processes involved, the C_0 values quoted earlier are not dependent on the type of index used. For the four river sites considered herein, C_0 is between 0.45 and 0.85. This is remarkably consistent, considering the associated large variation in magnitude, hydraulics and latitude of the respective streams. The effects of driving forces and channel plan geometry remain unclear. Progress in this regard requires additional case studies and conclusive identification of the mechanism of transverse crack formation.

Limitations

The foregoing discussion suggests several conditions under which the present model may not apply, ie:

(i) "Over mature" breakup events during which the ice cover largely disintegrates by thermal effects rather than breaking by mechanical action.

(ii) Reaches where the water level is strongly influenced by nearby controls. An example is the Thames River near the mouth where the stage is controlled by that of Lake St. Clair. In this reach, the stage hardly rises prior to breakup initiation so that the transverse crack and loose ice sheet pattern may not occur. Breakup is usually initiated by thermal deterioration and mechanical destruction effected by advancing ice jams.

(iii) Channel types that are significantly different from the single, meandering channels considered herein, e.g., braided; multiple islands; straight channels, etc.

DISCUSSION

A conceptual model of river ice breakup has been developed and used as a framework for analyzing pertinent data from four gauge sites. The analysis resulted in some encouraging findings but at the same time identified several gaps in existing observational information. Improved knowledge is needed for the following aspects of breakup: Ice cover thickness; discharge hydrograph; mechanism of thermal ice deterioration; ice cracking patterns prior to breakup initiation; and accumulation of additional case studies over representative ranges of river morphology and climate.

The present analysis focused on forecasting the onset and flooding potential of the breakup. Other things being equal, the breakup initiation stage increases with increasing freeze up stage, ice thickness and strength; and with decreasing channel width and slope. The flooding potential of breakup is largely governed by discharge which dictates the potential stage of any ice jams that might occur. The present model suggests that there should be an "ice-clearing" discharge such that larger discharges will be associated with unstable, if any, jams. This places an additional limitation on the flooding potential of breakup depending on freeze up stage, ice thickness and channel width and slope.

A major factor facilitating the onset and progress of breakup has been identified as the available water surface width in relation to the size of separate ice sheets that form by transverse cracking. In turn, this can suggest possible means of breakup flood control in addition to commonly used methods, e.g., keeping freeze up levels low or placing dykes some distance off the channel banks.

SUMMARY AND CONCLUSIONS

The breakup model developed herein provides a framework for interpreting and generalizing data pertaining to breakup forecasting which to date has been site-specific. The main factor facilitating the onset and progress of breakup has been identified as the available water surface width relative to the size of separate ice sheets formed by transverse cracking. Thus, it has been possible to quantify the effects of such factors as ice cover dimensions and (partly) channel geometry. Owing to lack of data, other parameters (e.g., mechanical properties of ice and driving forces) have only been considered indirectly to elucidate trends. The mechanism of transverse cracking was examined in the light of recent observations. Bending on planes parallel to the water surface, caused by stream curvature , could account for the observed crack spacing but more evidence is needed for positive conclusions. The present model does not apply in cases of "overmature" breakup events, proximity of stage controls and river planforms significantly different from the single meandering channel type.

ACKNOWLEDGMENTS

The work reported herein is mainly a part of a long-term research program on river ice jams and flooding, conducted by the Hydraulics Division, National Water Research Institute, Environment Canada. Valuable assistance in processing the Thames River records was provided by B. Poyser and H. McGarvey of Water Survey of Canada-Guelph. The Smoky and Peace River data were obtained by the writer as part of a monitoring program conducted by Alberta Research Council in cooperation with Alberta Environment and Transportation. W. J. Moody and H. Ng assisted with data processing. Review comments by T.M. Dick and Y.L. Lau are appreciated.

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Table 1. Summary of gauge site characteristics

Site Description	Type of Data	Latitude	Slope (m/km)	Long-Term Mean Conditions(1) Width (m)	Depth (m)	Discharge (m^3/s)
Thames R at Thamesville	Gauge records, 1960-79; Observations, 1980-82	42°32'42"N	0.23	37	2.0	49
Nashwaak R at Durham Bridge	Gauge records, 1965-1981	46°07'33"N	0.73	58	1.2	36
Peace R at Peace River	Observations, 1974-76, 1979	56°14'41"N	0.35	470	3.5	1800
Smoky R at Watino	Observations, 1976-79	55°42'56"N	0.52	225	1.9	370

Notes: (1) Width and depth values are for open-water conditions at the long-term mean discharge. For Peace and Smoky Rivers, data were obtained from Kellerhals et al (1972).

FIGURES

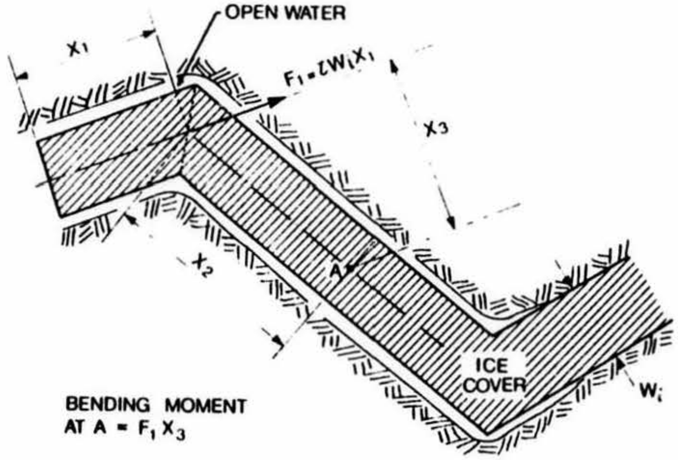


Fig.1. Shulyakovskii's mechanism of transverse crack formation.

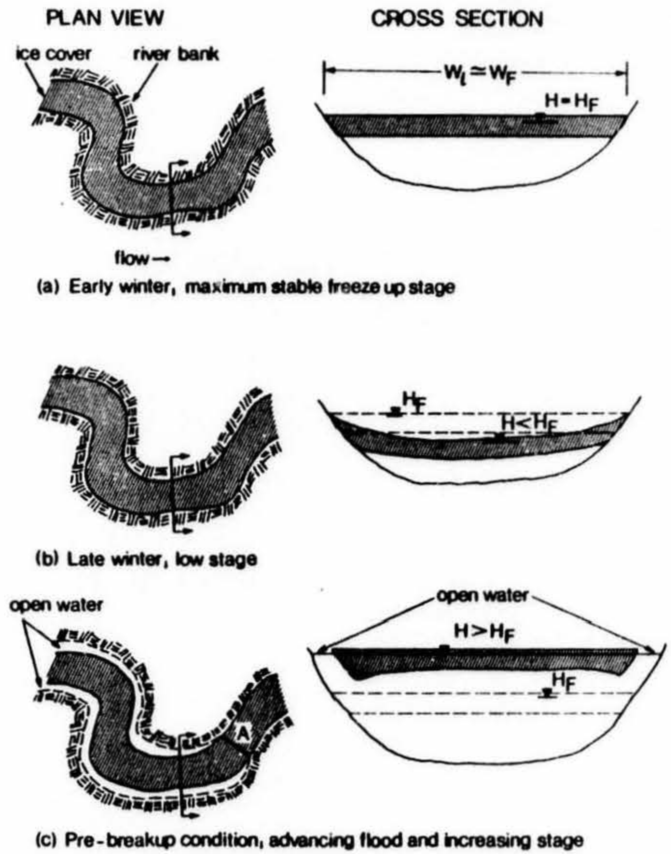
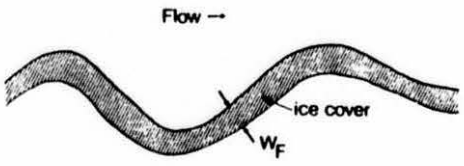


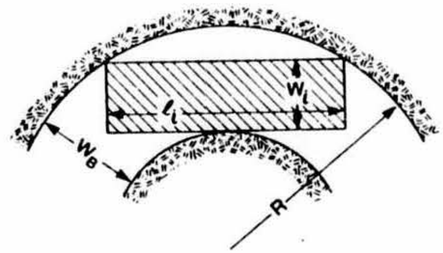
Fig. 2. Significance of H_F.



(a) $H \leq H_F$

(b) $H_F < H < H_B$

Fig.3. "Loose" arrangement of large ice sheets.



(a) straight sheet past circular bend

(b) curved sheet past straight reach

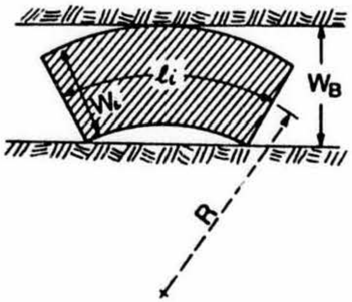


Fig. 4. Illustration of ice sheet movement threshold.

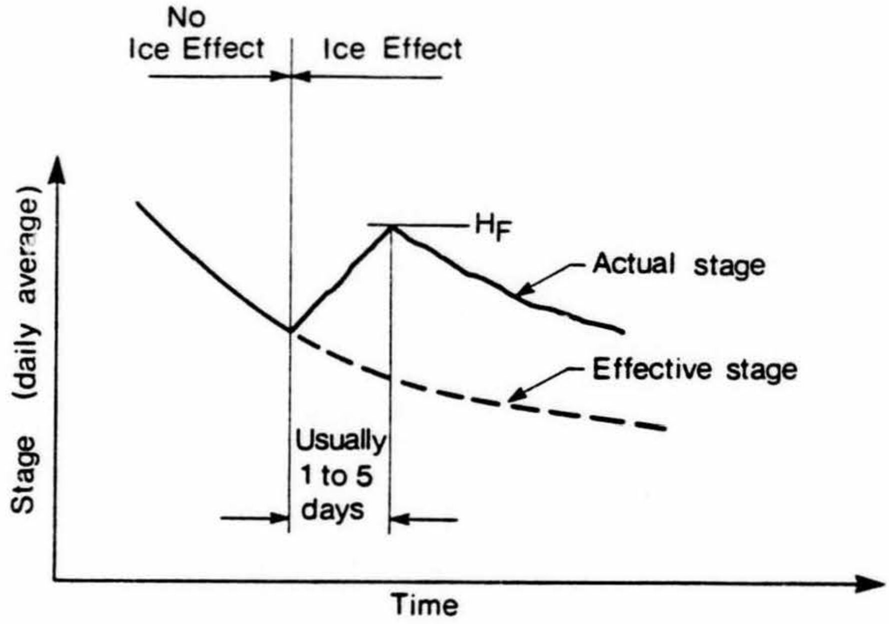


Fig. 5 Schematic illustration of daily stage variation with time during beginning of freeze up.

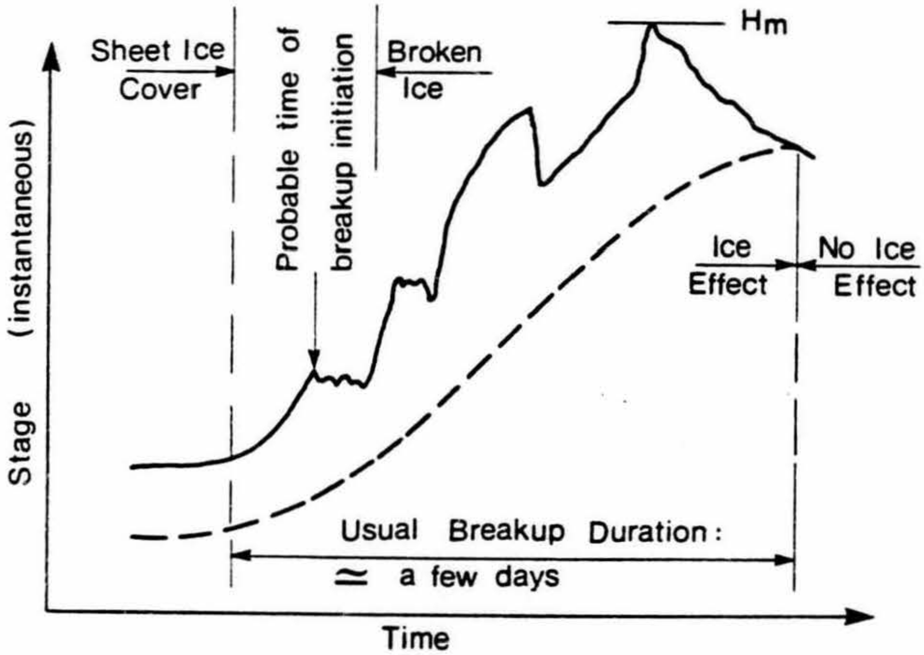
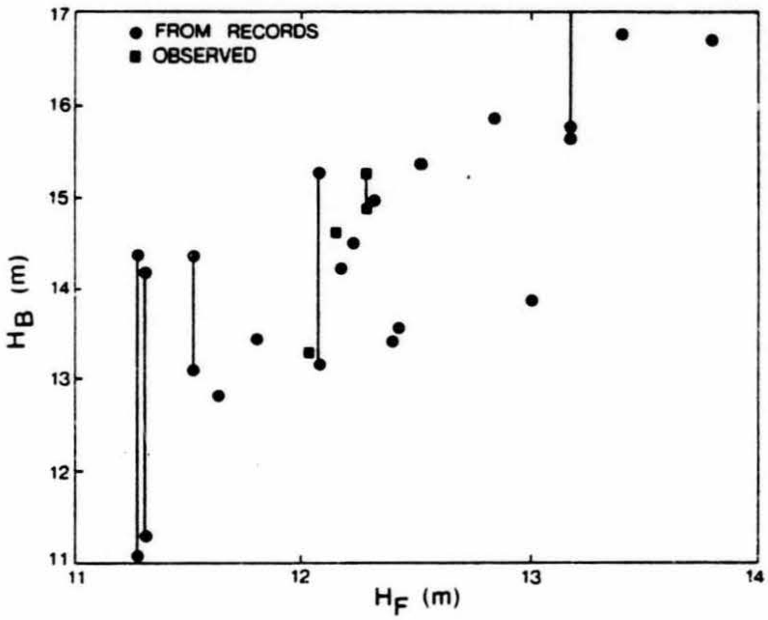


Fig. 6 Schematic illustration of instantaneous stage variation with time during breakup.

Fig. 7. Effect on H_F on H_B; Thames R. at Thamesville.



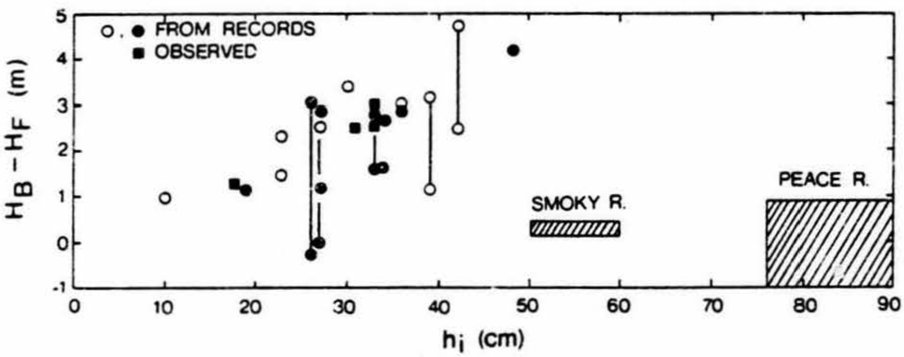


Fig. 8. Effect of ice thickness on H_B - H_F; data points are for Thames R. at Thamesville; open circles denote estimated h_i's.

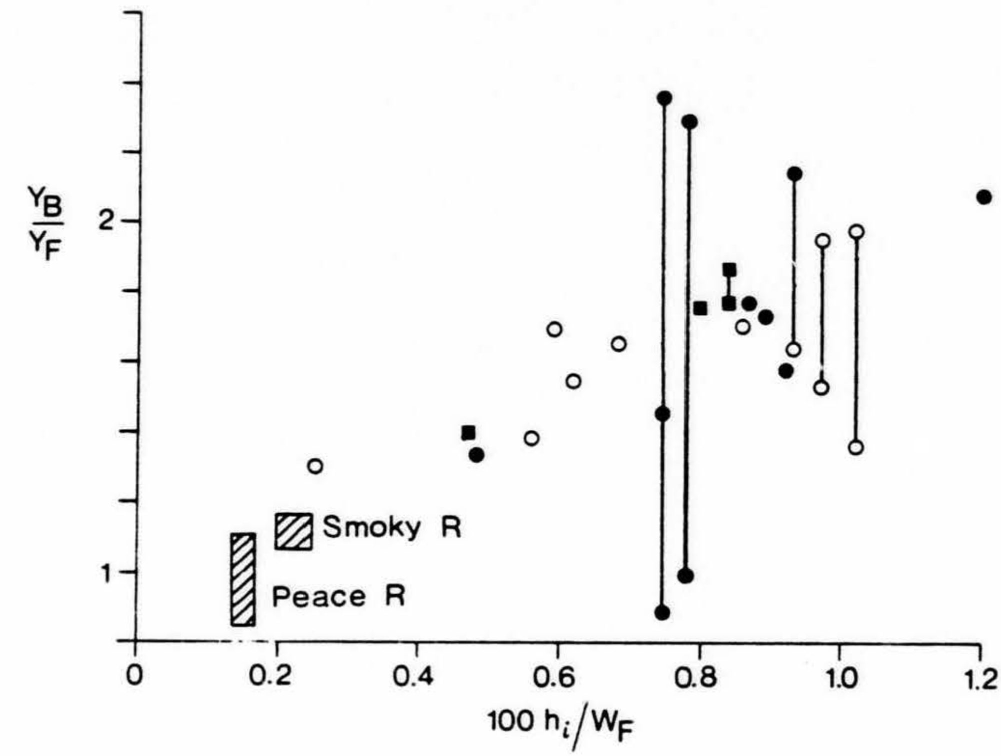


Fig. 9. Test of Eq. 9, Y_B/Y_F versus h_i/W_F; legend same as for Fig. 8.

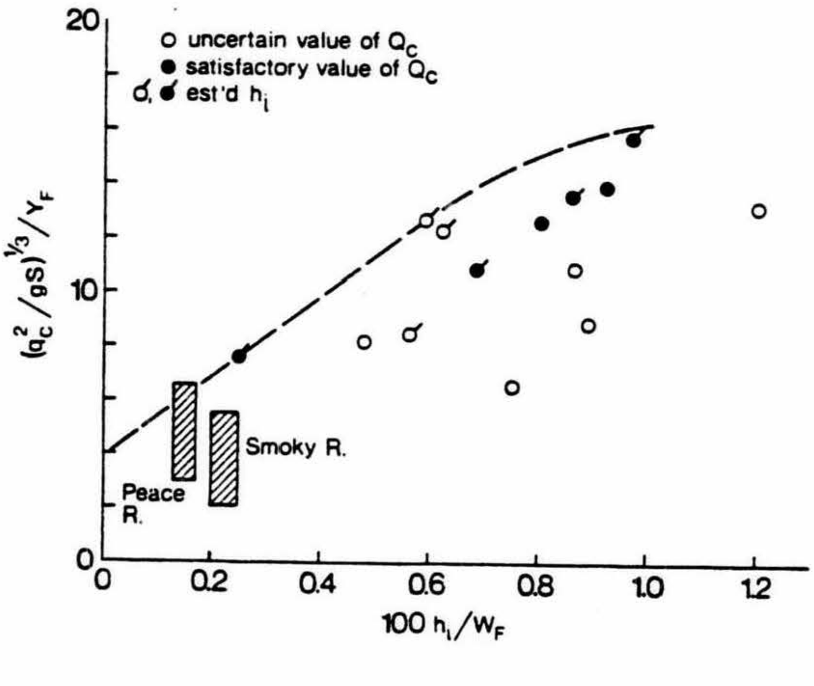
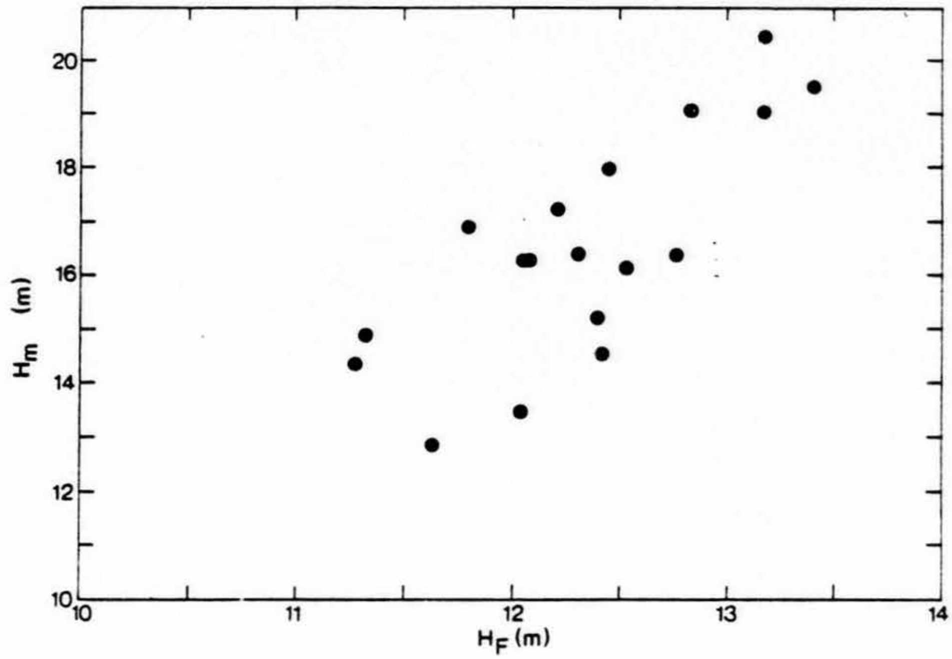


Fig. 10 Dimensionless "ice-clearing" discharge versus dimensionless ice thickness.

Fig. 11. Maximum breakup stage versus H_F; Thames R. at Thamesville.



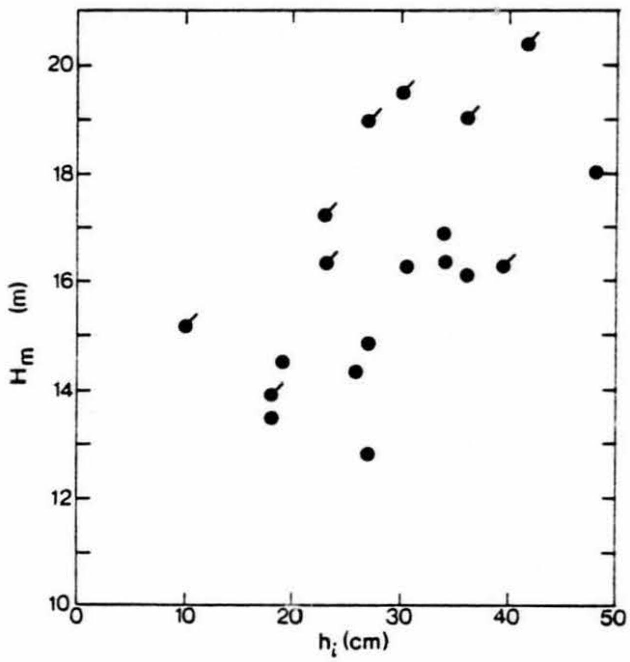


Fig. 12. Maximum breakup stage versus h_i; Thames R. at Thamesville.

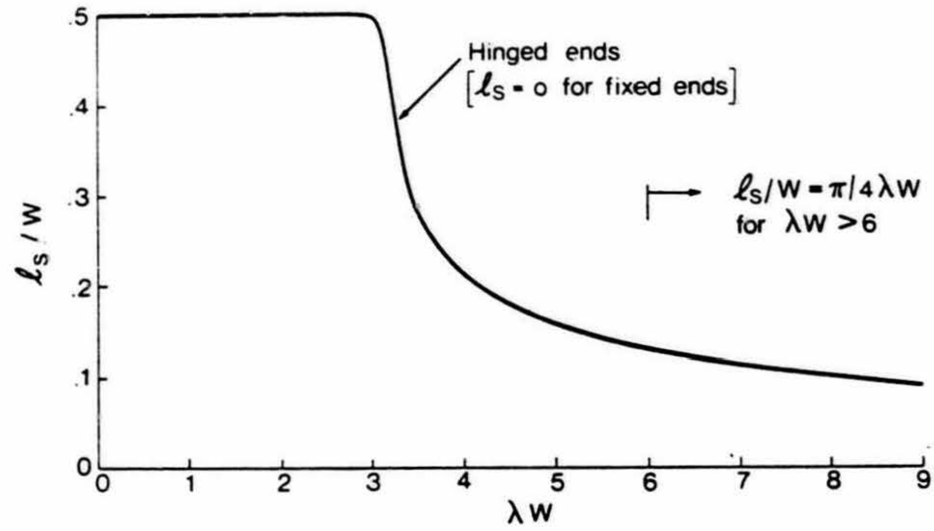


Fig. 13. Location of side cracks.

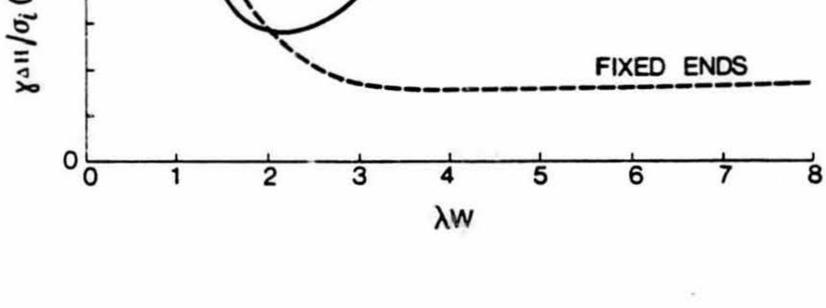


Fig. 14. Dimensionless uplift pressure head required to cause side cracking.

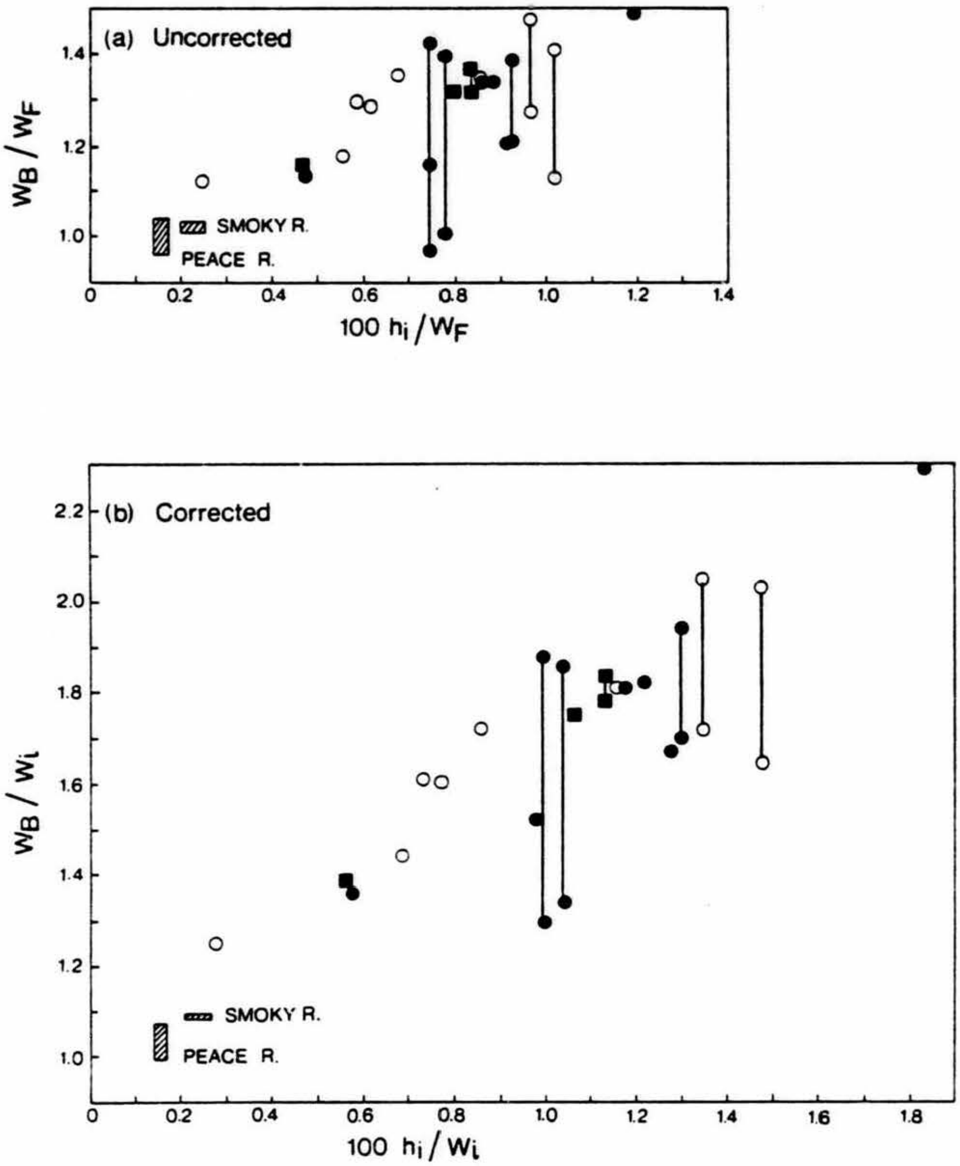


Fig. 15. Effect of side-crack correction; Thames, Peace and Smoky Rivers; legend same as for Fig. 8.

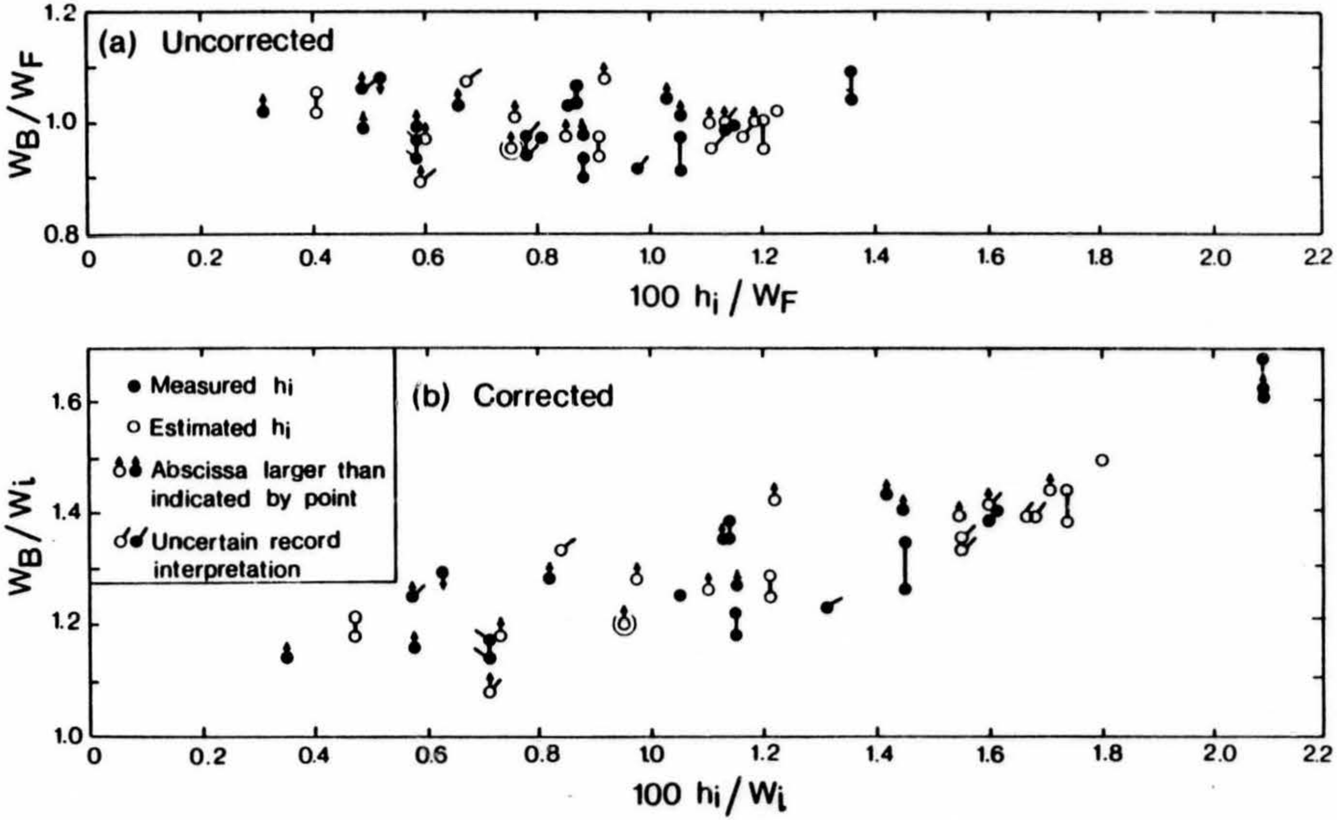


Fig. 16. Effect of side-crack correction; Nashwaak R.

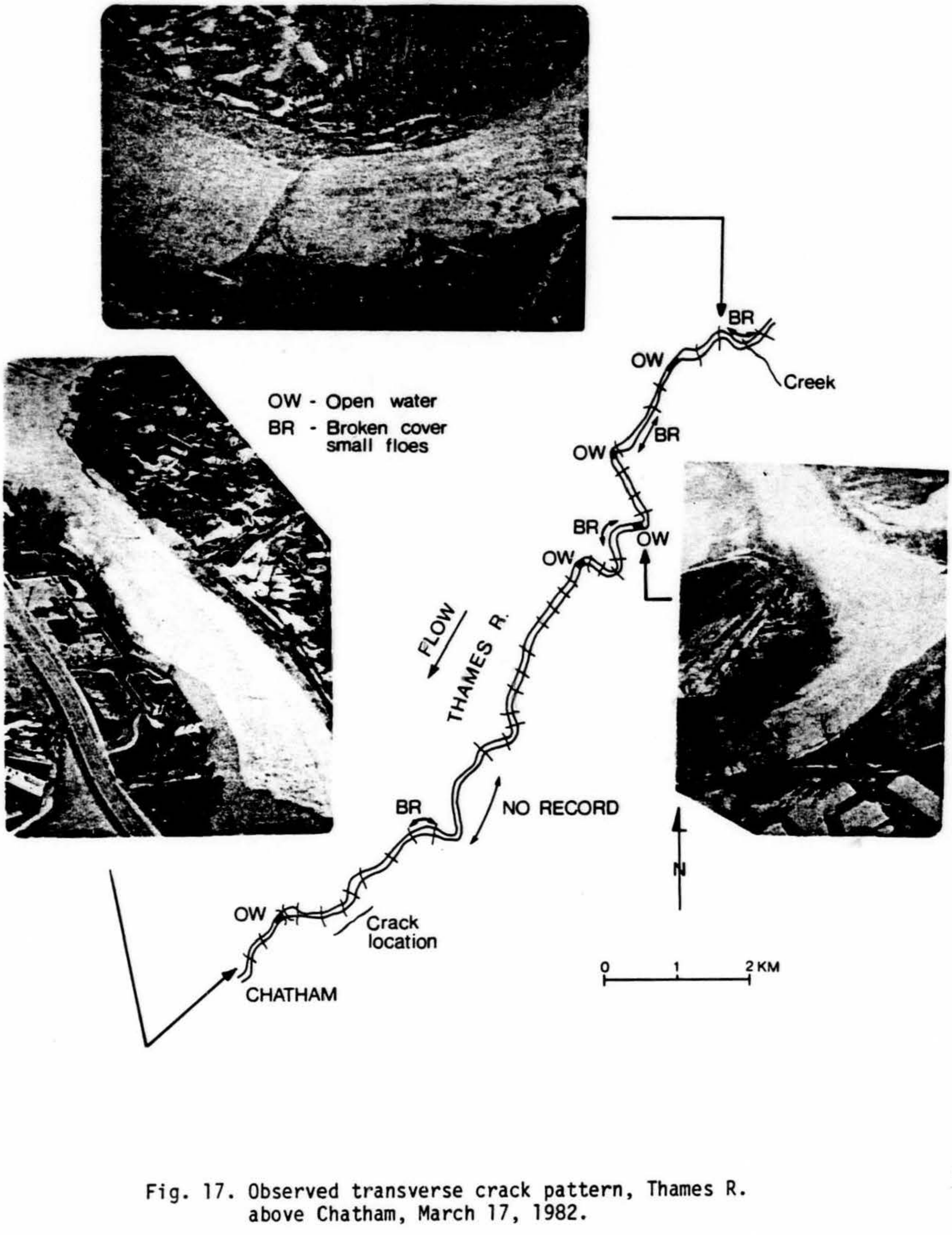
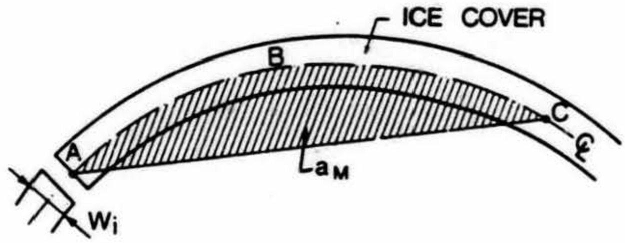
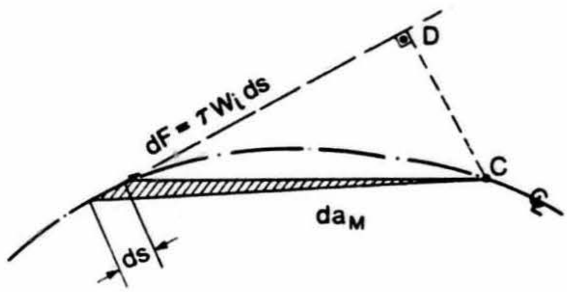


Fig. 17. Observed transverse crack pattern, Thames R. above Chatham, March 17, 1982.



(a) RIVER PLAN



$dM = (dF)(CD) = 2\tau W_i (da_M)$

(b) ELEMENTARY FORCE (dF) AND BENDING MOMENT (dM)

Fig. 18. Derivation of Eq. 14.

River Ice Jams: Theory, Case Studies, and Applications

By Spyridon Beltaos[1]

Abstract: The theory of river ice jams, as developed by several investigators over the past two decades, is reviewed and two methods for analyzing case studies are developed. The first method is based on a dimensionless equation that relates measurable ice jam characteristics. The second method compensates for the lack of measured thickness values for breakup jams by introducing a relationship between the hydraulic roughness and the thickness of a jam, based on data by others with respect to freeze up jams. These two methods of analysis are subsequently applied to several case studies performed in recent years by the writer and others. The results support the theory and at the same time show satisfactory consistency in the respective values of various coefficients obtained by the two methods. Procedures for practical application of the present results are outlined and assessed by means of examples.

Introduction

A major consequence of ice cover formation on northern rivers is the jamming that occurs during the spring breakup of the cover and, to a lesser degree, during the freeze-up period. Due to their large aggregate thickness and hydraulic resistance relative to those of sheet ice, jams tend to cause unusually high water stages. This has repercussions in many operational and design problems such as overturning moment on river structures due to moving ice, forces on ice booms, spring flooding and associated stage-frequency relationships, river bed scour due to surges from released jams, to mention but a few.

At present, complete mathematical simulation of water stages during breakup is only a hope for the future. There are simply too many unknowns: It is not known whether, where, and when a jam will form. Even if it is assumed that a jam has been initiated at a specified location, it is not known exactly what occurs at the toe (downstream end) and thus it is not possible to formulate an appropriate boundary condition for the jam's subsequent evolution; and even if the configuration of an ice jam at a specified time is given or assumed, it is not known how, why, and when the jam will release.

Faced with such difficulties, research has concentrated on the relatively simple problem of equilibrium jams, i.e., jams that no longer evolve. This approach has considerable practical merit since it can be argued that, under certain circumstances, the highest water stages occur when a jam has attained equilibrium. Theoretical work has resulted in a model for *floating* jams in equilibrium that has been tested with some success versus experimental results. Virtually nothing is known, however, about

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grounded (otherwise known as "dry") jams. Fig. 1 gives a schematic illustration of an equilibrium jam. In Fig. 1 it has been assumed that the jam was initiated at the edge of an undisturbed ice sheet and attained a steady-state condition. Typically, there are three regions within the length of such a jam:

1. Upstream transition: For a certain distance below the head of the jam, the thickness increases and approaches an asymptotic value. The flow under the jam is of the gradually varied type. As will be analyzed later, this should be the case for jams of the "wide channel" kind which represent a very common occurrence (see also pertinent measurements in Ref. 4). "Narrow channel" jams may not exhibit an upstream transition; from theoretical considerations it could be shown that for a prismatic channel the thickness of a narrow jam should not change in the downstream direction.
2. Equilibrium thickness reach: The thickness of the jam is relatively uniform and approximately equal to the asymptotic value mentioned earlier. This value has been termed the equilibrium thickness (21,22). The flow under the jam is uniform and the water surface is equal to the channel bed slope, S. This concept applies to prismatic channels but may be extrapolated to natural streams by replacing S with the open-water surface slope, provided: (1) The reach under consideration is long enough to permit meaningful averaging of channel characteristics; and (2) the flow is free to assume a uniform condition, i.e., there are no significant control effects in the reach of interest.
3. Downstream transition: Below the equilibrium reach, the water surface profile steepens progressively to meet the relatively low water stage that prevails at the toe of the jam (e.g., see Refs. 6–8). The jam and flow configurations in this region are difficult to assess and could vary depending on local conditions and mode of jam initiation.

It should be recognized that the type of jam depicted in Fig. 1 occurs often but is by no means universal. If the jam is too short, the equilibrium thickness region may not exist, while in reaches that are strongly influenced by controls (e.g., river mouths) uniform flow may not occur under the jam.

In the following sections, it will be attempted to review and assess the available theory and data on ice jams, present illustrative case studies

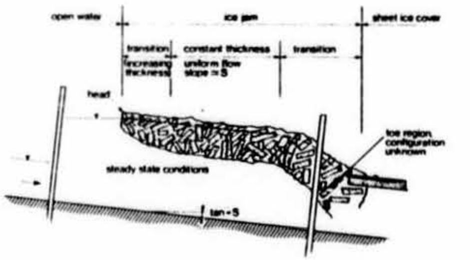

FIG. 1.—Schematic Illustration of Equilibrium Jam

Flow and Sediment Transport Processes," Iowa Institute of Hydraulic Research Report No. 218, The University of Iowa, Iowa City, Iowa, 1979.
19. Tatinclaux, J. C., "Equilibrium Thickness of Ice Jams," *Journal of the Hydraulics Division*, ASCE, Vol. 103, No. HY9, Proc. Paper 13179, Sept., 1977, pp. 959–974.
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22. Uzuner, M. S., and Kennedy, J. F., "Theoretical Model of River Ice Jams," *Journal of the Hydraulics Division*, ASCE, Vol. 102, No. HY9, Proc. Paper 12412, Sept., 1976, pp. 1365–1383.

and test the theory, and outline methods for practical application of the results.

Theoretical Background

Hydraulics of Flow under an Ice Jam.—A river cross section located in a reach where a floating jam has formed is sketched in Fig. 2. In the following, flow through the voids of the jam will be neglected and the jam assumed to have an equilibrium thickness reach with uniform flow underneath. The longitudinal water surface slope is then equal to the river slope under open-water conditions. The velocity distribution in any one vertical will be as sketched in Fig. 2 where the dashed line represents the locus of the maximum velocity points; for a very wide channel, relative to its depth, the shear stress along this line is nearly zero. As a first approximation, the flows in the two subsections defined by the maximum velocity line are, respectively, controlled by the average shear stresses on the jam underside, τ_i, and on the river bed, τ_b.

Let Q_i, A_i, V_i, and R_i be the discharge, area, average velocity, and hydraulic radius for the ice-controlled subsection, respectively. Then $V_i = Q_i/A_i$ and $R_i = A_i/W$ (Fig. 2); the Manning roughness coefficient, n_i, and friction factor, f_i, for the jam underside are defined as $n_i = V_i^{-1} R_i^{2/3} S^{1/2}$ (metric units), and $f_i = 8\tau_i/\rho V_i^2$ with $\tau_i = \rho g R_i S$ and ρ = water density, and g = acceleration of gravity. Similar relationships apply to quantities pertaining to the bed-controlled flow subsection; in the following, such quantities will be designated using the same symbols as in the preceding, but with the suffix "b" in place of "i."

For the overall, composite roughness flow under the jam (designated with the suffix "o"), the well-known Sabaneev equations may be used:

$$n_o = \left(\frac{n_b^{3/2} + n_i^{3/2}}{2}\right)^{2/3} \quad \text{(1a)}$$

$$f_o = \frac{f_i + f_b}{2} \quad \text{(1b)}$$

$$\frac{R_i}{R_b} = \left(\frac{n_i}{n_b}\right)^{3/2} \quad \text{(2a)}$$

$$\frac{R_i}{R_b} = \frac{f_i}{f_b} \quad \text{(2b)}$$

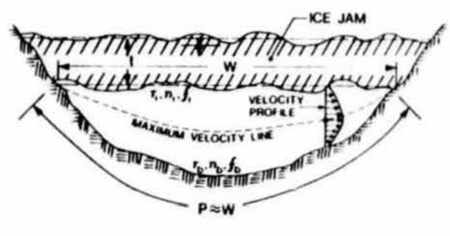

FIG. 2.—River Cross Section within Equilibrium Thickness Region of Floating Jam

in which equivalent relationships in terms of the friction factors instead of the Manning coefficients are numbered as Eq. 1*b*, Eq. 2*b*, etc. Moreover, for wide channels:

$$R_i + R_b \simeq 2R_o \quad \text{(3)}$$

It can be shown that Eqs. 1–2 are exact when $V_i = V_b$. For two-dimensional flow, this condition will be nearly satisfied if the shear velocities associated with the bed and jam, respectively, are either small compared to V_i and V_b or not very different from each other (2). This expectation is supported by laboratory and theoretical data (12,18). For wide channels of arbitrary cross-sectional shape, the condition $V_i = V_b$ can only be tested empirically. Examination of field data by the writer (unpublished) showed V_i and V_b to be within 10% of each other. However, the data used were for the relatively smooth sheet ice covers that occur in midwinter, thus this finding may not apply to the very rough flows under ice jams. To a large degree then, use of Eqs. 1–2 is justified by a lack of more reliable information.

Hydraulic resistance characteristics of the river bed, n_b and f_b, can be obtained from hydrometric surveys in the reach of interest during open-water conditions. Though jam stages are generally high, a large portion of the water depth is occupied by the jam itself and the flow part controlled by the jam underside (Fig. 2); thus, usual values of R_b may represent low open-water stages at which n_b (or f_b) is stage-dependent. The dependence of n_b on open-water depth is assumed to apply to flow under an ice jam by using R_b in place of the open-water depth. For a given reach, one may thus write

$$n_b \text{ (or } f_b) = \text{a function of } R_b \quad \text{(4)}$$

Resistance characteristics of the underside of an ice jam have not been documented widely to date. The few pertinent data known to the writer are analyzed briefly in the following.

Kennedy (11) investigated the characteristics of log jams; the bottom roughness of a jam was found to increase with increasing jam thickness, based on field and laboratory measurements. The absolute roughness-thickness relationship suggested by Kennedy is shown in Fig. 3. Intuitively, one would expect that the curve of Fig. 3 should have a horizontal asymptote, i.e., the roughness should not increase indefinitely but attain a constant value beyond a limiting value of thickness. The limiting thickness and the maximum roughness would probably depend on the di-

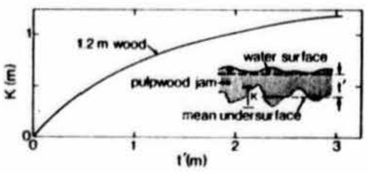

FIG. 3.—Kennedy's Roughness-Thickness Relationship for Log Jams (Note $t' = s_w t$; s_w = Specific Gravity of Wood)

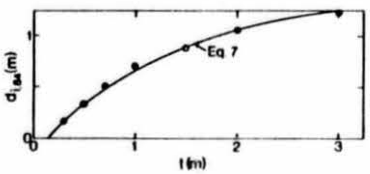

FIG. 4.—Variation of $d_{i,84}$ with t as Deduced from Nezhikovskiy's Results

search Council; F. Sampson, B. Tutt, M. Vanderkraan of B. C. Hydro; and W. Moody of Environment Canada. Review comments by T. M. Dick and Y. L. Lau are greatly appreciated.

Appendix.—References

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theory was tested with satisfactory outcome by plotting H/WS versus ξ using the available field data.

Because the thickness of spring jams cannot be measured at present, the data do not permit direct estimates of the applicable roughness and internal friction characteristics. A method has been outlined for arriving at such estimates indirectly, based on existing hydraulic resistance data for winter jams (15) of which the thickness can be measured once freezing in place has occurred to allow safe access. This method was applied to 13 case studies with the following results:

1. The coefficient, μ, which depends on the internal friction of the jam was between 0.6 and 3.5. The lower limit of this range was obtained under conditions of considerable uncertainty with regard to both jam stage and applicable discharge. The upper limit was found for a relatively thin jam (jam thickness $\simeq 2\times$ sheet ice thickness) and thus might have been influenced by ice-bank friction. In the remaining 11 cases, μ was between 0.8 and 1.3, which was considered encouraging in view of the crudeness of both the data and the analytical procedure. An average value of 1.2 was suggested for applications.
2. The composite friction factor, f_o, varied between 0.09 and 0.67 and exhibited a tendency to decrease with increasing dimensionless discharge, ξ. The same tendency was suggested earlier when comparing the conventional theory with data on the relationship between H/WS and ξ.
3. The ratio f_i/f_o ranged between 0.6 and 1.6 but showed no tendency to vary with ξ.

Two methods of applying the present results were outlined: (1) A detailed method that makes use of the ice jam theory in conjunction with hydraulic resistance considerations; and (2) a simplified method that uses the "average" η versus ξ relationship as defined by the data. Examples indicated that the detailed method gives better results than the simplified method.

The major limitations of applying the theory to practical questions derive from the assumption of an equilibrium jam affecting fully the site of interest. This may or may not be the case in any given year; thus the theory can only provide an upper bound of anticipated water level as a function of discharge. To account for special constraints that may be present at a given site (low flood plains, by-pass channels, etc.) careful field inspections are necessary.

Acknowledgments

A part of the work reported herein was performed under a field research program on ice jams that was carried out by the Transportation and Surface Water Engineering Department of Alberta Research Council in cooperation with Alberta Environment, Alberta Transportation, and University of Alberta. Permission by Environment Canada to prepare this paper is gratefully acknowledged. The field data reported herein have been obtained and processed with the assistance of M. Anderson, G. Childs, G. Putz, C. Ray, T. Ridgway, J. Thompson of Alberta Re-

mensions and geometry of the fragments that compose the jam.

Field data on ice jams in the Soviet Union have been reanalyzed by Nezhikhovskiy (15) for the period of freeze up. He found that the jam roughness, defined as the average deviation from the mean of numerous thickness measurements, increased with the average jam thickness, t, and thus proceeded to establish empirical relationships between n_i and t. Three types of accumulations were identified, respectively, comprising loose slush, dense slush, and ice floes. The latter type is thought to be the most relevant to spring jams. The writer (2) noted that, for the very rough boundaries of ice jams, n_i should depend on R_i as well as t, and attributed Nezhikhovskiy's findings to a restricted range of R_i values (approximately 1.0 m–1.5 m). With this interpretation, it was possible to account for the effects of R_i and derive the following empirical equation (2):

$$n_i = 0.072\, R_i^{-0.23} t^{0.4} \qquad (5a)$$

$$f_i = 0.4\left(\frac{t}{R_i}\right)^{0.8} \qquad (5b)$$

in which $0.5 \le t/R_i \le 3.0$; and $t \le 3$ m. Eqs. 5a–5b are based on Nezhikhovskiy's suggestion that the jam roughness increases linearly with t.

An alternative interpretation of Nezhikhovskiy's data can be obtained by assuming that the well-known logarithmic dependence of the friction factor on relative roughness (fully rough flow regime) applies to ice jams. Then, R_i may be taken as constant ($\simeq 1.25$ m) for Nezhikhovskiy's data, and the jam roughness can be calculated as a function of thickness. For the variation of f_i, an equation proposed by Limerinos (13) has been used, derived from natural stream data:

$$f_i = \left[1.16 + 2\log\left(\frac{R_i}{d_{i,84}}\right)\right]^{-2} \qquad (6)$$

in which $d_{i,84}$ = (by analogy with stream beds) the roughness value that exceeds 84% of the values in a representative set of individual roughness measurements. The writer (2) has found Limerinos' equation to apply in the range $0.4 \le R/d_{84} \le 150$, using data of several other investigators. Using the identity $n/R^{1/6} = 0.113\sqrt{f}$, values of $d_{i,84}$ were computed to fit Nezhikhovskiy's n_i-t relationship for ice floes and are shown plotted versus t in Fig. 4. The graph in Fig. 4 is adequately represented by the empirical equation (metric units)

$$d_{i,84} = 1.43\{1 - \exp[-0.734(t - 0.15)]\} \qquad (7)$$

It is noteworthy that the curve of Fig. 4 is similar to that of Fig. 3 that applies to log jams.

Eqs. 5a (or 5b) and 6 (with 7) give two different interpretations of Nezhikhovskiy's results, both designed to account for the effects of R_i; at present, it is difficult to state which is more realistic, though the latter is more appealing to intuition. Finally, it is noted that laboratory data by Tatinclaux and Cheng (20) and Calkins and Müller (5) also showed f_i and n_i to increase with t.

Ice Jam Theories.—The earliest quantitative treatment of jams known to the writer has been reported by Kennedy (11) and deals with the forces exerted by log jams. For shores that develop friction, Kennedy based his analysis on "the analogy between pulpwood in a river and granular material contained between two parallel walls" and noted that ". . . it was subsequently discovered that Janssen had developed the same relation in connection with the design of grain elevators."

Letting T = force per unit width with which the jam upstream is pressing against the jam downstream in the direction of the current; τ_i = the drag of the current on the undersurface of the jam per unit area (assumed constant along the stream); k_o = the coefficient of lateral thrust; and λ = the sliding coefficient of wood against wood, the force equilibrium for the jam is

$$-W\frac{dT}{dx} + W\tau_i - 2k_o\lambda T = 0 \quad \text{(8)}$$

in which W = streamwidth (assumed constant); and x = streamwise distance measured from the head (upstream end) of the jam. Integrating Eq. 8 gives

$$T = \frac{W\tau_i}{2k_o\lambda}\left[1 - \exp\left(-\frac{2k_o\lambda x}{W}\right)\right] \quad \text{(9)}$$

in which it has been assumed that $T = 0$ at $x = 0$. According to Eq. 9, $T \simeq T_\infty$ = constant for $x/W > 3/2k_o\lambda$, with

$$T_\infty = \frac{W\tau_i}{2k_o\lambda} \quad \text{(10)}$$

Kennedy (11) reported that Eq. 9 was adequate only when the log length was less than one-thirtieth of W. Otherwise, the percentage of force transmitted directly to the shore increased with the ratio of log length to stream width. From laboratory tests, the coefficient $2k_o\lambda$ was evaluated as 0.4.

There are two shortcomings in this analysis, as has been pointed out in Ref. 21: The streamwise component of the aggregate weight of the jam has been neglected, and it has been implicitly assumed that t is independent of x and that the flow depth under the jam is constant. Another debatable point is the assumed boundary condition, e.g., $T = 0$ at $x = 0$, as will be analyzed later.

An analysis similar to Kennedy's was presented later for ice jams by Berdennikov (3) who found an equivalent expression for T. From force measurements on ice booms, Berdennikov reported that T/T_∞ was adequately described by Eq. 9 with $2k_o\lambda = 1.0$, and the predicted values of T_∞ ranged from 75% to 137% of respective measured values.

Pariset and others (16,17) developed a more comprehensive theory: The streamwise component of the jam weight was accounted for but the jam thickness was again assumed to be independent of x. The upstream boundary condition was generalized by putting $T = T_o \neq 0$ at $x = 0$. The resistance to shear at the sides of the jam was assumed to consist of a cohesive term as well as a frictional term. It should be noted here that cohesion is not likely to develop under spring breakup conditions but

the jam released, either due to local blasting that was underway at that time or due to natural causes. Later, a new jam, some 10 km long, formed further downstream. The head of this jam was about 15 km below the gage and the latter experienced a secondary peak about 1 m less than that plotted in Fig. 9. It is estimated that, had the jam formed at the gage, it would have caused a stage of about 322 m which is fairly close to the prediction of the detailed method-alternative 2. (In reality, such a stage is improbable for the gage site where the crest of local flood protection dykes has an elevation of only 319.4 m.) The 1975 and 1976 breakups were mild with only minor and brief jamming. The 1979 breakup was caused by the surge of a jam that released a few kilometers upstream of the gage site and resulted in a maximum rate of water level rise of 4.5 m/h. The peak stage was caused by brief downstream jamming during which the stage kept rising (1 m/h on release); thus, equilibrium was not attained.

The 1976 peak at Watino (Fig. 10) was estimated from a post-breakup survey of ice shear walls left on the river banks and is 0.6 m higher than the predicted value. A jam believed to have reached equilibrium is thought responsible for the 1977 peak which is 1 m less than the corresponding prediction. The 1979 peak was caused by a surge from a jam that released some 14 km upstream of Watino.

Fig. 11 shows the best agreement between data and theory (detailed method). It is perhaps significant that gage malfunction due to ice damage, which is frequent at Peace River and Watino, is rare at Thamesville owing to relatively deep, tranquil flow and thin ice cover.

Figs. 9–11 suggest that alternative 2 of the detailed method gives the best results; at the same time, an increasing overprediction as the discharge increases is evident. This is plausible since the larger the discharge the lesser the probability that ice jams will form and attain equilibrium.

Finally, it is noted that the freeze up peaks shown in Fig. 11 appear to fit the general trend exhibited by the breakup peaks. The main difference lies in the fact that freeze up flows are generally much less than breakup ones so that the freeze up points occupy the lower left side of the graph.

Summary and Conclusions

The existing theory of floating ice jams in rivers has been examined in the light of field data accumulated in recent years. The theory developed gradually since the late 1950s (11,16,17,3,21,22,19) and predicts the jam thickness so that the jam, considered a granular mass, can withstand the applied forces. These forces are caused by the flow shear stress and by the streamwise component of the jam's own weight. Two types of jams have been identified: "wide" and "narrow" channel jams. It was shown herein that the former type should be far more common in nature since the latter type can only exist at very small aspect ratios. The theory was shown to give the dimensionless jam thickness, t/WS, and overall depth, H/WS, as increasing functions of the dimensionless discharge ξ ($= (q^2/gS)^{1/3}/WS$). The internal friction of the jam along with the river bed and jam friction factors appear as parameters of these functions. The

The data points in Figs. 9–10 are peak stages during breakup as determined from the following sources: Water Survey of Canada gage records; Alberta Envrionment; British Columbia Hydro; and writer's own observations. The open-water rating curves are also shown for convenience in assessing the ice effect. The discharge values associated with the data points are generally mere estimates since the gage rating curves do not apply during periods of ice effects. These estimates are provided by Water Survey of Canada based on such evidence as upstream inflows, runoff conditions, and interpolations between times of discharge measurements. The applicable error limits are not known and would probably vary from site to site; the percent error should also depend on discharge and, at a given site, should decrease with increasing discharge.

Fig. 11 is based on a comprehensive analysis of gage records for Thamesville during the period 1959–79 (Beltaos and Poyser, unpublished data). The points associated with breakup are of three kinds: (1) Maximum ice-influenced stage; (2) stage at maximum ice effect or "backwater"; and (3) points applicable to discharge measurements that have been carried out during breakup. Items 1 and 2 do not generally coincide at this site because breakup usually occurs during a relatively steep rise in flow. The peak stage may be associated with unstable jams due to relatively high flow which results in relatively low backwaters. More stable jams may form at lower flows and thus produce larger backwaters. Item 3 provides the most accurate information but the associated data points may or may not represent peak stages or backwaters. Significant freezeup peaks are also shown in Fig. 11.

In general, ice conditions associated with the data points of Figs. 9–11 are unknown, i.e., it is unknown whether a plotted peak was caused by an ice jam affecting fully or partly the gage site or by surges from released upstream jams. Where simultaneous visual observations are available, pertinent notes are made in Figs. 9–11.

Reach-average hydraulic parameters under open-water conditions were obtained from Kellerhals, et al. (10) for the Peace and Smoky Rivers, and from a hydrometric survey near Thamesville that was carried out in June 1980. For the Peace and Smoky Rivers, constant widths were assumed. For the Thames, however, it was necessary to account for the (significant) variation of W with stage in applying the detailed method. The ranges of the dimensionless discharge, ξ, for the breakup peaks of Figs. 9–11 are 34–115 (Peace River), 35–84 (Smoky River) and 900–2,900 (Thames River).

Considering that, ideally, the theory should provide upper envelopes of the observed peaks, the largest discrepancy between theory and data occurs for Peace River (Fig. 9). It is likely that significant equilibrium jams do not occur at or near this site and this coincides with the writer's experience during observations carried out in 1974, 1975, 1976, and 1979. The 1974 peak was caused by a 4.8 km long jam of which the head was 4.5 km below the gage. Allowing for the channel slope, it is estimated that a 1.6 m high stage would have been experienced had the jam head been located at the gage. Fig. 9 shows that even this higher stage is low compared to the theoretical prediction. This may be attributed to the fact that the jam did not reach equilibrium: During the "life" of this jam the water level kept rising at the gage. The rate of rise was 0.2 m/h when

may occur briefly during freeze up when the water is supercooled. Limiting the present analysis to breakup jams, cohesion may be neglected, and Pariset, et al.'s (17) prediction of T reads

$$T = T_o + \left\{\left[\frac{W(\tau_i + w_i)}{2k_o\lambda}\right] - T_o\right\}\left[1 - \exp\left(-\frac{2k_o\lambda x}{W}\right)\right] \quad \text{(11)}$$

in which $\lambda = \tan\phi$; and ϕ = angle of internal friction of the jam. The asymptotic value of T is

$$T_\infty = \frac{W(\tau_i + w_i)}{2k_o\lambda} \quad \text{(12)}$$

with w_i = streamwise component of the weight of the jam per unit area:

$$w_i = \rho_i g S t \quad \text{(13)}$$

in which ρ_i = density of ice.

Pariset, et al. (17) distinguished between "narrow" and "wide" channels depending on the sign of $T_\infty - T_o$:

1. *"Narrow" channels*: $T_\infty - T_o < 0$. The stress, T, will decrease with x and its maximum value is T_o; the thickness of the jam is governed by the "no spill" condition at the head, which leads to:

$$V_u = \sqrt{2g(1 - s_i)t} \quad \text{(14)}$$

in which V_u = average flow velocity under the jam; and s_i = specific gravity of ice. The thickness of "narrow" channel jams has been investigated recently by Tatinclaux (19) who presented a new theoretical formulation and laboratory data.

2. *"Wide" channels*: $T_\infty - T_o > 0$. The stress, T, increases with x and the jam thickness is governed by structural considerations. The asymptotic value of T is equal to the compressive strength of the jam. The latter is equal to $k_1\rho_i(1 - s_i)\,gt^2/2$ in which k_1 varies from 1.0 to the more probable value of $\tan^2(\pi/4 + \phi/2)$, if it is assumed that the full passive resistance of the jam is mobilized. This requirement leads to

$$\mu\rho_i(1 - s_i)\,gt^2 = W(\tau_i + \rho_i g S t) \quad \text{(15)}$$

$$\text{with} \quad \mu = k_o\lambda k_1 = k_o\lambda\tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \quad \text{(16)}$$

Eq. 15 provides a means for predicting t if τ_i can be estimated. From data on the Beauharnois Canal, Pariset, et al., reported $\mu = 1.28$. Using this value and putting $k_o\lambda = 0.5$ as suggested by Berdennikov, Eq. 16 can be solved for ϕ to give $\phi = 26°$. Then $\lambda = \tan 26° = 0.49$ which implies $k_o \simeq 1.0$; this is the upper limit of the range suggested for k_o by Pariset, et al. (17).

A decade later, Uzuner and Kennedy (21,22) formulated the time-dependent differential equations describing the force equilibrium within, and the flow hydraulics beneath, the jam. These equations are too complex to permit general solution. It was shown, however, that after an evolutionary period the jam attains a quasi-steady condition. This condition is characterized by a steady upstream advance of the jam's head at a rate governed by the flux of incoming ice floes; an observer moving with the head of the jam perceives a steady condition. Over a certain distance below the head, the jam thickness increases to an equilibrium value, t, remaining constant thereafter. Uzuner and Kennedy's analysis is probably the most precise formulation of the problem so far and introduces the least number of assumptions; for this reason, it will be reviewed here in some detail with a view to comparing it with previous theories.

For an ice jam formed by internal collapse, the normal stress, σ_x, in the flow direction and the shear stress, τ_f (equal to the strength), at the sides are

$$\sigma_x = k_x \bar{\sigma}_z \quad (17)$$

$$\tau_f = C_o \bar{\sigma}_z + C_i \quad (18)$$

in which k_x, C_o = dimensionless coefficients; $\bar{\sigma}_z$ = thickness-averaged normal stress in the direction perpendicular to the water surface = 0.5 $(1 - s_i)(1 - p)\rho_i g t$ with p = porosity of the jam; and C_i = cohesion of the jam. The parameters k_x and C_o were found to be strain rate dependent in laboratory tests and this was attributed to cohesion. According to Merino (14), cohesion between ice fragments develops when the water film surrounding the fragments freezes, thus forming a natural weld. This phenomenon is not expected to occur during breakup, as mentioned earlier; thus it may be assumed $C_i \simeq 0$ and $k_x \simeq$ const., $C_o \simeq$ const.

It can be shown (1) that Uzuner and Kennedy's equation expressing the balance of forces within the jam will coincide with that of Pariset, et al., for "wide" channels if μ is taken equal to $C_o(1 - p)$. This coincidence might have been expected since the former analysis assumes a jam formed by internal collapse which is also the underlying principle of the latter for "wide" jams.

At this point it is of interest to consider further the distinction between "narrow" and "wide" channel jams. As indicated earlier, this distinction is based on the sign of $T_\infty - T_o$, which in turn depends on the magnitude of T_o. Pariset, et al. (17) suggested that T_o is "the hydrodynamic force of the current against the upstream limit of the cover." On the other hand, Uzuner and Kennedy (22) attributed T_o to the momentum of ice floes arriving at the head of the jam which implicitly neglects the hydrodynamic force. Evidently, the value of T_o depends on several factors that cannot be easily included in a compact equation. There exists, however, an upper limit for T_o, imposed by the compressive strength of the jam:

$$T_o \leq \frac{k_1 \rho_i (1 - s_i) g t^2}{2} \quad (19)$$

Since T_∞ must be less than T_o for "narrow" jams, the following condition must be satisfied (recall Eq. 12):

$$\frac{W(\tau_i + w_i)}{2 k_o \lambda} < \frac{k_1 \rho_i (1 - s_i) g t^2}{2} \quad (20)$$

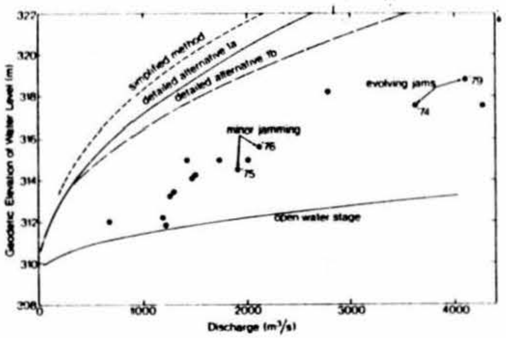
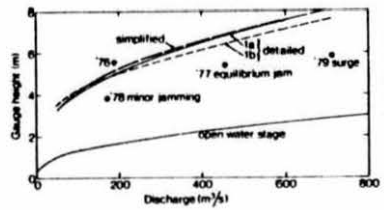


FIG. 9.—Peak Breakup Stage versus Discharge; Peace River at Peace River (W = 600 m, S = 0.00035)



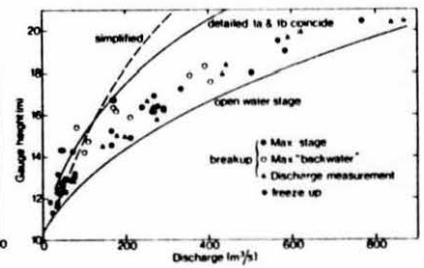


FIG. 10.—Peak Breakup Stage versus Discharge; Smoky River at Watino (W = 260 m, S = 0.0052)

FIG. 11.—Freeze Up and Breakup Stages versus Discharges; Thames River at Thamesville (W = 40–65 m; S = 0.00023)

sired to account for variations of W with stage, the preceding procedure may be modified to carry out the computation as a trial-and-error process.

Simplified Method.—This is based on direct use of Fig. 7 by first drawing an "average" line through the data points. The computation steps are then as follows, for any given Q:

1. Compute ξ by Eq. 27 and use the "average" η-ξ line to determine η.
2. Compute $H = \eta WS$; this should be close to the average water depth and by entering the open-water stage versus R_b graph, with H in place of R_b, the desired jam stage can be determined. Note that this procedure assumes that W does not change with stage. It would be an unnecessary elaboration to account for changes of width with stage when using the simplified method.

Examples.—Figs. 9–11 give the results of calculations by the preceding methods for the following sites: (1) Peace River at the town of Peace River (Fig. 9); (2) Smoky River at the town of Watino (Fig. 10); and (3) Thames River at the town of Thamesville (Fig. 11).

of a jam's underside which has been evaluated on the basis of only one set of field data.

4. The theory does not take into account special constraints that may be present such as existence of low flood plains, by-pass channels, possible effects of bridge piers on jamming frequency, etc. The possible effects of such features require assessment by careful inspection of the site of interest.

Clearly, no great confidence can be placed in predictions based on the existing theoretical knowledge. At the same time, such predictions may be useful in cases where there is little or no reliable information on breakup stages from other sources (e.g., gage records, newspaper accounts, visible high ice marks, etc.).

With these qualifications, let it be assumed that a jam stage-discharge curve is to be generated for a reach about which the following information is given: channel slope; open-water rating curve (stage versus discharge); reach-average flow area and reach-average water surface width versus stage. From this information, the relationships R_b (reach-average, open-water flow depth) versus stage and f_b versus R_b can be derived. From considerations outlined in the previous sections, two methods of calculation seem to be possible: a detailed method and a simplified method.

Detailed Method.—After manipulation of the equations stated so far, the following procedure is suggested:

1. Assume a value of t and compute R_i from (see Eq. 28)

$$R_i = t\left(\frac{\mu t}{13.6\ WS} - 0.92\right) \qquad (29)$$

Use $\mu = 1.2$ unless there exists evidence favoring a different value.

2. Compute f_i; this may be done either by Eq. 5*b* (alternative 1) or by Eqs. 6–7 (alternative 2).

3. Determine f_b and R_b so as to satisfy both Eqs. 2*b* and 4. This may be done conveniently by first preparing a graph of f_b/R_b versus R_b using the already known relationship between f_b and R_b; since Eq. 2*b* requires that $f_i/R_i = f_b/R_b$, compute f_i/R_i and enter the graph with this value to find R_b and compute f_b as $(f_i/R_i)\ R_b$.

4. Compute f_o and h from Eqs. 1*b* and 3, respectively (note $h = 2R_o$), and determine Q from

$$Q = Wh\sqrt{\left(\frac{4}{f_o}\right) ghS} \qquad (30)$$

5. Enter the open-water stage versus R_b graph with h in place of R_b, and find the stage corresponding to the bottom surface of the jam; add $0.92t$ to find the jam stage for the discharge value computed in the previous step.

6. Repeat for a few other values of t and plot jam stage versus discharge.

It is noted that this approach assumes that W does not change with stage which is a fair approximation for most natural streams. It it is de-

Putting $\tau_i = \rho g R_i S$, using Eqs. 13, 14, and 16, and recalling earlier analysis of the hydraulics of flow under a jam gives, after some algebra

$$\frac{W}{H} < \frac{\left(\frac{2\mu s_i}{f_o}\right)\left(\frac{t}{H}\right)\left(1 - \frac{s_i t}{H}\right)}{\left[\frac{f_i}{2f_o} + \frac{s_i t}{H}\left(1 - \frac{f_i}{2f_o}\right)\right]} \qquad (21)$$

in which H = overall water depth:

$$H = h + s_i t \qquad (22)$$

with h = average flow depth under the jam. According to Pariset, et al. (17), the maximum possible value of t/H is 1/3 for a narrow jam. Using this value, and putting $\mu = 1.3$, $s_i = 0.92$, $f_i/2f_o = 1/2$ (relatively "smooth" jam), Eq. 21 shows that a "narrow" jam cannot form when the aspect ratio, W/H, exceeds the following limiting value:

$$\left(\frac{W}{H}\right)_{max} = \frac{0.85}{f_o} \qquad (23)$$

Even with a relatively "smooth" jam for which f_o is as small as 0.1, Eq. 23 gives $(W/H)_{max} = 8.5$; if t/H had been taken equal to 0.1, this value would have been reduced to 4.0. Clearly, "narrow" channel jams should be rare occurrences in nature.

Calculation of Jam Stage.—For reasons explained in the previous section, the remaining analysis will be confined to "wide" channel jams. The overall water depth due to an ice jam, H, is given by Eq. 22 and represents the most important dependent parameter of the jam. Moreover, the present state of technology is such that only H can be observed directly; it is practically impossible to measure the thickness of a spring jam.

The depth of flow, h, under the jam is

$$h = \left[\frac{q}{\left(\frac{4gS}{f_o}\right)^{1/2}}\right]^{2/3} \qquad (24)$$

in which $q = Q/W$. Recalling Eq. 15 and solving for t gives, after some algebra

$$t = \frac{WS}{2\mu(1 - s_i)}\left\{1 + \left[1 + \frac{(2f_o)^{1/3}\mu(1 - s_i)}{s_i}\left(\frac{f_i}{f_o}\right)\frac{\left(\frac{q^2}{gS}\right)^{1/3}}{WS}\right]^{1/2}\right\} \qquad (25)$$

Multiplying Eq. 25 by s_i, adding to Eq. 24 and working out the numerical coefficients (assume $s_i = 0.92$), gives

$$\eta \equiv \frac{H}{WS} = 0.63 f_o^{1/3}\xi + \frac{5.75}{\mu}\left[1 + \sqrt{1 + 0.11\,\mu f_o^{1/3}\left(\frac{f_i}{f_o}\right)\xi}\right] \qquad (26)$$

$$\text{in which} \quad \xi \equiv \frac{\left(\dfrac{q^2}{gS}\right)^{1/3}}{WS} = \frac{y_c}{WS^{4/3}} \quad \text{(27)}$$

with y_c = critical flow depth. Clearly, the dimensionless jam stage, η, depends primarily on the dimensionless discharge, ξ, and on the internal friction of the jam, and secondarily on the friction factors at the flow boundaries. The dimensionless parameters of Eq. 26 have been chosen for convenience in interpreting the data.

An interesting feature of Eq. 26 is that η does not vanish when Q (and thus ξ) are zero. This result is contrary to intuition and can be explained as follows. Firstly, it is noted that at $\xi = 0$, the flow depth vanishes but the (submerged) jam thickness becomes equal to $11.5\ WS/\mu$. Recalling Eq. 15 shows that the jam thickens to withstand two types of force: the hydraulic friction and the streamwise component of the jam's own weight. When $Q \rightarrow 0$, the former vanishes but the latter does not since S remains constant. Putting $\tau_i = 0$ and $s_i = 0.92$ in Eq. 15 gives again $11.5\ WS/\mu$ for the submerged jam thickness. It is now obvious that this implausible result is due to the assumption that the flow through the jam is negligible. This assumption is realistic under normal circumstances; however, as Q approaches zero, an increasing fraction of Q will flow through the voids of the jam and even before Q becomes zero, the jam will ground. When this occurs, the jam need not be as thick as indicated by Eqs. 15 and 25 because additional frictional resistance becomes available by contact with the river bed.

Comparison with Case Studies

Description of Data.—Most of the ice jam case studies utilized herein derive from field research programs that have been described in Refs. 1 and 9. A novelty of this program consists of documenting the "instantaneous" water level profile along any observed jam as follows: from small aircraft or from ground access points, photos are taken of the jam stage against the river banks and used later for identification and survey. Examples are shown in Figs. 5–6. When a jam profile has a section that is parallel to the open-water surface, the jam can be assumed to be in equilibrium. From cross-sectional measurements and slope surveys, reach-averaged values of H, W, and S can be determined. Unfortunately, it is not possible to determine directly h and t because there is no capability at present for measuring the thickness of a spring jam. Estimates of the average thickness of the jam in the equilibrium reach are possible only through an indirect analysis, as will be analyzed later. The discharge, Q, assigned to each jam is that which prevailed at the time the jam was observed. The actual value of Q responsible for the formation of the jam can be higher than the assumed Q but not lower. Thus, what is observed is a jam with possibly oversized thickness for the assigned Q. Letting H_{obs} and Q_{obs} be the "observed" values of H and Q, as outlined previously, and H_a, Q_a be the corresponding values at the time when the jam was formed, we have $Q_a \geq Q_{obs}$ and $H_a \geq H_{obs}$. The pair H_a, Q_a satisfies the conditions of ice jam formation, i.e., it satisfies Eq. 26, if

the true internal friction of a granular ice mass. Note that the Thames River jams, though only 0.7 m–0.9 m thick, represent jam thickness to sheet ice thickness ratios of more than 3.5. Overall, Table 1 gives a measure of support to the method of analysis used herein which is based on Nezhikhovskiy's (15) resistance data, because it shows that μ takes on consistent values and at the same time is close to the value of 1.3 that has been reported earlier by others.

The parameters f_o and f_i/f_o that appear in Eq. 26 are seen in Table 1 to range from 0.09 to 0.67 and from 0.63 to 1.64, respectively; corresponding average values are 0.37 and 1.25. There is no consistent variation of f_i/f_o with ξ, thus the average value of 1.25 could conveniently be substituted in Eq. 26 considering that η is insensitive to f_i/f_o. At the same time, there is a trend for f_o to decrease with increasing ξ, as an earlier analysis concerning Fig. 7 had indicated. This trend is shown in Fig. 8. It should be understood at this point that no unique relationship between f_o and ξ can be expected because f_o should also depend on channel bed characteristics. This aspect is probably responsible for the large scatter of the data points in Fig. 8.

Table 2 summarizes additional but less comprehensive data for which the detailed analysis has not been performed. These data have also been used in Fig. 7.

Methods of Practical Application

Before considering methods of applying the preceding results to practical problems, it is advisable to enumerate the limitations of the theory:

1. The theory assumes a very wide rectangular prismatic channel. Application to rivers implies that a natural stream may, for the purpose of ice jam calculations, be replaced by a rectangular prismatic channel of equivalent average dimensions.
2. The theory applies to floating jams in equilibrium and gives the jam stage *assuming that a jam has formed, has reached equilibrium, and fully affects the location of interest*. In reality, one or more of these conditions may not be satisfied during a given breakup period. It follows that a theoretically derived jam stage-discharge relationship can only provide an upper envelope of actual events, barring the occurrence of severe grounded jams about which no theory is available.
3. Theoretical prediction depends partly on the hydraulic resistance

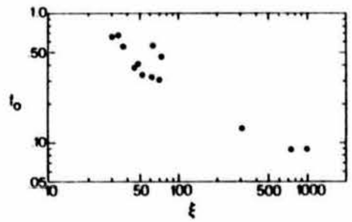


FIG. 8.—Variation of f_o with ξ

TABLE 2.—Additional Case Studies (Writer's Data)

Location (1)	Date (2)	Q, in cubic meters per second (3)	S, in meters per kilometer (4)	W, in meters (5)	H, in meters (6)	ξ (7)	η (8)	Probable condition of jam (9)
Smoky R. below Hunting Creek	Apr. 7, 1977	400	0.86	145	4.6	77.3	36.8	Evolving
Smoky R. at Watino	Apr. 7, 1977	456	0.52	250	4.1	66.7	31.5	Evolving
Smoky R. near mouth	Apr. 30, 1979	1,360	0.72	280	8.0	74.6	39.8	Evolving
Smoky R. near mouth	Apr. 29, 1979	1,270	0.72	286	9.2	68.8	44.9	Evolving
Peace R. below Peace River	May 1, 1979	3,930	0.15	600	9.3	342	103	Equilibrium
Heart R. near mouth	Apr. 8-9, 1977	10.5-13.3	4.36	36	2.8	8.0-9.4	17.8	Equilibrium
Thames R. near Middlemiss	Jan. 14, 1980	100	0.05	45	4.8	1,766	584	Equilibrium
Thames R. near Bothwell	Jan. 14, 1980	165	0.26	56	4.4	1,002	296	Equilibrium
Thames R. near Fairfield Museum	Mar. 18, 1980	130	0.81	44	4.2	290	118	Evolving

up nor for breakup. Thus, use of this approach cannot be considered "satisfactory" but may be viewed as the "least objectionable," for the present.

The procedure of analysis is as follows. For an assumed value of t, plot the lower jam boundary (0.92 t below the water stage) on each river cross section and determine reach-averaged values of A, W, $V(= Q/A)$, R_o $(= A/2W)$, and thus n_o (or f_o). Use Eqs. 1–4 to determine the remaining four unknowns, R_b, R_i, and n_b, n_i (or f_b, f_i). Repeat for a few additional values of t and plot n_i (or f_i) versus t. The intersection of this plot with Eq. 5 (5b) or with Eq. 6 (with 7) which can be evaluated from the data already generated, gives the value of t that satisfies all of the specified relationships and is, thus, the desired jam thickness. With this information, the coefficient μ may be computed from

$$\mu = 12.5 \frac{SW}{t} \left(1 + \frac{R_i}{0.92t}\right) \quad \text{(28)}$$

which is a rearranged version of Eq. 15.

Thirteen case studies, analyzed according to the aforementioned procedure, are summarized in Table 1 where it may be noted that fairly wide ranges of stream width, slope, and discharge are represented. The coefficient μ has an average value of about 1.2 and, for most case studies, individual μ's are close to this average. The lowest μ (= 0.6) was obtained for the jam on the Athabasca River near Pelican rapids. The data for this jam are, however, uncertain because it was documented using post-breakup evidence. The highest value of μ (= 3.5) was obtained for the Smoky River near Hunting Creek and does not seem to fit the pattern of the other jams. The sheet ice thickness in that case was about 0.6 m, i.e., one-half of the estimated jam thickness. It is possible that, as a jam approaches the configuration of a single layer of ice floes, μ will more and more reflect the effective ice-bank friction rather than

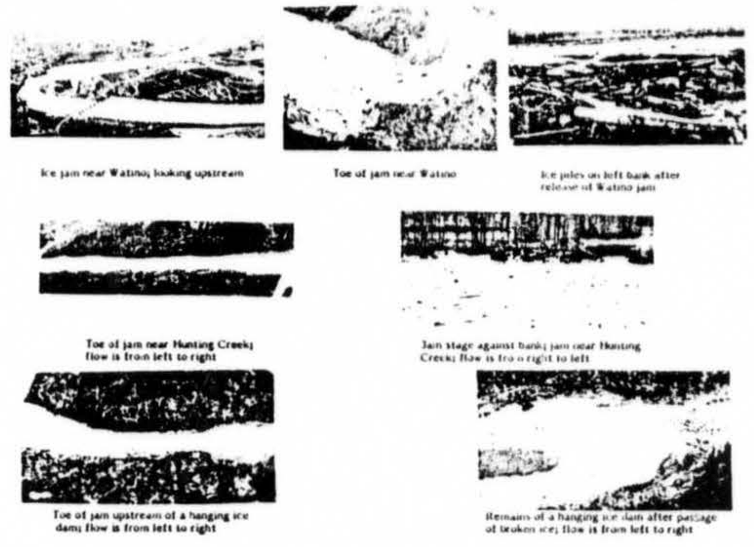


FIG. 5.—Photographs of Lower Smoky River Jams, April 1977

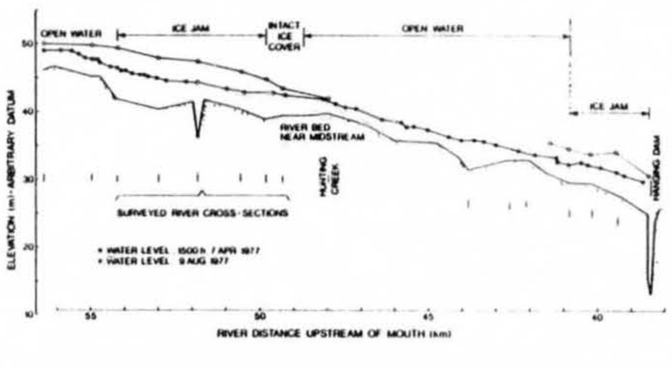


FIG. 6.—Profiles of Lower Smoky River under Open-Water and Breakup (Apr. 1977) Conditions (38–56 km above Mouth)

the theory is assumed valid. As Q decreases from Q_a to Q_{obs}, it is reasonable to assume further that the thickness of the jam does not change but that the flow depth under the jam decreases, as indicated by Eq. 24. Thus, the pair H_{obs}, Q_{obs} will not satisfy Eq. 26; if H' is the value of H obtained from Eq. 26 with $Q = Q_{obs}$, then $H' \le H_{obs}$. With plausible values of μ, f_i, f_o (see later analysis), Eq. 26 can be used as a rough guide to evaluate the relative error, $(H_{obs} - H')/H'$, inherent in the pair H_{obs}, Q_{obs}. Fortunately, it is found that even if Q_a is as large as $2Q_{obs}$, this error does not exceed 7%. This is acceptable considering the errors inherent in field data pertaining to ice jams.

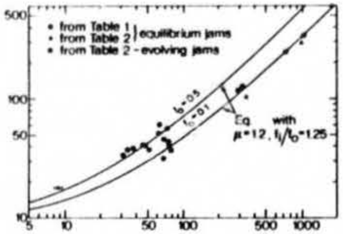


FIG. 7.—Test of Theory, Eq. 26

Testing of Theory.—Eq. 26 indicates that, according to the theory, the dimensionless jam stage, η, should depend on ξ with f_i, f_o, and μ as parameters. Available data are plotted in the form of η versus ξ in Fig. 7. The data are summarized in Tables 1–2 which will be analyzed later. In Fig. 7, different symbols have been used to describe ice jams deemed to have been, respectively, in equilibrium and in evolution. Despite considerable scatter, the data points in Fig. 7 show an unquestionable trend for η to increase with ξ which qualitatively supports Eq. 26. For a quantitative test of Eq. 26, μ and f_i/f_o were fixed at 1.2 and 1.25, respectively (average values found from a detailed analysis to be analyzed later), and η was calculated for f_o = 0.1 and 0.5. Comparison of the resulting curves with the data points indicates that the theory is basically sound while there seems to be a general trend for f_o to decrease with increasing ξ. The latter is plausible because f_o should decrease when t/H decreases and this can be shown to occur when ξ increases. It is of interest to note in Fig. 7 that the data points for evolving jams plot at or below the line defined by the points corresponding to equilibrium jams; this gives a measure of support to the expectation that the peak stage is attained at equilibrium.

Detailed Analysis—Indirect Methods

Detailed analysis of the data available to date is hampered by a lack of means to measure ice jam thicknesses during breakup. Typically, the measurable quantities are H, Q, W, S, a few representative river cross sections, and the relationship of Eq. 4. It is desired to determine h, t, R_i, R_b, and n_i, n_b (or f_i, f_b), i.e., a total of six unknowns. The available equations are five, i.e., Eqs. 1–4, and the flotation relation, Eq. 22. Clearly, the problem cannot be solved unless an additional relationship is introduced. Pariset, et al. (17) assumed $n_i = n_b = n_o$ ($f_i = f_b = f_o$). This assumption is arbitrary as there is no a priori reason why n_i should be equal to n_b for all jams in all rivers. The writer believes that use of Nezhikhovskiy's data (15), as interpreted in Eq. 5 (or Eq. 5*b*) or Eq. 6 (with Eq. 7), is preferable because these relationships have a basis on measurement. It is recognized that Nezhikhovskiy's data are subject to the usual inaccuracies one may expect for field observations of ice jams; in addition, these data have not been duplicated by other investigators (though indirectly corroborated in Refs. 5, 11, and 20) neither for freeze-

TABLE 1.—Analysis of 13 Case Studies of Ice Jams

Location (1)	Date (2)	Source (3)	Q, in cubic meters per second (4)	W, in meters (5)	S, in meters per kilometer (6)	H, in meters (7)	t, in meters (8)	f_o (9)	f_i/f_o (10)	μ (11)	ξ (12)	η (13)	Remarks (14)
Smoky R. near mouth	Apr. 9, 1976	Writer	210	227	0.72	5.5	3.1	0.65	1.12	1.0	30.5	33.8	Slope taken from topographic maps
Smoky R. near mouth	Apr. 10, 1976	Writer	283	228	0.72	6.2	3.3	0.55	1.17	1.0	36.9	37.9	Slope taken from topographic maps
Smoky R. near Hunting Creek	Apr. 7, 1977	Writer	456	163	0.83	5.9	1.2	0.46	0.63	3.5	73.0	43.6	Slope surveyed at site
Wapiti R. near Grande Prairie	Apr. 12, 1976	Writer	185	153	1.02	5.8	3.1	0.67	0.97	1.0	33.8	37.2	Slope surveyed at site
Thames R. near Thamesville	Jan. 12, 1980	Writer	108	41	0.77	3.9	0.9	0.13	0.81	1.0	308	125	Slope surveyed at site
Thames R. above Sherman Brown Bridge	Mar. 19, 1980	Writer	196	74	0.26	4.7	0.9	0.09	1.44	1.2	728	244	Slope surveyed at site
Thames R. at mouth	Mar. 20, 1980	Writer	195	106	0.13	4.6	0.7	0.09	1.44	1.3	1,005	334	Slope surveyed at site
Athabasca R. near Ft. McMurray	Apr. 16, 1977	(6)	930	560	0.36	8.2	4.4	0.38	1.43	1.0	45.7	40.7	Slope surveyed at site
	Apr. 18, 1977	(6)	1,100	560	0.36	7.4	3.2	0.33	1.42	1.3	51.1	36.7	Slope surveyed at site
	Apr. 20, 1978	(7)	800	410	0.35	8.1	4.0	0.31	1.64	0.9	72.2	56.4	Slope surveyed at site
	Apr. 21, 1978	(7)	600	405	0.35	7.4	4.0	0.32	1.62	0.8	60.8	52.2	Slope surveyed at site
	Apr. 30, 1979	(8)	1,150	680	0.30	8.1	3.8	0.40	1.30	1.2	48.6	39.7	Slope surveyed at site
Athabaska R. near Pelican Rapids	≈Apr. 17, 1978	(7)	450	193	0.75	8.7	4.7	0.56	1.27	0.6	62.5	60.1	Data based on high ice marks after jam release; slope taken from topographic maps

Beltaos (23)

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3rd Workshop on Hydraulics of River Ice

This copy is to provide information prior to publication

STUDY OF RIVER ICE BREAKUP USING HYDROMETRIC STATION RECORDS

by

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MANAGEMENT PERSPECTIVE

River ice breakup may cause floods or costly delays to navigation. Breakup and water levels are a complex combination of meteorological conditions and physical characteristics of the site. Understanding and eventual control depends very much on using historical information which was not obtained to study ice jams. This report is a pilot study to establish the information pertinenent to understanding the phenomena and to providing useful guidance for planning and management.

This report shows that existing data in gauge records of the Water Survey of Canada may be used to obtain useful information which may help in forecasts of future flood levels.

However, before general conclusions can be drawn, other similar studies at sites throughout Canada will be useful if not necessary to obtain progress. It is notable that if the data gathering were to be minimally supplemented that much more could be done with future data records.

T. Milne Dick
Chief, Hydraulics Divison

PERSPECTIVE DE GESTION

Les débâcles fluviales peuvent causer des inondations ou retarder indument la navigation. Les débâcles et les fluctuations de niveau d'eau proviennent d'un ensemble complexe de conditions météorologiques et de caractéristiques physiques du lieu. Leur compréhension et leur contrôle éventuel dépendent étroitement de données historiques qui n'ont pas été recueillies lors d'études d'embâcles. Le présent rapport est une étude pilote visant à établir quelle information est pertinente à la compréhension des phénomènes et peut servir à planifier et à gérer.

Le présent rapport montre que les mesures de jaugeage des Relevés hydrologiques du Canada peuvent servir à obtenir de l'information utile pour prévoir la hauteur des inondations futures.

Or, avant de tirer des conclusions générales, il serait utile, voire nécessaire, d'effectuer des études semblables dans divers endroits du Canada pour faire des progrès. Il est à noter qu'en recueillant un peu plus de données, on pourrait tirer beaucoup plus d'information des mesures futures.

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STUDY OF RIVER ICE BREAKUP USING

HYDROMETRIC STATION RECORDS

S. Beltaos[1]

ABSTRACT

The possibility of using hydrometric station records to extract information related to ice breakup forecasting is explored. Methods for interpretation of the records are outlined and utilized to study breakup characteristics of the Nashwaak River at Durham Bridge, N.B. The results are then compared with recent insitu observations of ice conditions. It is concluded that useful but incomplete information can be extracted from existing records and a need for a theoretical framework of breakup processes is demonstrated. The value of records would be enhanced by collection of additional data such as actual ice thickness; one or more discharge measurements during breakup; and wider utilization of local observers for descriptions of ice conditions.

RÉSUMÉ

Les auteurs se sont penchés sur les possibilitiés d'utiliser les relevés de station hydrométrique pour extraire des données liées à la prévision du déglacement. Ils décrivent leurs méthodes d'interprétation des relevés et ils s'en servent pour étudier les caractéristiques de la débâcle de la Nashwaak à Durham Bridge (N.-B.). Les résultats sont ensuite comparés à de récentes observations sur place des conditions glacielles. Ils en concluent qu'une information utile mais incomplète peut être tirée des relevés disponibles et ils démontrent qu'il serait nécessaire d'élaborer un cadre théorique des processus de déglacement. La valeur des relevés serait augmentée par la collecte de données supplémentaires comme celles qui ont trait à l'épaisseur de la glace, une ou plusieurs mesures du débit pendant la dislocation et finalement, un plus large recours aux observateurs locaux pour la description des conditions glacielles.

[1] Environmental Hydraulics Section, Hydraulics Division, National Water Research Institute, Burlington, Ontario

INTRODUCTION

During the summer 1980 meeting of the N.B. Subcommittee on River Ice (formerly: Ad Hoc Committee on Ice and Ice Jams), a question arose as to whether existing hydrometric station records could be utilized to forecast the onset and severity of river ice breakup. To explore this possibility a joint (NWRI/WSC*) study was initiated for the hydrometric station located on the Nashwaak River at Durham Bridge in New Brunswick. The undertaking of this task was facilitated by the fact that a similar study had been initiated by the writer in late 1979 for Ontario rivers, in co-operation with the WSC Guelph office.

Preliminary results of the Nashwaak River study (Beltaos and Lane 1982) indicated that useful, though incomplete, information can be extracted from existing records. This finding prompted the writer to extend the study to include factors not previously considered and assess the resulting forecast methods using insitu ice observations that have since been performed under the auspices of the N.B. Subcommittee and N.B. Environment. The results to date are reported herein.

RIVER ICE BREAKUP

When an ice-covered river basin is subjected to mild weather, two processes generally begin: increased runoff due to rainfall or snowmelt or both; and heat input to the ice cover. The former process results in increased uplift and frictional forces applied on the ice cover; and in increased water stage which, in turn reduces the support provided to the ice cover by the channel banks and provides increased channel width for movement of the cover. Heat input to the ice cover results in reduced dimensions and strength. It follows that during the mild weather spell, the forces applied on the ice cover increase while the cover's ability to resist these forces decreases. If the mild weather lasts for a sufficient time, the ice cover begins to break up which is often followed by formation of large ice jams, major ice runs and eventual clearance of the ice from the reach of interest. This general description of the breakup process includes two extreme cases, i.e., the "premature" and "overmature" breakup (Deslauriers 1968). Premature breakup occurs under conditions of intense runoff with little, if any, deterioration of the ice cover. Clearly, this type of event has the greatest damage potential, other things being equal. On the other hand, conditions of slow or no runoff with intense ice deterioration lead to overmature breakup. This event is characterized by gradual ice disintegration and has minimal potential for damage.

The first question a forecaster might ask would be how to predict whether and when breakup will be initiated. And once initiated, how

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*NWRI = National Water Research Institute
WSC = Water Survey of Canada

severe it is likely to be in terms of magnitude and duration of ice jam stages at various locations.

Concerning breakup initiation, pertinent literature often advocates use of the corresponding water stage, H_B (= height above an arbitrary datum, e.g. gauge height) as a convenient and meaningful index (Shulyakovskii, 1963; Gerard, 1979; Beltaos, 1981, 1982). From our earlier discussion, it would appear that H_B is indeed a desirable parameter because it reflects the ice driving forces as well as the water surface width available for ice movement. Moreover, the above noted literature suggests that, in a given reach, H_B depends on: the thickness of the ice just prior to breakup, h_i; the degree of ice strength reduction caused by thermal effects; and the stage during freeze up when a stable ice cover forms, H_F. The latter is an index of the width of the ice cover and, excepting mature breakup events, has to be exceeded before contact of the ice with channel boundaries is eliminated. As will be discussed later, approximate values of these parameters can be extracted from gauge records. As for ice strength, there is no direct information. The best that can be done at present is to use a meteorological index intended to describe the effects of thermal deterioration.

With regard to the severity of breakup, one would ideally wish to predict the complete stage hydrograph during the breakup period at any given location. This appears to be too ambitious a task at present; it is thought more practical to limit the goal of the study to forecasts of the maximum stage during breakup, H_m. This stage can be easily identified on gauge recorder charts and is usually caused by a nearby ice jam. Theoretical considerations and field data (Pariset et al, 1966 Beltaos, 1983) have shown that the maximum stage that can be caused by an ice jam occurs when the jam has attained a condition of equilibrium and fully affects the site of interest. This equilibrium stage depends mainly on discharge, channel slope and width. During any one breakup period, H_m may or may not reach the equilibrium jam stage owing to one or more of the following reasons. (a) Ice jam located far downstream of the gauge site. Even if this jam attains equilibrium, the gauge site will experience a fraction of the jam's effect on stage. (b) Ice jam is located far upstream of the gauge site. The gauge site will again experience a fraction of the jam's effect on stage owing to attenuation effects during the jam's release. (c) Unstable jam that releases prior to attaining equilibrium. (d) Overbank flooding. Water and ice spread out onto the flood plain so that the jam's potential is dissipated. This case could be viewed as a particular instance of the unstable jam case. Considering that a major factor contributing to ice jam formation is the original ice cover itself, it is reasonable to expect that not only discharge but also competence of this cover may influence the value of H_m (see also later discussion).

DESCRIPTION OF DATA

The main data source has been the WSC record of gauge height versus time for the period 1965-81. Supplementary information consisted of daily discharge data (WSC); meteorological data (Atmospheric Environment -"Monthly Records"); channel hydraulics in the vicinity of the gauge (B.Burrell, N.B. Environment); and recent ice thickness measurements (P.Tang, N.B. Environment). From these "raw" data, several parameters thought to be characteristic of the ice regime have been extracted as described below.

Maximum Stable Freeze Up Stage (H_F)

A typical but not universal configuration of the daily average stage hydrograph near the start of the ice season is sketched in Figure 1. The solid line represents the actual stage whereas the broken line gives the "effective" stage (=stage that would have occured had the flow been unaffected by ice). At a certain time which may be termed the beginning of freeze up, the actual stage starts to rise while the effective stage continues to drop. Eventually, the actual stage attains a peak and then declines. This sequence reflects the dynamic nature of ice cover formation in rivers. With the onset of cold weather, frazil ice forms and is initially transported freely. The effect of this moving ice on the water stage is small. As more and more frazil is produced, it begins to agglomerate into slush and pancakes. Eventually, the ice transport is impeded somewhere downstream of the gauge (due to border ice growth or other constricting feature) and an ice cover begins to propagate upstream. The presence of the ice cover causes a local stage increase which eventually begins to be "felt" at the gauge site. The gauge height then increases until the time when the edge of the ice cover arrives at the gauge site. Subsequently the gauge height decreases due to decreasing discharge and thermal smoothing of the underside of the cover. The peak stage (H_F) during this period is considered an important factor influencing the succeeding breakup because it defines the stage at which the ice cover is formed; the width of the cover is approximately equal to the channel width at the stage H_F. To eliminate brief peaks during which there is little time for freezing, H_F is defined as a daily average value. It is recognized that this definition of H_F only provides an index for the width of the stable ice cover and could, perhaps be improved by taking an average over a number of days after the peak. While this is a matter that should be investigated in the future, it was considered an unnecessary elaboration for the present exploratory study.

Moreover, it should be kept in mind that the above described freeze up process occurs frequently but not always due to occasional presence of complicating factors, e.g., severe flow and stage controls; incomplete ice cover; very rapid drop in discharge that suppresses occurrence of a peak on the stage hydrograph. Because of these and possibly other unforeseen difficulties, H_F should be determined in conjunction with consultations of prevailing weather conditions and (if available) local observers' reports* while keeping in mind its physical

* At many gauge sites, local observers are temporarily employed by the operating agency to provide brief descriptions of ice conditions at a specified frequency.

significance as outlined earlier. In the present study, interpretation of freeze up stage records presented little difficulty except on a few occasions where H_F determination was designated "uncertain".

"Winter" Peaks.

Occasionally, a brief thaw may occur during the winter period. If such a thaw causes sufficient runoff, the gauge record will show a peak which may or may not initiate breakup. In the latter case, the peak stage represents a lower limit for the stage required to initiate breakup at that time. The term "winter" peak is used conventionally and includes any peak that does not initiate breakup. While such peaks usually occur in the winter, there are instances where "winter" peaks occur a few days before the spring breakup.

Stage at Initiation of Breakup (H_B).

Usually,when a thaw does lead to breakup of the ice cover, the stage begins to rise from its fairly steady winter value and shortly after exhibits spikes and peaks that can only be caused by breaking or broken ice effects (Fig.2). A probable value of the stage at the initiation of breakup, H_B, may be fixed at the first significant spike* . Unfortunately, this definition is not always objective or meaningful. Only a probable range of H_B can then be determined, by considering: (a) the latest time for which it can be confidently assumed that there still was continuous ice cover; and (b) the earliest time for which broken ice effects became evident on the stage hydrograph. Difficulties may be experienced in cases of absence of spikes owing to very rapid stage increases caused by intense runoff or release of upstream ice jams; "misleading" spikes caused by discharge reductions due to upstream jam formation; or "overmature" breakup events where breakup can be initiated during constant or even decreasing stage conditions.

Because of such complications, H_B determination should utilize all supplementary information, e.g., prevailing weather conditions, local observers' reports and prior experience of local ice conditions. For the present study no overmature events were encountered, with the possible exception of the 1964-65 event which has been designated "undefinable". This circumstance compensated somewhat for the lack of local observers' reports that have proved extremely helpful in other studies (Beltaos, Unpublished Data).

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* Initiation of breakup is defined herein as the instant when a sustained movement of the ice cover begins. When the cover is set in motion, the resistance to flow is reduced and the stage should tend to drop thus producing a spike on the stage hydrograph. Sometimes, however, the stage rise may be so steep as to suppress spike appearance. Only a slowdown in the rate of rise would then be evident.

Maximum Breakup Stage (H_m)

This is the maximum stage reached during the breakup period and its determination is straightforward (Fig. 2).

Effective Stage and Maximum Ice Effect on Stage (ΔH_m)

The ice effect on stage is the difference between the actual stage and the effective stage. The time of maximum ice effect can usually be determined by simple inspection (Fig. 2) and does not necessarily coincide with the time of H_m.

Daily average discharge values are estimated by WSC based on interpolations between discharge measurements as well as on such evidence as upstream and tributary flows, runoff and weather conditions, etc. Such estimates may involve considerable error. This has repercussions on the accuracy of the effective stage which is determined by joining daily values plotted at noon of each day. For the Nashwaak R. at Durham Bridge, very little confidence can be placed on discharge estimates during breakup conditions (Beltaos and Lane 1982).

Ice Thickness (h_i)

Ice thickness can be estimated from WSC discharge measurement notes. Such notes give the distance from the water surface to the bottom of the ice which, under free flotation conditions, represents about 92% of the total ice thickness. However, this assumption may or may not be valid depending on whether there is significant bank support of the ice or snow cover which may cause the free water surface to rise above the top of the ice. The presence of a slush deposit under the solid ice may render thickness values completely unreliable because the notes would then show the distance from the water surface to the bottom of the slush. Another source of error may be (unreported) instances when "water surface" has been used nominally, i.e., substituted by a more convenient datum such as the top of a deep snow layer.

Usually, a few ice thickness values will be available during any one winter season. These can be plotted versus time and extrapolated to the start of the mild weather spell that led to breakup. Where the winter season involves highly variable weather conditions, it may be preferable to extrapolate using a more complex correlation, e.g., h_i versus accumulated degree-days of frost. Such procedures would generally give fair indications of h_i at the time breakup starts but ignore thickness reductions that may occur during the pre-breakup period (onset of mild weather spell to onset of breakup). This assumption is considered adequate for the present in view of (a) the crudeness of the other data involved; and (b) the partial accounting of this effect by introducing a meteorological index of heat input to the ice cover.

Meteorological Index of Ice Strength

Few data on ice strength at the time of breakup are available and the manner of ice strength reduction by thermal effects is not well understood at present (Frankenstein, 1961; Korzhavin, 1971; Butyagin, 1972). In general, it is reasonable to expect that ice strength will decrease with increasing amounts of heat absorbed by the ice cover but there is no consensus on the most appropriate index for the latter. A very simple and well known index is the accumulated degree-days of thaw, S_T (see for example, Williams, 1965; Bilello, 1980). However, S_T can only be satisfactory in cases where the time of year when breakup occurs and the number of "thawing" days do not vary appreciably. Otherwise, the important effect of solar radiation will not be considered. For example, a sunny day in April would be much more effective in weakening the ice than a cloudy day of the same average air temperature in January. To fully account for thermal effects on ice strength, several parameters are needed in addition to air temperature, e.g., short wave radiation, cloudiness, wind speed, water temperature, snow cover, ice composition, etc.

Unfortunately, not all of this information is usually available and even if it were, it would be impractical to attempt multiple correlations with so many parameters. Shulyakovskii (1963) suggested the use of a calculated value of heat input to the ice cover from the surface, thus ignoring heat transfer from the water since water temperature is, as a rule, unknown. A similar but somewhat simplified approach was suggested by Williams (1965). Bulatov (1972) outlined a method for computing ice strength based on theoretical and experimental correlations with radiation effects. However, Bulatov's paper was too general to permit application of his method by others. Ashton's (1983) analysis is similar to Bulatov's and shows that the main agent of deterioration is the penetrating solar radiation, once the ice has been warmed to 0°C. Additional radiation absorption causes melting at the grain boundaries with a resulting decrease in strength. However, Ashton's analysis cannot be applied to the data under consideration because information on snow cover, albedo and ice structure is lacking.

Evidently, only empirical indices of ice strength can be employed at present. Some of the simplest ones are accumulated degree-days of thaw, hours of sunshine and solar radiation but their simultaneous consideration would complicate the analysis. Shulyakovskii's (1963) single heat input parameter, $\sum q$, has the advantage of simplicity as well as a background of practical usage and was thus utilized by Beltaos and Lane (1982). However, there is no theoretical evidence that this parameter adequately accounts for the qualitatively different effects on ice strength of the various heat components involved.

ANALYSIS OF DATA

Table 1 summarizes the data for the Nashwaak R. at Durham Bridge (Fig.3). Of the 21 freeze up - breakup events that occurred during the period of record (1965 -81), one has proved impossible to interpret, while six presented serious difficulties. At the time of writing the report by Beltaos and Lane (1982), only a few ice thickness values were available and thus no attempt was made to consider h_i in the analysis. Subsequently, additional ice thickness data were made available to the writer by P. Tang of N.B. Environment which enabled determination of h_i for many of the events under consideration. For events without any thickness measurements h_i was estimated via a correlation between measured values and time from H_F. This procedure involves errors as large as 30%. Water surface to bottom of ice distances quoted by WSC have been divided by 0.92 to obtain h_i though this is recognized to be a first approximation. $\sum q$ values quoted by Beltaos and Lane (1982) have been revised to account for daily variations of associated heat input coefficients but only in a few instances did this result in substantial changes.

Initiation of Breakup.

Beltaos and Lane's analysis (1982) followed Shulyakovskii (1963), after some initial verifications of the basic premises. First, H_B was plotted versus H_F where a trend for H_B to increase with H_F was indicated. However, there was considerable scatter suggesting additional effects. Next, the difference (H_B - H_F) was postulated to depend on h_i and $\sum q$, the total amount of heat input to the ice cover per unit surface area. The latter is an accumulation of daily heat fluxes (q) during daylight hours until the time of breakup initiation; heat transfer from the water is ignored. Calculation of $\sum q$ involves many simplifying assumptions so that $\sum q$ must be viewed as a mere index of the true amount of absorbed heat (see Beltaos and Lane 1982 for details of the calculation). A plot of (H_B - H_F) versus $\sum q$ indicated the expected trend but exhibited considerable scatter. To explore the possible effects of h_i, the following procedure was adopted. First, (H_B - H_F) was plotted versus h_i by noting the value of $\sum q$ beside each data point. This indicated an increase of (H_B - H_F) with h_i and a decrease with $\sum q$. The upper envelope of the data points, assumed representative of the case $\sum q \approx 0$, was then described by the straightline $(H_B - H_F) \approx 2.5\ h_i$*. Next, the deviation of any one data point from the upper envelope [$= 2.5\ h_i - (H_B - H_F)$] was computed and plotted versus $\sum q$, as shown in Fig. 4. Data ranges in Fig. 4 indicate instances where only ranges for H_B could be identified; vertical ranges indicate cases where lower and upper limits of H_B occurred within a short time period so that the corresponding $\sum q$'s were nearly equal. Data points with arrows denote winter peaks or otherwise known limits for H_B; such points are occasionally of little value (e.g., two uppermost points at $\sum q = 0$) but often give useful indications as to how a correlation line should be drawn.

Fig. 4 confirms the anticipated trends but with considerable scatter. The latter can be partly attributed to the crudeness of H_B determinations (no local observers' reports) and the empiricism introduced in the analysis (lack of a theoretical framework for breakup processes). A compensating feature is that even a large error in predicting H_B usually translates to acceptable error in forecasting

* Linear plots of this kind have also been found by the writer at other sites but with different numerical coefficients (Beltaos, unpublished data).

the time of H_B because the prevailing temporal gradients of stage are usually large.

It may be noticed in Fig. 4 that two data ranges are plotted for the 1979 event, designated (1) and (2). The former reflects the interpretation given by Beltaos and Lane (1982) and involves serious uncertainty; it plots far off the band of the other data. Later on, it was discovered (P. Tang, personal communication) that a site visit by WSC staff in March 1979 indicated the presence of intact ice cover which dictated the following revision. What was originally thought to have been breakup initiation was in fact a winter peak whereas breakup was initiated later in March. The event designated 1979(2) reflects the new interpretation and plots at a much improved position. This result illustrates the importance of local observers' reports.

Maximum Breakup Stage

As discussed earlier, flow discharge is a major factor influencing H_m. However, discharge data for the Nasnwaak River study are uncertain so that the plot of Fig. 5, showing H_m' (=H_m - stage at zero discharge) versus prevailing discharge is of qualitative value. It is noted that some of the data points in Fig. 5 represent conditions of maximum ice effect, ΔH_m, in instances where the latter did not occur at the same time as did H_m. Also plotted in Fig. 5 is the theoretical relationship between equilibrium jam stage and discharge for comparison (Beltaos 1983). The latter is seen to provide a satisfactory upper envelope up to a certain discharge, but to consistently overpredict the stage beyond this discharge. This is a typical trend, reflecting the fact that increasing discharge reduces the probability of equilibrium jam formation (Beltaos 1983). For practical purposes, an upper envelope of the data points could be drawn and used to forecast potential H_m values. Whether and how closely the potential H_m is to be realized in a given season depends on the number and stability of ice jams that form near the gauge site, as discussed earlier. In turn, such effects are controlled by channel and floodplain configuration as well as the competence of the ice cover during breakup. The former factor is difficult to assess at present because the behaviour of ice jams is unknown once the bankfull stage is exceeded (see also Calkins 1983). On the other hand, experience suggests that the competence of an ice cover should be an important factor influencing H_m and this possibility is considered next.

Since the competence of an ice cover can be defined in terms of its strength, thickness and width, it may be of interest to explore $\sum q$, h_i, and H_F (rough index of ice cover width) as possible factors influencing H_m. Fig. 6 shows H_m plotted versus H_F. The data points define an upper envelope that increases with H_F. The deviation of the observed value of H_m from the corresponding upper envelope value is plotted versus $\sum q$ in Figure 7. This results in another upper envelope that confirms the anticipated trend. It thus appears that H_F and $\sum q$ define a potential or, an upper limit for, H_m. Whether and how closely this potential will be realized in any one breakup event, depends on a number of other factors, e.g., discharge, local jamming conditions, etc. One would expect that h_i should also be relevant here but, owing to discharge uncertainties, this possibility has not been investigated herein, though h_i effects on H_m have been discerned elsewhere (Beltaos, Unpublished Data). Moreover, it is noted that strictly speaking, $\sum q$ should be calculated to the time of H_m. However, the value used in Fig. 7 applies to the time of H_B. This was thought sufficient given that the present study was exploratory.

Frequency of Occurrence of H_m

A simple frequency analysis on H_m values was also performed by Beltaos and Lane (1982). The fact that, occasionally, there have been two breakup events in the same season was ignored and all events were assumed to be independent so as to increase the effective length of record. This may or may not be valid but more data are needed to clarify this point. For the present study, it was found that the above approach resulted in a frequency curve that differed very little from the one obtained by use of only the highest breakup stage in any one season.

For convenience of plotting, use has been made of, $H_m' = H_m$ - stage at zero discharge. In this manner, the event $H_m' \geq 0$ has a probability of 1. After performing the frequency analysis, H_m' can be plotted versus probability on various types of charts as a means of exploring the mathematical form of the H_m' distribution. Gerard and Karpuk (1979) suggested that the log-normal distribution is a possible candidate and found a linear relationship after plotting their data on log-normal probability paper. Figure 8 shows that only in the range $0.1 \leq P \leq 0.9$ do the present data adhere to a linear relationship.

Limitations

Clearly, the results presented so far are site-specific and empirical. Therefore, extrapolation to other sites or hydrometeorological conditions different from those covered by the years of record is not justified. Accumulation and comparison of several case studies such as the present would facilitate development of more general forecasting methods.

COMPARISON WITH OBSERVATIONS

Since 1981, ice conditions in the Nashwaak River near Durham Bridge are monitored under the auspices of the N.B. Subcommittee on River Ice and N.B. Environment. The results of the field observations are used in this section to assess the effectiveness of the relationships derived so far.

1981-82 Event

Ice effects on stage commenced on Dec. 26, 1981, and a value of 2.18 m was chosen for H_F on Dec. 28. Breakup was initiated at about 1100 h on Apr. 1, 1981, with H_B = 2.50 m and H_m = 3.00 m occurring at 1800 h on Apr. 3. From measurements, h_i was estimated as 0.61 m and $\sum q$ was calculated as 5784 J/cm^2. The quantity 2.5 h_i - $(H_B - H_F)$ is 1.21 m and inspection of Fig. 4 indicates that the data point for this event would not fit the trend defined by the historical data. To explain this discrepancy, a close examination of the weather records was undertaken and revealed a highly atypical sequence of events: A warming trend began on Mar. 11 and continued until Mar. 20. Subsequently, the weather turned cold but q values remained positive, excepting the dates Mar. 22, 27, 28 and 29. A total of 15 cm of snow fell during the period Mar. 19-22. A continuous warm trend began on Mar. 30 and led to breakup. Between Mar. 11 and 29, a net of 34.3°C - days of frost was accumulated. This sequence of events suggests that sustained thermal ice deterioration would have started on Mar. 30 even though the value of $\sum q$ up to Mar. 29 was 3789 J/cm^2. This illustrates a shortcoming of Shulyakovskii's $\sum q$ calculation. The latter only accounts for heat exchange during daylight hours and would thus seriously underestimate recovery of ice strength during a cold spell that intervenes between two warm ones. If $\sum q$ were accumulated from Mar. 30 on, a value of 1995 J/cm^2 would be obtained. This would improve the plotting position of the 1981-82 event in Fig. 4. However, such a correction involves a measure of arbitrariness and the writer cannot see how to improve this situation without resort to a theoretical model of ice deterioration. Though some research has been done in this regard (Bulatov 1972; Ashton 1983), it has not advanced to the point where it can be applied in practice. For the present, it can only be hoped that the forecaster would recognize atypical events and make necessary allowances based on experience.

With $\sum q$ = 5784, Fig. 7 indicates that the quantity $(H_m - 1.22 - 1.18H_F)$ should not exceed -0.68 m which gives $H_m < 3.11$ m, as was the case (H_m = 3.00 m). If $\sum q$ were taken as 1995 J/cm^2, Fig. 7 would have given $H_m < 3.63$ m. In cases where reliable discharge data are available, a plot such as Fig. 5 could also be used to improve forecasts of the potential H_m value. However, this is not possible in the present study owing to the serious uncertainties associated with breakup discharges.

1982-83 Events

Ice effects on stage commenced on Dec. 13, 1982, while a value of 2.50 m was chosen for H_F on Dec. 19. A mild weather spell in January led to breakup with H_B = 2.65 m at 0900 h on Jan. 12 and H_m = 3.83 m at 1500 h on Jan. 12. The values of h_i and $\sum q$ are estimated as 0.24 m and 375 J/cm^2 respectively. It follows that 2.5 h_i - $(H_B - H_F)$ = 0.45 m and this would plot satisfactorily in Fig. 4. Use of Fig. 7 gives $H_m < 4.24$ m, as was the case (H_m = 3.83 m).

The peak stage during the January event was caused by a local ice jam that did not release but froze in place as cold weather resumed. A new H_F of 2.55 m occurred on Jan. 16. Breakup was initiated at 1000 h on Mar. 22 with H_B = 3.30 m, $\sum q$ = 3669 J/cm^2 and H_m = 3.97 m at 1800 h on Mar. 22. No ice thickness measurements are available for this event, hence h_i can only be estimated, as follows. If other years' experience is used and the presence of the frozen jam is ignored, h_i would be estimated as 0.61 m. On the other hand, the thickness of the jam at the time it formed is estimated to have been about 1.2 m, (see for example, Beltaos 1983). Calkins (1979) has shown that ice growth is accelerated in the presence of a porous ice accumulation under the lower boundary of a solid ice cover. If, as a first approximation, h_i is assumed to increase as the square-root of degree-days of frost, then a factor of $\sqrt{1/p}$ should be applied to the normally expected ice thickness (p = porosity). For $p = 0.4$, this gives $h_i = 0.61/\sqrt{0.4} = 0.96$ m. With this, the value of $2.5\ h_i - (H_B - H_F)$ becomes 1.66 m which would plot satisfactorily in Fig. 4. Use of Fig. 7 gives $H_m < 3.83$ m. The observed H_m was 3.97 m, i.e., 0.14 m higher than would have been thought possible from the historical data. This is very likely due to the extremely thick ice cover caused by the freezing of the January jam.

CONCLUSIONS

The present results indicate that useful though incomplete information can be extracted from existing gauge records. This information can be utilized in forecasting the onset and potential severity of breakup, subject to the limitations outlined next.

The present analysis is empirical and site-specific; hence, it cannot be extrapolated to other sites or to hydrometeorological conditions that are not covered by the years of record. While studies similar to the present can be used as an aid to forecasting, it was shown that some reliance on experience would be necessary for unusual events. The lack of a theoretical framework for breakup processes is considered a major obstacle to eliminating empiricism from pertinent forecasting methods. Accumulation and comparison of additional case studies would contribute toward this goal.

As a by-product of this study, several instances were identified where moderate increases of the gauge operation effort would greatly increase the value of records for breakup-related studies. These include measurement of the true ice thickness and, where applicable, delineation between solid and slush ice layers; wider utilization of local observers and increase of reporting frequency during freezeup and breakup; and performing one or more discharge measurements during breakup events.

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Table 1. Summary of Breakup Characteristics

Season	H_F (m)	H_B (m)	H_m (m)	h_i (cm) From Measurements	h_i (cm) Estimated ±30%	$\sum q$ (J/cm^2)	Remarks
1964-65	2.56	>2.20	NA		55	2125	Breakup undefinable
1965-66	1.40	1.58	2.23		73	5420	
1966-67	1.80	1.71	1.73	70		5944	
1967-68	2.34	3.25	3.87		43	425	
-"-	3.35	>2.78	NA		52	3748	
-"-	3.35	2.71	3.40		52	6246	
1968-69	2.25	>1.84	NA		38	0	
-"-	2.25	1.87	1.87		74	4240	H_B uncertain
1969-70	3.44	1.98-5.31	5.31		41	511	H_B uncertain
1969-70	2.29	1.43-1.80	2.05	37		4782	H_B uncertain
1970-71	1.65	>2.07	NA	40		197	
-"-	1.65	>1.83	NA	64		2119	
-"-	1.65	0.90-1.31	NA	64		6819-7359	
1971-72	1.19	>1.34	NA	18		251	
-"-	1.19	>1.43	NA	50		0	
-"-	1.19	>1.59	NA	60		1370	
-"-	1.19	>1.74	NA	79		0	
-"-	1.19	1.62-2.19	2.49	79		978-1467	
1972-73	2.72	>2.79	NA		50	241	
-"-	2.72	>2.16	NA		60	3251	
-"-	2.72	1.91-2.35	2.61		60	5294	
1973-74	2.20	>2.22	NA		75	2900	
-"-	2.20	1.56-2.19	2.19		76	5353-7162	
1974-75	2.19	>2.15	NA		70	2205	
-"-	2.19	1.48-1.68	1.68		70	4642-6267	H_B uncertain
1975-76	2.44	>3.38	NA		59	338	
-"-	2.44	2.19	2.94	73		3933	
1976-77	1.89	>2.69	NA	30		0	H_F and H_B uncertain
-"-	1.89	>1.91	NA		70	3897	H_F = 2.29 m
-"-	1.89	1.49-1.53	2.29		72	7442	might be better
1977-78	1.82	2.15-2.51	2.51		25	21	
-"-	2.74	1.42-1.54	1.76	64		8176	H_F uncertain
1978-79	1.10	<1.93	2.57	30		293	
-"-	3.40	>3.26	NA	34		1074	
-"-	3.40	>3.12	NA	61		2650	
-"-	3.40	2.07-2.56	3.20	61		6234-6719	
1979-80	2.25	2.71-3.01	3.03	55		658	
1980-81	2.41	1.81-2.11	2.12	52		736-2542	

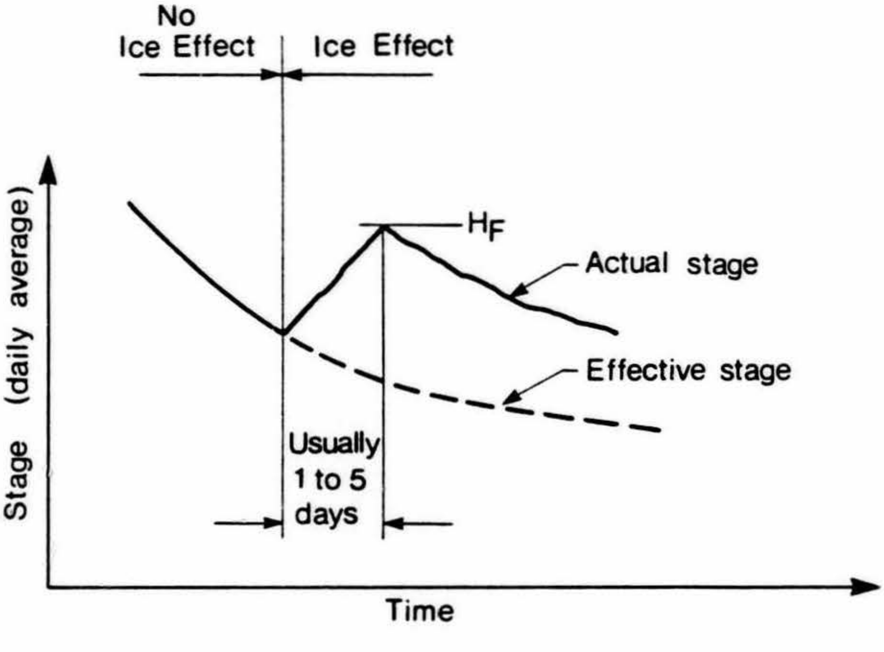

Fig. 1 Schematic illustration of daily stage variation with time during beginning of freeze up.

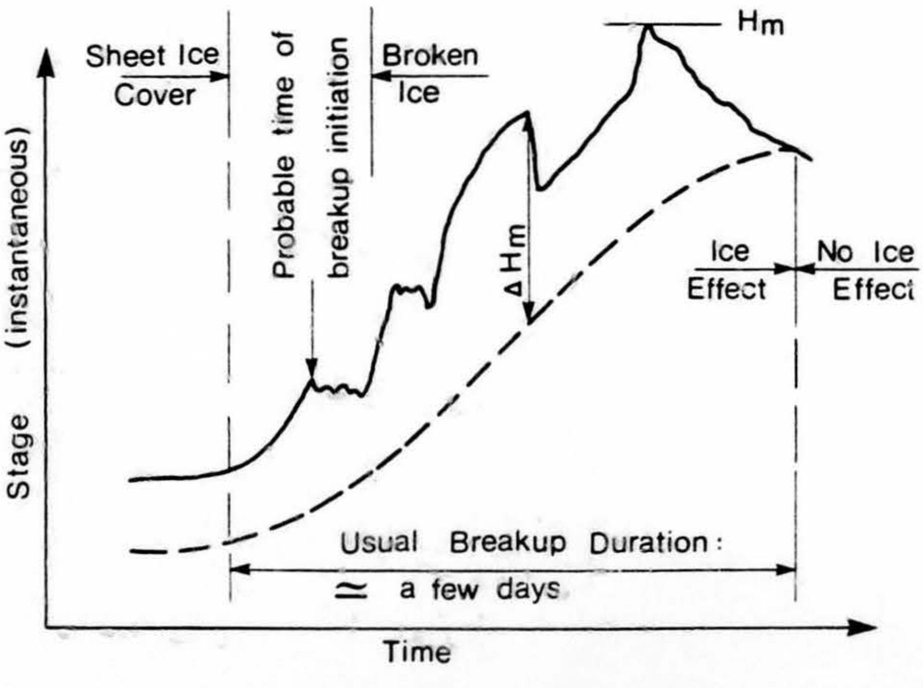

Fig. 2 Schematic illustration of instantaneous stage variation with time during breakup.

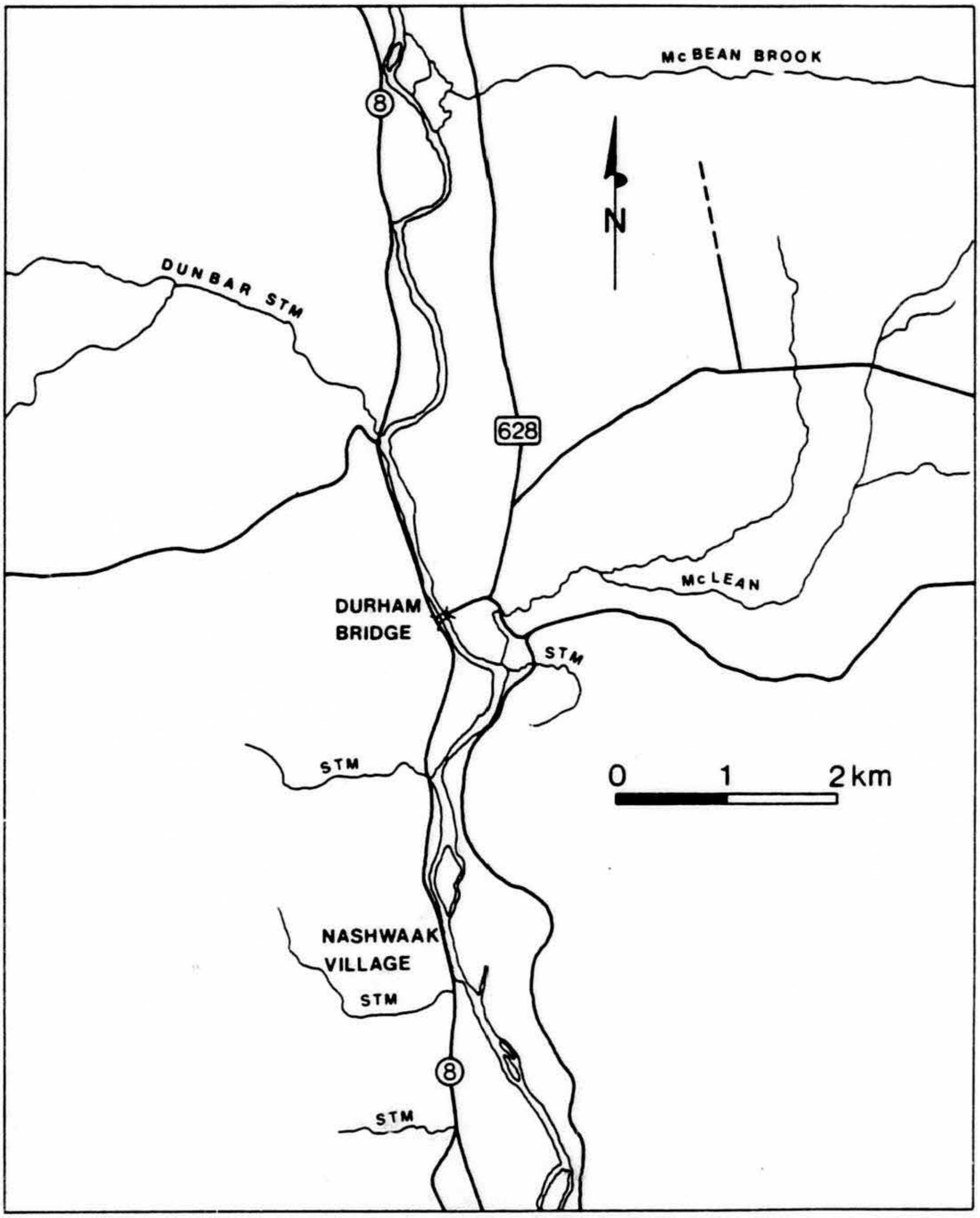

Fig. 3. Plan of Nashwaak River in the vicinity of Durham Bridge.

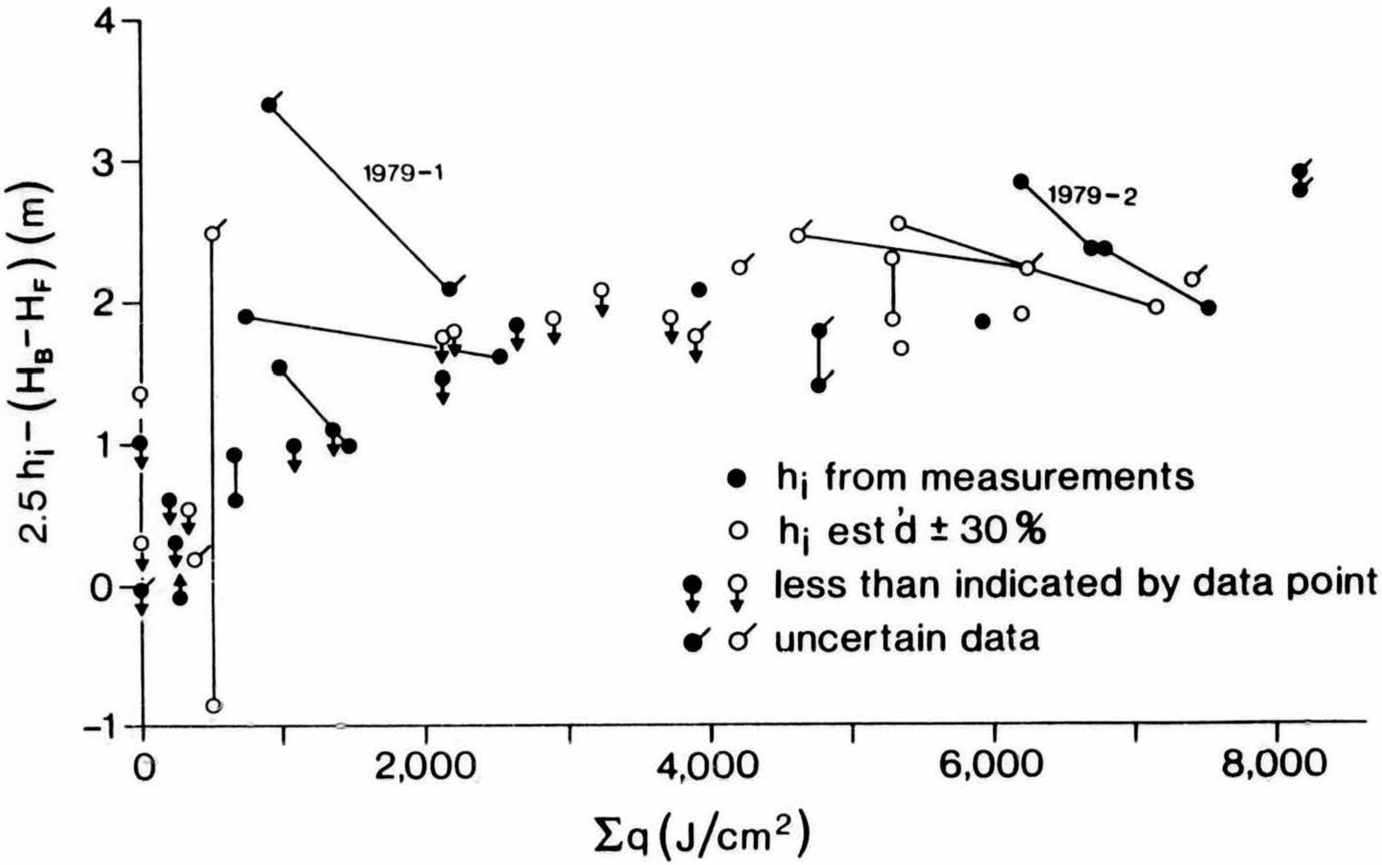

Fig. 4. Variation of $2.5h_i - (H_B - H_F)$ with Σ_q

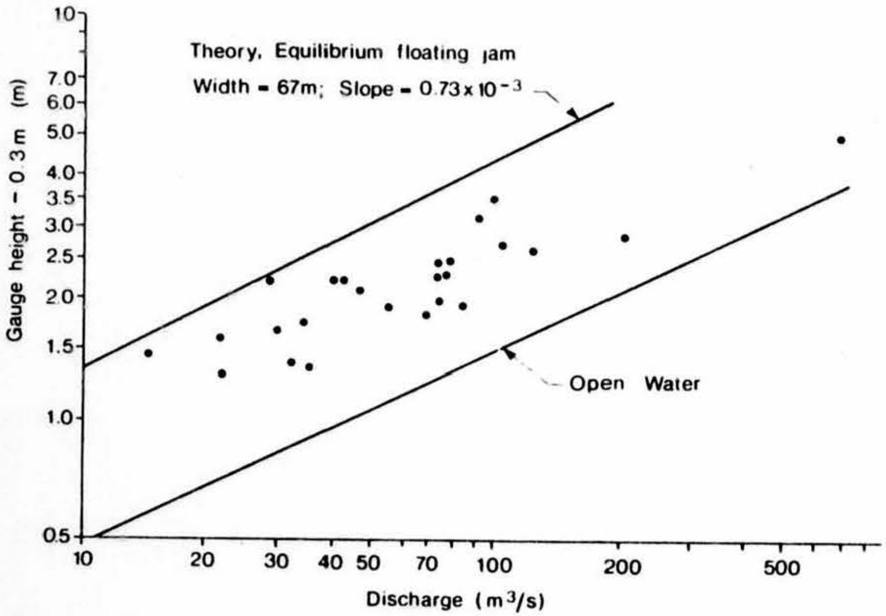

Fig. 5. Effect of discharge on breakup stage.

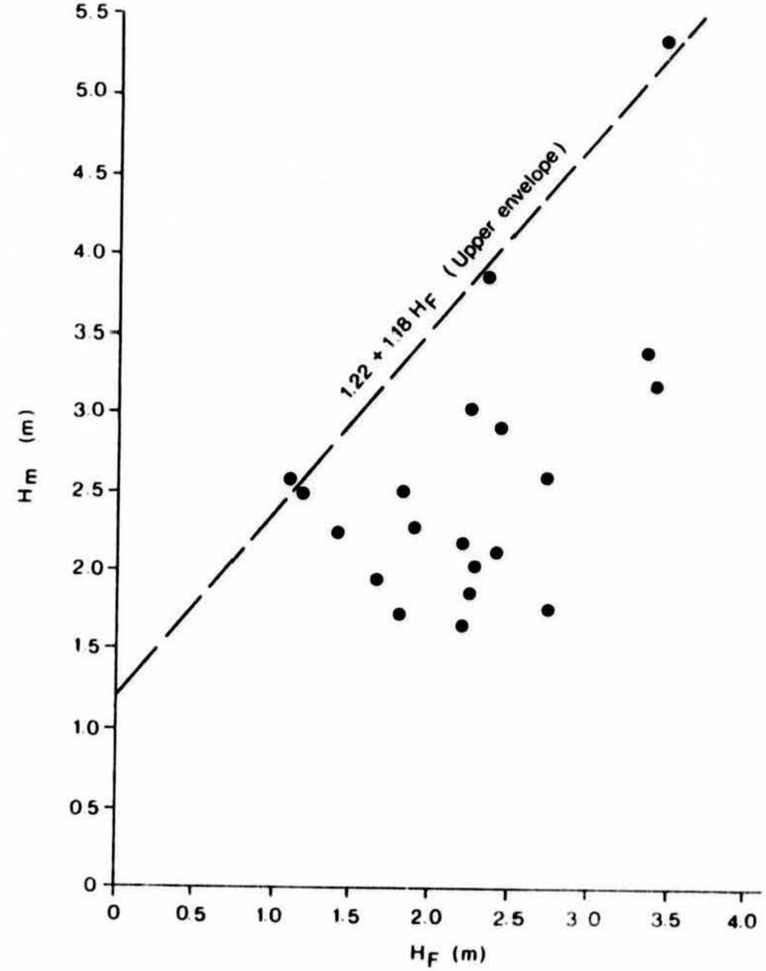

Fig. 6. Maximum stage during breakup versus H_F.

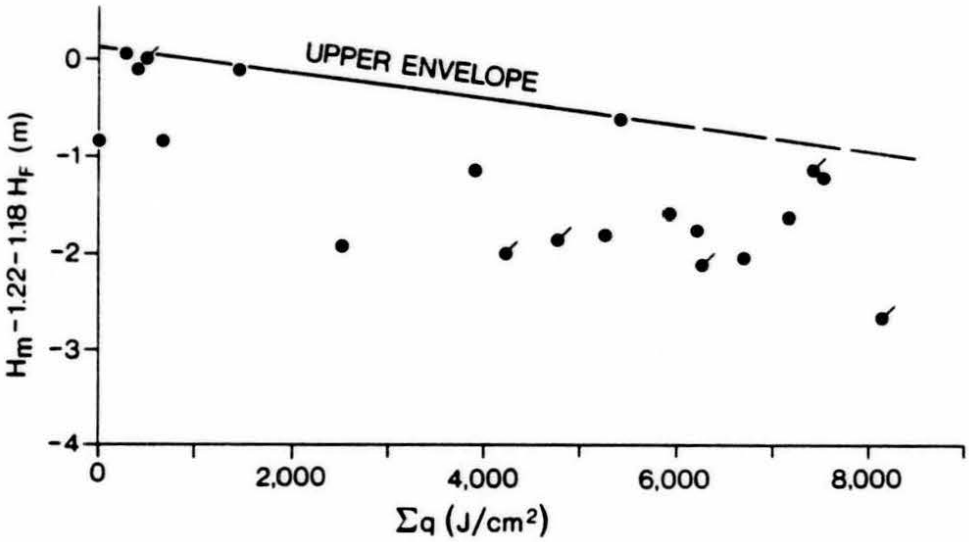

Fig. 7. Effect of Σ_q on H_m; legend same as for Fig. 4.

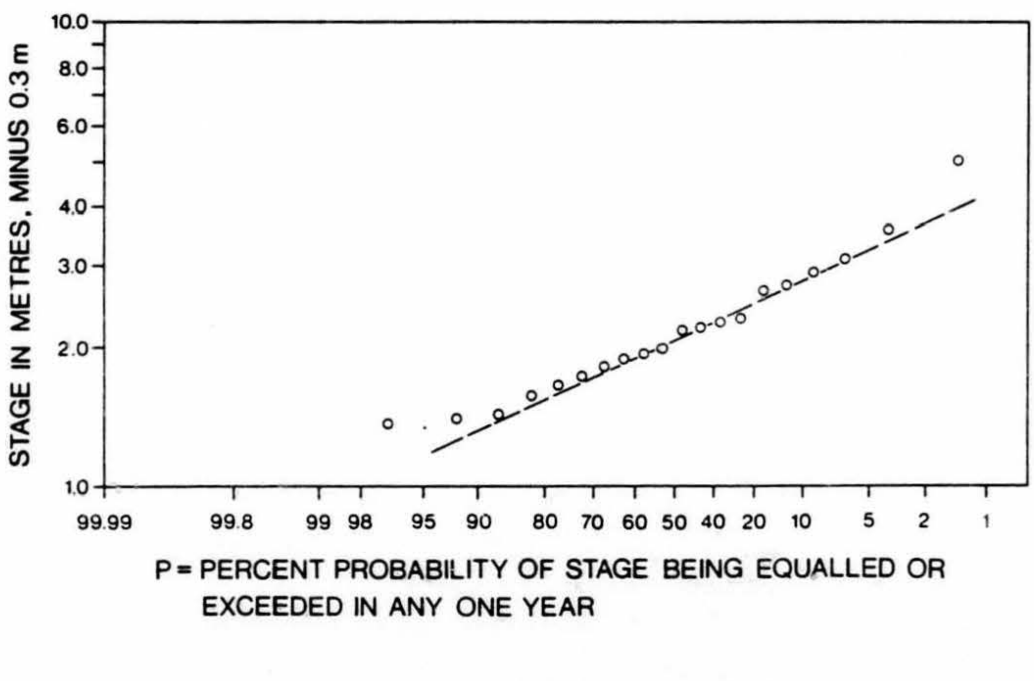

Fig. 8. Frequency curve of breakup peak stage.

Beltaos (25)

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RIVER ICE BREAKUP

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RIVER ICE BREAKUP

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Ice breakup is an important event in the regime of northern rivers mainly due to ice jamming and associated problems, e.g., flooding, forces on structures and erosion. Breakup is triggered by mild weather and the attendant increases in runoff and heat input to the ice. At present, the onset of breakup can only be forecast empirically, using site-specific historical records. Recent work has produced partial understanding of the early phases of breakup (formation of hinge and transverse cracks, first movement of the ice) but more observational data are needed for a complete model of breakup initiation. When set in motion, the ice quickly breaks down into small fragments and ice jams begin to form. Subsequent developments are highly disorderly owing to the multitude of factors that are at work, e.g., hydrologic, geomorphic, structural. Present understanding of ice jam initiation and evolution is poor and theoretical jam models have to date focused on equilibrium conditions. Their practical utility is thus restricted to forecasting potential high water levels that may or may not be realized during breakup. Progress in this regard requires consideration of the breakability of the ice cover and its effects on jam formation and release.

INTRODUCTION

Ice breakup is a relatively brief but very important event in the regime of northern rivers. Annual peak stages often occur during breakup, owing to formation of major ice jams. The result is frequent flooding with the added inconvenience of ice on the flood plains (Fig. 1). Moving ice during breakup can apply large loads on bridge piers and similar structures, or cause damaging piles on river banks and islands. Sudden releases of major ice jams can result in rapidly rising water levels and extreme water speeds with possible consequences to channel erosion. Avoidance of premature breakup imposes serious constraints to hydropower production during the winter. Man-induced changes in the hydrologic regime of a river can have significant environmental impact because of consequent changes in the ice regime and especially in breakup characteristics.

Jam and flooding at mouth of Thames R. (Ont.) Feb. 1981.

Remnants of jam on flood plain of Credit R. (Ont.) Mar. 1980.

Toe of a jam in Smoky R. (Alberta) Apr. 1976. Note leads in intact ice cover.

Shear wall on bank of Smoky R. (Alberta), formed by release of a jam, Apr. 1976. Est'd height = 5 m

Fig. 1. Illustrations of ice jams and their effects

Despite its importance, river ice breakup remains largely intractable from the viewpoint of hydraulic engineering. There is little guidance, other than historical information and experience, with regard to: short-term forecasting and warning of the onset and severity of breakup or long-term forecasting of peak breakup water levels and flood risk assessments; evaluation of the impact of river structures; and design criteria to prevent or control some of the consequences of breakup. This state of affairs is partly due to the multitude of factors that influence breakup and to the complexities of ice jamming phenomena. At the same time, it must be admitted that the ice regime of rivers has received relatively little attention even in regions where ice is present for significant portions of the year.

A brief account of existing knowledge of breakup is given in this paper and an attempt is made to identify gaps that seriously hamper progress.

DESCRIPTION OF BREAKUP PROCESSES

When the basin of an ice-covered river is subjected to mild weather, two major processes begin: increased runoff due to snowmelt or rain, or both; and increased heat input to the cover. The former process results in increased discharge with consequent increases in the uplift and frictional forces applied on the cover. The water stage also increases and this reduces the contact areas between the ice cover and the channel boundaries and provides increased channel width for movement of the ice. Heat input to the cover results in reduced dimensions and strength. Thus, mild weather causes an increase in the forces that are applied on the cover while the latter's ability to resist these forces and remain stationary is reduced. Eventually, this process leads to movement and breakup of the cover which is often followed by large ice jams, major ice runs and eventual clearance of the ice.

The following quotation from Shulyakovskii (1963) is a good, though not universally applicable, description of the early stages of breakup.

> "On partially freezing rivers the destruction of the ice cover when the water stage rises, begins with the formation of cracks and the separation of the ice from the river banks or from the shore ice. Cracks in the ice cover form in this case not only along the banks, but also

across the river and at various angles to the banks. This is due to the nonuniform thickness and strength of the ice cover and to the nonuniform strength with which it is attached to the banks. As the water discharge further increases, the ice cover continues to rise and ice flanges form. At the same time the melting of the snow on the ice and of the ice cover itself continues. The strength of the ice decreases, mainly due to the penetration of solar radiation. The ice cover also melts and is washed out owing to the flow of water around it. If the ice on tributaries breaks up earlier, the integrity of the ice cover is often upset. When the rising water stage reaches a certain limit, corresponding to the nature of the river bed on the given stretch, to the thickness and state of the ice cover, an ice push occurs."

Once large ice floes and sheets are set in motion, they quickly break down into smaller fragments due to impacts with channel boundaries or other floes. Where the downstream movement of the fragments is impeded, jams begin to form, sometimes attaining very large dimensions. The water level rises to accommodate the submerged portion of the jams' thickness and large hydraulic resistance of their underside. Continued thermal deterioration or increasing discharge may cause ice jams to release and surge-like phenomena to occur. Such surges may trigger breakup at downstream locations and, if arrested, new jams may form or join existing ones. While such events are in progress, hydro-thermal processes intensify by increasing water temperatures (due to increasing open water area). Eventually broken or intact ice are so far downstream of a given site that their effects on local stage become negligible and breakup can be considered complete at this location.

The preceding description implies the possibility of two extreme cases, the "premature" and "overmature" events (Deslauriers, 1968). Premature breakup occurs under conditions of intense runoff with little thermal ice deterioration and has the greatest damage potential, other things being equal. This type of breakup is common in moderately cold regions (e.g., S. Ontario) where brief "thaws" accompanied by intense rainfalls occur often during the winter. The ensuing breakups are brief but violent. In addition, it is possible that cold weather resumes while ice jams are still in place. This may lead to renewed freeze up and relatively thick ice formation in the sections where the jams had been; the damage potential of the next breakup is thus enhanced. Overmature breakup occurs under conditions of slow or no runoff with intense thermal ice weakening. The ice cover disintegrates gradually and jamming is inconsequential. A common type of overmature breakup involves the (relatively) orderly advance of a breakup "front". This is a very short jam that forms upstream of stationary sheet ice. Thermal and mechanical action causes the stationary ice to develop open leads and cracks near the toe of the jam. Deterioration continues until the jam is able to move into the leads where it comes to a temporary halt and the process is repeated. A characteristic of this process is that the jam does not lengthen which implies that melting is one of the governing factors.

The downstream motion of water introduces a similar bias in the direction of breakup advance but this must be understood to apply in a general and "average" sense. Orderly, downstream progression of breakup is the exception rather than the rule. Local hydraulic and geomorphic conditions, tributaries, weather patterns and freeze up conditions often combine to eliminate any semblance of order in the progress of breakup.

Based on the above discussion, the factors that affect breakup can be summarized as follows:

- hydrologic (discharge hydrograph)
- hydraulic (flow velocities, depths, shear stresses)
- geomorphic (channel width and plan geometry)
- meteorological (weather conditions and heat transfer)
- antecedent conditions (freeze up and winter).

Given the large number of pertinent factors, it is easy to appreciate the complexity and variability of breakup phenomena. Not only does breakup differ from one site to another but it can change character from year to year at the same site.

INITIATION OF BREAKUP

A major practical requirement is to forecast the onset of breakup because this event usually heralds the period of ice jamming and the attendant problems. Breakup initiation is not necessarily an abrupt and well-defined event; often, it is a succession of phases, leading from the condition of intact and stationary ice cover to that of moving or jammed ice fragments. Moreover, it should be kept in mind that the breakup process can be arrested at any time if cold weather resumes and flow discharge begins to decrease. A convenient, though not always meaningful, definition of breakup initiation is the time when the first sustained movement of the ice cover takes place.

Using this definition and confining discussion to nonovermature events, empirical work (Shulyakovskii, 1963; Beltaos, 1984a) suggests that breakup initiation can be roughly forecast in terms of the prevailing water stage, H_B. At a given site, this stage depends primarily on H_F (= stage at formation of a stable ice cover during freeze up); h_i (= ice thickness); and competence of the ice cover. The latter parameter is difficult to quantify at present though theoretical work has shown it to be strongly dependent on the penetrating solar radiation and crystal structure of the ice (Bulatov, 1972; Ashton, 1983). Empirical, site-specific, correlations are useful where good historical records are available but cannot be extrapolated to sites with no records. An attempt to generalize empirical forecasting techniques was presented by Margolin (1980) but this, too, requires historical (and not usually available) information.

Lack of theoretical models of breakup processes is an obstacle to progress in forecasting the onset of breakup. In recent years, however, tangible advances in our understanding of the early stages of breakup have been made and are reviewed next, even though they are not sufficiently detailed to allow complete description of the problem.

Formation of longitudinal cracks

During the winter, when the discharge is fairly steady, the ice cover is, for the most part, in a condition of free flotation. If a prismatic channel with uniform flow and ice thickness is assumed for simplicity, then the longitudinal pressure gradient should be zero. The flow is driven by gravity, much as in an open channel. An increase in discharge while the ice cover remains firmly attached to the channel boundaries can only occur if a pressure gradient develops. The result is that an uplift pressure is applied on the underside of the cover in addition to that required to keep the cover floating. Structural considerations indicate that the cover may then be considered a plate supported by an elastic foundation whose modulus is equal to γ, the unit weight of water (Hetenyi, 1946). A further simplification can be made by assuming that the pressure gradient is small enough to allow a two-dimensional analysis, i.e., to view the ice cover not as a plate but as a series of beams of unit width that do not interfere with each other.

Billfalk (1981) considered this problem for the case of an infinitely wide channel, assuming elastic response of the cover; measurements showed good agreement with predictions. The solution was extended to finite channel widths (Beltaos, 1984b) and the results are depicted in Figs. 2 and 3 (W = beam length = cover width; ℓ_s = distance of a crack from the respective edge; h_i = ice cover thickness; σ_i = flexural strength of ice; ΔH = uplift pressure head).

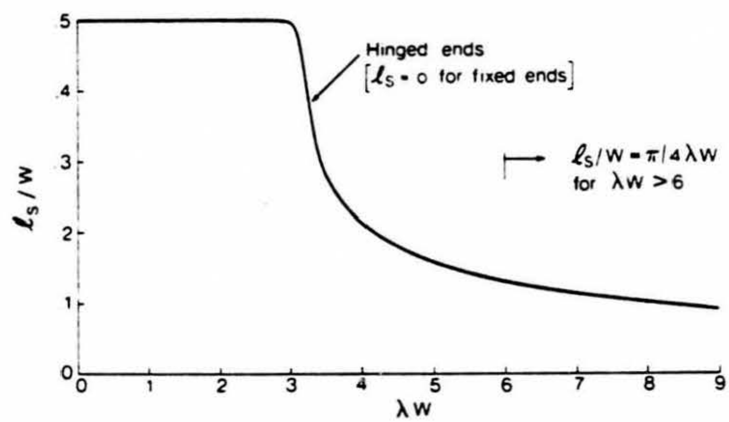

Fig. 2. Location of longitudinal cracks

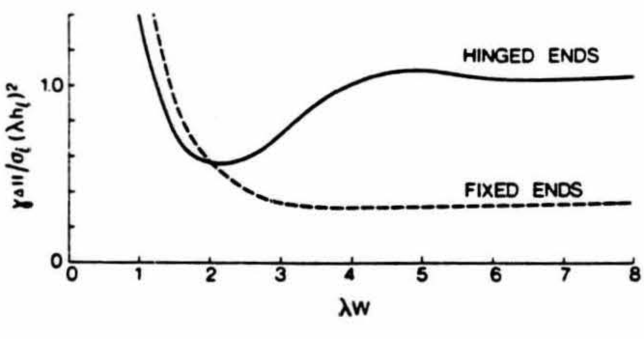

Fig. 3. Dimensionless uplift pressure head required to cause longitudinal cracking

The parameter λ is defined by

$$\lambda = (\gamma/4E_i\ I)^{1/4} \tag{1}$$

with E_i = elastic modulus of ice; and I = moment of inertia per unit cover width = $h_i^3/12$ (note $1/\lambda$ has the dimension of length). The type of edge support assumed for the ice cover has a large effect on ℓ_s as shown in Fig. 2 ($\ell_s > 0$ for hinged ends; $\ell_s = 0$ for fixed ends). In nature, longitudinal cracks are often observed some distance off the edges which suggests hinged end conditions. These cracks are commonly called "hinge" cracks. Figure 3 indicates that, for $\lambda W < 3$, only one central hinge crack develops whereas, for $\lambda W > 6$, the solution becomes independent of W and coincides with Billfalk's results (1981) for the infinitely wide channel case.

Formation of cracks permits the water to escape upwards and, with increasing stage, to lift and detach the middle portion of the cover (if two cracks form). The strips that are attached to the shore become submerged but they too may eventually detach owing to thermal effects.

Formation of transverse cracks

An ice cover that is no longer restrained by the channel boundaries becomes subject to substantial bending moments both on vertical and "horizontal" planes (the quotation marks indicate that this term is used with some license - such planes are parallel to the water surface). As shown by Billfalk (1982), steep waves can cause transverse cracks by vertical bending while Shulyakovskii (1972) suggested that "horizontal" bending could also produce transverse cracks. Though such cracks are often observed and mentioned in the literature, (see for example MacKenzie River Basin Committee 1981) few attempts have been made to document their spacings and patterns. Beltaos (1984b) reported a fairly regular pattern observed in a S. Ontario stream (Thames R.) during the 1982 breakup. The median spacing was ≈300 m with h_i and W ≈0.35 m and 55 m, respectively. During a secondary breakup event in the same river (primary breakup in February 1984 and new ice cover formation in March), the median crack spacing was ≈130 m with h_i ≈0.10 m and W ≈40 m (Fig. 4). Such spacings are far too large to have been caused by vertical bending (Billfalk 1982). Beltaos (1984b) considered "horizontal" bending as a possible cause but the outcome was inconclusive. More data and theory are thus needed to elucidate the causes of transverse cracks.

Fig. 4. Transverse cracks in Thames R. (Ontario)

Other types of fracture

Michel and Abdelnour (1976) presented the results of a laboratory study on the initial breakup of a solid ice cover, using a wax-based material to simulate the scaled-down ice properties. However, it is difficult to decide how applicable their results are to natural streams because (a) a rectangular straight flume was used as opposed to the meandering planform and outward sloping banks of natural streams; and (b) the failure mechanism involved submergence of the leading edge of the cover and subsequent oscillations which suggests that, if this mechanism occurs in nature, it should be limited to the vicinities of leading edges.

Another cracking pattern that has been observed involves the passage of a steep flood wave in a very wide stream (Mackenzie R., Parkinson 1982). In this case both transverse and longitudinal cracks appeared first, fracturing the cover into large sheets with dimensions of the order of hundreds of meters. With continued rise of the water level, some of these sheets moved until they wedged against the shore. This movement was accompanied by widespread breakage and crushing, resulting in formation of floes of the order of tens of meters. After these initial movements, the water level dropped and the ice remained stationary until "the rising discharge reached the point where it could lift the broken ice and carry it downstream". The initial cracking pattern prior to ice movement suggests three-dimensionality in the shape of the flood wave.

Initial movement of the ice cover

We have seen so far some of the mechanisms and patterns by which an intact ice cover can be fractured into large sheets and floes. Such large sheets may or may not be set in motion, depending on whether there exists sufficient room on the water surface. Using this notion and field observations in the Thames R. (S. Ontario), Beltaos (1984b) formulated a dimensionless criterion for the initial movement of the ice. The principal factor facilitating the movement was identified as the water surface width in relation to the dimensions of the sheets that form after the initial cracking of the cover. This concept led to some success in generalizing forecasts of breakup initiation but it was pointed out that many additional factors remain to be accounted for.

Limitations

The preceding discussion focused on phenomena that result from the interaction among the forces applied on the ice cover by the flow, the cover's structural integrity and the boundary constraints imposed by channel geometry. Thermal processes may greatly complicate the picture because they interfere with the integrity of the ice in largely unpredictable ways, e.g., formation of holes and open leads in the ice, reduction of thickness and width by melting, loss of strength and candling.

SEVERITY OF BREAKUP - ICE JAMS

Once the initial phase of breakup has been completed and the ice cover has been set in motion, subsequent developments become almost chaotic. Moving ice sheets impact on channel boundaries or on other sheets and break down into small fragments. Ice jams begin to form and the water level profile becomes highly irregular and unsteady as it is now controlled by the backwater of the jams. With increasing discharge and thermal deterioration, some of the jams dislodge, move slowly or in surges and join other jams or cause further breakage of sections of ice cover that had remained intact. Considering the multitude of factors that are at work, the possible configurations of ice covered-open water sequences, and thence of the water level profile, are almost limitless. Despite its disorderly nature, this phase of breakup is the most important because it is associated with the various problems caused by ice jams. Consequently, a large part of this section will be devoted to ice jamming.

Initiation of ice jams

In nature, the most common cause of breakup jams is competent, stationary ice cover that may be encountered by moving ice fragments. While this occurrence alone is often sufficient to cause jams, it can combine with morphologic or man-made features to enhance the likelihood of jamming, e.g., constrictions, bends, shallows, slope reductions, bridge piers, etc.

The stability of a floating ice block that has come to rest against a transverse obstacle (e.g., ice cover) has been studied extensively. Depending on its own characteristics and pressure distribution on its wetted boundary, the block may or may not submerge under the obstacle. Ashton's simple theory gives good results under most practical situations (see Ashton 1978 for details and a review of pertinent studies):

$$V_C/\sqrt{(1-s_i)gh_i} = 2(1 - \frac{h_i}{H_u})/\sqrt{5-3(1 - \frac{h_i}{H_u})^2} \qquad (2)$$

in which V_C = "critical" velocity such that a block of thickness h_i submerges when the average upstream velocity exceeds V_C; g = acceleration of gravity; s_i = specific gravity of ice; and H_u = upstream flow depth. If the incoming blocks do not submerge, a jam comprising a single layer of blocks will be initiated. If the blocks submerge, a multi-layered jam may form, depending on the ability of the flow to transport the submerging blocks under the obstacle. Using the well-known "no-spill" assumption, Pariset et al (1966) predicted the thickness, t, of this kind of jam as:

$$t = V^2/2(1 - s_i)g \qquad (3)$$

in which V = average velocity under the jam. A different theory, based on energy considerations, was advanced by Tatinclaux (1977) and generally gives larger t's than does Eq. 3. If the flow depth under the obstacle is comparable to the size of the submerging blocks, grounding may occur but little else is known about this phenomenon (Mathieu and Michel 1967).

The above results are principally founded on laboratory tests and implicitly assume that ice is unbreakable. In nature, however, violent ice runs are often seen to arrive at ice cover edges where they eventually come to a halt after intense breaking and piling up. It is unlikely that the submergence criterion can alone describe such phenomena and thus research is needed with simulated breakable covers.

A jam initiation mechanism that does not require the presence of an obstacle is congestion. This occurs when the channel capacity to transport ice fragments is exceeded by the ice discharge (Frankenstein and Assur, 1972; Calkins and Ashton, 1975; Tatinclaux and Lee, 1978). Congestion is not a frequent occurrence during breakup but plays an important role during freeze up.

Evolution and equilibrium

Once a stable toe (downstream end of a jam) has formed, the jam lengthens upstream but it is not clear at present how thickness and length vary with time during this transient phase. Under certain circumstances a jam can attain a steady state and, if sufficiently long, it may have an equilibrium reach, as sketched in Fig. 5.

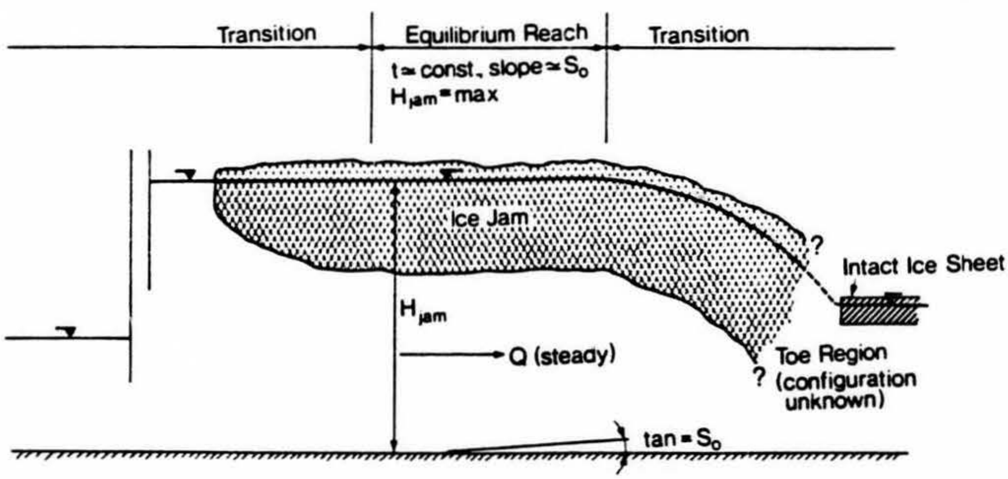

Fig. 5. Profile of a jam with an equilibrium reach

The equilibrium reach is characterized by constant jam thickness and flow depth so that the slope of the water surface is equal to that of the channel bed. Moreover, it can be shown that the water depth attains a maximum value in the equilibrium reach. Under these conditions, some theoretical reasoning has been possible (Pariset et al 1966; Uzuner and Kennedy 1976). The jam is considered a granular mass and the internal stresses are calculated in terms of the applied forces. Pariset et al (1966) identified two different cases, i.e.

(i) the "narrow" channel jam in which the internal stresses decrease in the downstream direction and the thickness is governed by hydrodynamic constraints at the leading edge (Eq. 3); and

(ii) the "wide" channel jam in which the internal stresses increase in the downstream direction, reaching an asymptotic value within a few river widths from the leading edge. The equilibrium thickness, t, of the jam is just sufficient to withstand the applied forces and satisfies the equation

$$(\tau_i + w_i)W = 2\ C_i t + \mu s_i\ (1 - s_i)\rho g t^2 \tag{4}$$

in which W = channel width; τ_i = flow shear stress on jam underside; w_i = streamwise component of jam's own weight per unit area = $s_i \rho g t S$; S = channel slope; C_i = jam cohesion; μ =dimensionless coefficient that depends on the internal friction of the jam; and ρ = water density. Comparison of Eqs. 3 and 4 has shown that "narrow" jams should not occur in any but very small streams (Beltaos, 1983).

There are many difficulties in testing Eq. 4 with field data. Often the jam thickness cannot be measured owing to access and safety problems; assessment of τ_i usually requires introduction of assumptions pertaining to the composite-resistance flow under the jam; and frequently flow discharge is unknown. Nevertheless, field measurements (Pariset et al, 1966; Calkins, 1983; Beltaos, 1983) seem to support the "wide" jam theory and yield consistent values of about 1.3 for the coefficient μ. For breakup jams cohesion seems to have a negligible effect in Eq. 4.

If the above mentioned assumptions are not made and jam thickness measurements are unavailable, as is usually the case with breakup jams, the theory can be tested indirectly in terms of the water depth in the equilibrium reach, H. This depth can be measured by combining photographs of jam levels against identifiable features on the river banks with subsequent hydrometric surveys under open-water conditions, e.g., see Beltaos (1983). Moreover, use of Eq. 4 in conjunction with hydraulic resistance considerations for the flow under the jam results in a dimensionless relationship that has the form (Beltaos, 1983):

$$\eta \equiv H/WS = F\ (f_o,\ f_i/f_o,\ \mu,\ \xi) \tag{5}$$

in which f_o = composite friction factor of the flow under the jam = 0.5 $(f_i + f_b)$; f_i, f_b = ice and bed friction factors, respectively; and ξ = dimensionless discharge $\equiv(q^2/gS)^{1/3}/WS$ with q = discharge intensity. Equation 5 neglects cohesion, as explained earlier. The main independent variable in Eq. 5 is ξ, so that field data can be plotted in the form of η versus ξ as shown in Fig. 6. The equilibrium jam data points define a satisfactory relationship while the non-equilibrium ones generally fall below this relationship, as expected. This result provides additional (though indirect) support for the theory. It is also noted that the graph of Fig. 6 and the formulation of Eq. 5 are suitable for practical applications where q, W and S are usually given and H is to be estimated. The implications of Fig. 6 and an alternative, more detailed, method to compute H are discussed by Beltaos (1983).

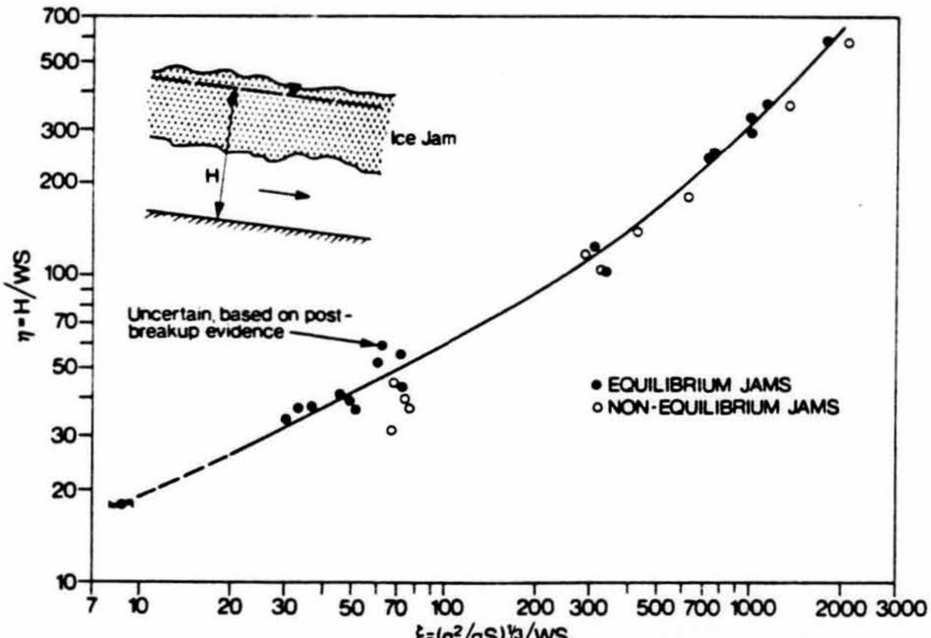

Fig. 6. Dimensionless depth versus dimensionless discharge for breakup jams

Release of ice jams

How, why and when jams release is generally unknown, but it is suspected that discharge, toe conditions and thermal effects play a role. Two common modes of release that have been observed by the writer are described below.

(i) Jam held by long section of intact ice cover: in this instance, the intact ice cover is, at least, tens of river widths long. After formation of the jam, open water leads begin to develop in the ice cover, downstream of and close to the toe of the jam; occasionally, ice blocks from the jam move into these leads. Shortly before release, the water speed in one or more leads increases drastically; more ice moves in from the jam and enlarges the lead as it impacts at its downstream end. Often, the front of the lead is seen to advance even in the absence of ice blocks. Release of the jam occurs during this time and is preceded by movement of large amounts of ice blocks in the leads. Once a jam begins to move, it may gain enough momentum so as to completely "clear" the reach of observation or it may be arrested again. In the latter case, new leads begin to develop and the jam may keep advancing in this manner for several days.

(ii) Jam held by a short section of intact ice cover: in this case the ice sheet holding the jam has dimensions of the order of the river width and is lodged against the channel boundaries (e.g., constrictions, bends) or other obstacles (e.g., bridge piers). While formation of leads may also occur in this case, the sheets often dislodge when increasing stage causes them to rise and "clear" the channel boundaries. In the case of bridge piers, sheets often break against the piers and move downstream followed by the jam.

A sudden jam release and the ensuing ice run is one of the most spectacular and violent events that occur during breakup. The stage rises very rapidly at downstream locations and water velocities far exceed even those attained during extreme open-water floods (see Gerard, 1979 for a few witness accounts). Sometimes the ice run encounters competent ice cover where it is arrested and a new jam begins to form. In such instances, rapid rise of the water levels upstream of the toe can again take place.

Such dynamic aspects of breakup are obviously very important but have received little attention. Work done to date (Mercer and Cooper, 1977; Henderson and Gerard, 1981; Joliffe and Gerard, 1982; Beltaos and Krishnappan, 1982) tentatively suggests that, apart from the first few minutes of movement, the release can be modelled by ignoring the presence of ice. However, modelling ice jam formation due to the arrest of a surging ice run is not possible at present. The main difficulty here is the lack of knowledge regarding conditions at the toe of the jam.

Forecasting

At a given site, the severity of breakup could be quantified by the magnitude and duration of high water levels and speeds. In turn, these are related to the magnitude, number and persistence of nearby jams. Present knowledge can only help identify potential jam sites; it cannot predict whether, where and when jams will actually form and release. Similarly, the equilibrium "wide" jam theory has limited practical utility because it can only furnish a potential value for the maximum breakup stage, H_m (see for example, Beltaos 1983). Whether and how closely this potential will actually be realized is unknown and depends on many factors in addition to discharge, e.g., stability of jams that actually form and proximity to the site of interest; competence of the ice cover and degree of thermal deterioration; and possibility of overbank spreading of water and ice. While there is little theoretical guidance in this regard, empirical evidence suggests that ice thickness, width and strength influence the value of H_m by limiting the discharge up to which stable jams can form (Beltaos, 1984a, 1984b). An example is shown in Fig. 7 where a partial dependency of H_m on h_i is illustrated.

Long-term forecasting and flood risk assessments require derivation of the peak breakup stage-frequency relationship because the annual flood peak often occurs during breakup at relatively low discharge. Use of historical data greatly facilitates this task (Gerard and Karpuk, 1979; Beltaos, 1984a). Where historical information is unavailable, a limited degree of guidance can be obtained from the equilibrium jam theory. No effort has been made yet to develop correlations analogous to those obtained by regional analysis on open-water floods; this would enable transposition of data from sites with historical records to those without.

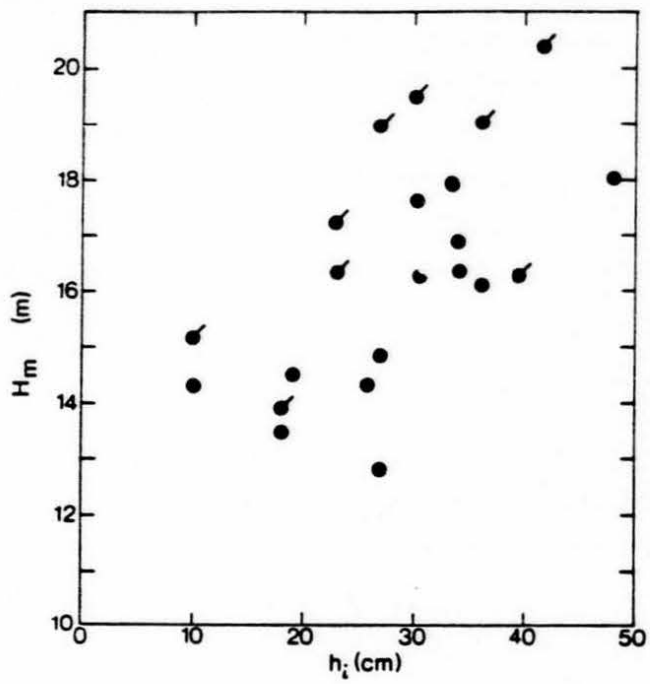

Fig. 7. Effect of ice thickness on maximum breakup stage, Thames R. at Thamesville, Ontario. (H_m = water surface elevation above an arbitrary datum. Data points with strokes indicate that ice thickness was estimated - error up to 30%).

CONTROL MEASURES

An extensive review of control measures to alleviate breakup effects is given by Bolsenga (1968). In general, control methods can be subdivided into ice modification and river modification, as outlined below.

Ice modification

- Dusting: material of low albedo is spread on the ice surface to promote heat transfer. The effectiveness of dusting depends on weather conditions.
- Thermal regime modification: the water temperature is altered to prevent formation of or weaken the ice cover at critical areas.
- Ice breaking: ice breakers or other vessels are used to break the ice downstream of ice jams or at critical areas prior to breakup. Sometimes, different equipment is used to cut open leads in the cover before breakup. Ice breaking is usually effective but costly.
- Explosives: blasting and even bombing have been employed in the past to remove ice jams but their effectiveness is uncertain.

River modification

This involves permanent measures resulting in alterations of the flow pattern and regime of the river, i.e.:

- channelization, e.g., elimination of morphological features conducive to jamming.
- erection of ice retention or diversion structures, e.g., ice booms, dykes and dams.

Because of the relative underdevelopment of the state of the art on breakup, the design of control measures depends largely on experience. Where feasible, field observations over one or more seasons are considered highly advisable.

PHYSICAL MODELLING

In many hydrotechnical applications related to breakup, a detailed understanding of the processes involved is required along with their impact on proposed structures and vice-versa. As has already been shown, present capabilities for mathematical prediction are very limited and resort to physical modelling is often the only satisfactory alternative. The main difficulty in physical modelling is proper scaling down of ice properties which precludes the use of freshwater ice. Kotras et al (1977) and Michel (1978) presented comprehensive discussions of scaling requirements and procedures. For applications at room temperature, commonly used materials are synthetic wax - or plaster of paris - based. Where cold room facilities are available, saline ice and doped ice can be used (Timco, 1981; Hirayama, 1983). Most of the physical modelling performed to date pertains to the interaction of ice with structures and ships. Applications to river ice breakup are few (e.g., see Michel et al, 1973; Michel and Abdelnour, 1976) and quantitative extrapolations of model results to prototype conditions should not be made without verification.

SUMMARY

Ice breakup is an important event in the regime of northern rivers and has serious repercussions to many aspects of hydrotechnical engineering such as flooding, forces on river structures, erosion, hydropower production and environmental impact assessments.

Breakup is triggered by mild weather via increased runoff and heat input to the ice cover. Forecasting the onset of breakup has largely been an empirical endeavour that relies heavily on historical data. Recent work has produced partial insight for common occurrences during the early stages of breakup, i.e., uplift pressures and formation of hinge cracks; lifting and detachment of the ice cover; formation of transverse cracks and breakdown of the cover into large sheets; increased water stage and movement of ice sheets; subsequent breakdown into smaller fragments by impacts. However, this sequence is not necessarily a unique one and more observational evidence is needed to elucidate other mechanisms that could initiate breakup.

After initiation, the development of breakup becomes almost chaotic owing to the multitude and unpredictability of the factors that are at work. Yet, this phase of breakup is the most important in hydrotechnical engineering because of the attendant ice jams and ice runs. The present knowledge of ice jam initiation and evolution is meagre: it can help identify potential ice jam sites but cannot predict whether, where and when jams will actually form and release. Lack of understanding with regard to the effects of ice breakability is considered a major gap. Only if it is assumed that a jam has formed, attained equilibrium and fully affects the site of interest can its stage and thickness be estimated, using the granular mass theory. In nature, however, the conditions assumed by the theory are not always fulfilled and thus only potential high water stages can be estimated. Actual breakup stages depend, in addition to the factors considered by the theory, on channel geometry and competence of the ice cover. In view of the magnitude of the complexities involved, physical modelling is an attractive alternative to mathematical modeling but has not yet achieved the same degree of advancement as modelling of open-water phenomena.

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Shoreline Erosion and Restabilization in the Southern Indian Lake Reservoir[1]

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Newbury, R. W., and G. K. McCullough. 1984. Shoreline erosion and restabilization in the Southern Indian Lake reservoir. Can. J. Fish. Aquat. Sci. 41: 558–566.

Prior to a 3-m impoundment in 1976, bedrock comprised 76% of the shoreline of Southern Indian Lake in northern Manitoba. This was reduced to only 14% of the shoreline as the water level rose above the wave-washed zone and flooded into the predominantly fine-grained, frozen overburden materials. Twenty monitoring sites were surveyed annually to determine rates of permafrost melting and solifluction and shoreline erosion. The sequence of shoreline erosion in permafrost materials was found to be cyclic, consisting of melting and undercutting of the backshore zone, massive faulting of the overhanging shoreline, and removal of the melting and fractured debris. Rates of shoreline erosion varied widely, depending on the exposure of the site to wave action and the composition of the backshore materials. At sites in fine-grained frozen silts and clays, representative of over three quarters of the postimpoundment shoreline, rates of retreat of up to $12\ m \cdot yr^{-1}$ were observed. The total volume of shoreline materials removed varied from less than 1 to over $23\ m^3 \cdot m$ shoreline $length^{-1} \cdot yr^{-1}$. Clearing of the forested backshore prior to flooding did not affect the erosion rates. The index of erosion based on the hindcast wave energy component perpendicular to the shoreline was $0.00035\ m^3 \cdot t\text{-}m$ wave $energy^{-1}$ ($0.036\ m^3 \cdot MJ^{-1}$). The minimum period of restabilization of the shoreline based on the volume of backshore materials that must be eroded before bedrock conditions are reestablished was estimated to be 35 yr for three quarters of the shoreline surrounding the lake.

Avant la retenue et l'élévation de 3 m du niveau de l'eau en 1976, 76 % de la ligne de rivage du lac Sud des Indiens, dans le nord du Manitoba, étaient constitués par la couche rocheuse. Ce pourcentage a diminué à seulement 14 % à mesure que le niveau de l'eau s'éleva au-dessus de la zone balayée par les vagues et inonda la région formée de matériaux de surcharge congelés. Vingt sites de surveillance ont été couverts annuellement en vue de déterminer les taux de fonte et de solifluxion du pergélisol, et d'érosion de la ligne de rivage. On a constaté que la succession de cette érosion dans les matériaux du pergélisol était de nature cyclique, comprenant la fonte et le sappement de la zone d'arrière-rivage, le faillage massif de la ligne de rivage en surplomb et l'enlèvement des débris dégelés et fracturés. Les taux d'érosion de la ligne de rivage accusent de fortes variations, selon le degré d'exposition du site à l'action des vagues et la composition des matériaux de l'arrière-rivage. Aux sites d'argiles et de vases à grains fins congelés, représentant plus des trois quarts de la ligne de rivage d'après retenue des eaux, on a observé des taux de retrait allant jusqu'à 12 m par année. Le volume total de matériaux enlevés de la ligne de rivage a varié de moins de 1 m^3 à plus de 23 m^3 par mètre de longueur de rivage par année. Le déboisement de l'arrière-rivage avant l'inondation n'a pas influencé les taux d'érosion. L'indice d'érosion fondé sur la composante énergétique des vagues perpendiculaires à la ligne de rivage, obtenu par modèle prévisionnel à rebours, était de $0{,}00035\ m^3 \cdot m\text{-}t$ d'énergie des $vagues^{-1}$ ($0{,}036\ m^3 \cdot MJ^{-1}$). La période de restabilisation minimale de la ligne de rivage, fondée sur le volume de matériaux de l'arrière-rivage qui doit être érodé pour que les conditions de la couche dure soient rétablies, a été estimée à 35 ans pour les trois quarts de la ligne de rivage entourant le lac.

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The mean water level of Southern Indian Lake (latitude 57°N, longitude 99°W) was raised 3 m in 1976 to facilitate the southward diversion of the Churchill River to hydroelectric generating stations located on the lower Nelson River in northern Manitoba, Canada. The results of a field study of the rates of shoreline erosion in the permafrost materials at selected sites surrounding the lake have been reported previously (Newbury et al. 1978; Newbury and McCullough 1983). In this paper, the observed rates have been projected over the entire lake shoreline to obtain an estimate of the total weight of mineral materials added to the lake annually and to predict the total period of shoreline instability.

The rate of shoreline erosion in open water bodies is dependent on the onshore wave energy, the resistance of the shoreline materials to the drag forces exerted by the waves and return flows, and the configuration of the shoreline and offshore zone. On most natural shorelines, erosion rates are low due to the development of an offshore bar and shallow foreshore shelf that acts as a barrier to incoming waves (Bruun 1962; Kondratjev 1966; Penner and Swedlo 1975). On many of the erosion-resistant bedrock shorelines of lakes in the Precambrian Shield,

[1]This paper is one of a series on the effects of the Southern Indian Lake impoundment and Churchill River diversion.

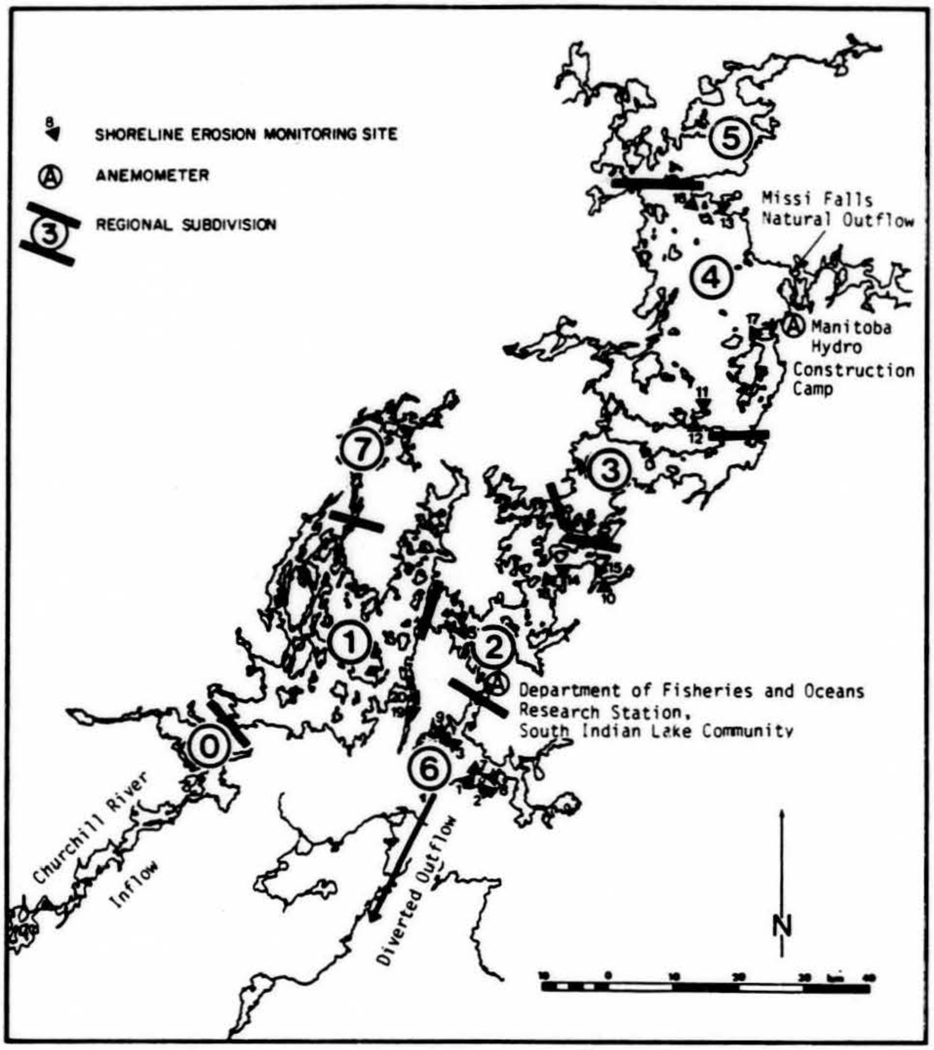

FIG. 1. Shoreline erosion monitoring sites on Southern Indian Lake.

wave erosion has removed overlying deposits, leaving a stable bedrock contact at the water's edge. In a newly created reservoir, or in an impounded lake where the water level extends inland beyond the established shoreline zone, erosion of overlying unconsolidated deposits can occur rapidly, as there is no barrier or eroded shelf to dissipate the energy of incoming waves. The shoreline changes from its initial preimpoundment configuration to a wave-cut bank with growing offshore deposits. The erosion continues until an equilibrium profile is reestablished or the overburden is removed and the underlying bedrock is exposed. The length of time required to establish the long-term stable form depends on the erosion rate and the characteristics of the eroding materials.

Observations in several reservoirs in the central USSR suggest that an erosion index (k_e) or "washout coefficient" can be derived for different backshore materials expressed as the volume eroded per unit of onshore wave energy. For silty-clays similar to those of the Southern Indian Lake region, Kachugin (1966) suggested an erosion index of $0.0005\ m^2 \cdot t^{-1}$ of wave energy, which is equivalent to $0.051\ m^3 \cdot MJ^{-1}$. This figure was derived from observations gathered in new reservoirs south of the permafrost region. The previously reported (Newbury et al. 1978) initial erosion index for shorelines at Southern Indian Lake in similar but permanently frozen materials was $0.00063\ m^2 \cdot t^{-1}$. The longer term index comparable with Kachugin's (1966) estimate was $0.00035\ m^2 \cdot t^{-1}$ or $0.036\ m^3 \cdot MJ^{-1}$.

Methods

Prior to impoundment, 38 cross-sections of the backshore, beach, and foreshore zones were surveyed using a theodolite, tape, stadia rod, and acoustic sounder at 20 erosion monitoring sites surrounding the lake (Fig. 1). The sites were selected to represent a variety of erodible backshore materials with both high and low exposure to the main body of the lake. In 1974, 1975, and 1976, wind records were obtained using a type 45B recorder on a 10-m tower at Missi Falls on the northeast shore of the lake (Atmospheric Environment Service, Environment Canada, unpubl. data). In 1975, a type 45B and a type U2A wind recorder were installed at the Department of Fisheries and Oceans camp near the village of South Indian Lake. After 1976, the Missi Falls tower was dismantled and only the South Indian records were available for wave hindcasting.

Beginning in 1976, the 1st yr of impoundment, annual erosion surveys were conducted at each location. Topographic surveys and offshore soundings to a depth of 4 m at the preselected sites were plotted on the original cross-sections to obtain the annual volume of erosion and sedimentation. Low-level aerial photography was used to determine the average rate of shoreline retreat between each cross-section. Backshore and beach soil profiles were surveyed by cutting through the overlying moss layers to expose the overburden surface. Representative overburden samples from each site were analyzed for sand, silt, and clay content by standard sieve and settling–pipette techniques (McKeague 1976). Ice content in frozen materials was determined by weighing bulk samples before and after thawing and drying. Depth to permafrost was determined by hand-augering in late August or September. Offshore deposits were sampled with an Ekman dredge and a 7.5 cm × 3 m coring tube.

Wind data recorded over land for the periods between surveys were adjusted to overwater speeds in accordance with Richards and Philips (1970). The duration of winds was compiled in six speed classes from 0–8 to 41–48 $km \cdot h^{-1}$ in 8–45° directional sectors. Onshore erosive wave energies were determined for each speed class, directional sector, and duration acting upon a particular monitoring site in accordance with the modified Sverdrup–Munk method of hindcasting the significant wave height, H_s. For the convenient use of the USCE (1966) wave hindcasting charts, wind and wave data were compiled in f.p.s. units, producing wave energy estimates expressed as foot-pound per unit length of shoreline. For comparison with the erosion indices observed in USSR reservoirs by Kachugin (1966), the wave energy was converted to metric units: tonne-metres per unit length of shoreline. In this paper, the erosion index is also given in SI units as megajoules per unit length of shoreline. Estimates of the portion of the wave energy expended on a unit length of shoreline by approaching deep-water waves generated in the adjacent lake basin were based on the USCE (1966) approximate wave energy relationships for a unit length of wave crest, corrected for the angle of approach of the wave train in each directional sector:

$$\Sigma E_t = \frac{\rho g H_e^2 L}{16} \cos^2 \theta (t/T)$$

where ΣE_t = perpendicular component of wave energy reaching the shore from each wind direction sector and speed class between erosion surveys (foot-pounds), H_e = equivalent wave height ($0.71H_s$), a representative single wave having an energy equal to the sum of the energies of all waves in the spectrum hindcast for the average velocity in each wind speed class, directional sector, offshore depth, and offshore fetch (feet), L = wavelength (feet), θ = angle between the midpoint of each wind directional sector acting on the shoreline and a line perpendicular to the shoreline segment (degrees), t = total duration of winds in each speed class and directional sector between erosion surveys (seconds), T = wave period (seconds), ρ = density of water (slugs per cubic foot), and g = acceleration of gravity (feet per square second).

The net volume eroded between surveys was divided into the incident wave energy to obtain a gross erosion index for each site, K_e (square metres per tonne), which expresses the erosion (cubic metres) per unit of wave energy (tonne-metres) (alternatively, cubic metres per megajoule in SI units). A mineral erosion index was determined by subtracting the peat, water, and ice content from the volume eroded between surveys.

The total volume of eroded materials that was contributed to each major basin of the lake was estimated for the years 1976–78. Eroding, noneroding, and newly exposed bedrock shorelines were located in a reconnaissance of the entire shoreline made by air and boat in 1978. Shoreline materials had been mapped before impoundment (Water Resources Branch 1974). The volume of shoreline materials contributed from eroding reaches was estimated using the onshore wave energy and mineral erosion index calculated from the rates observed at the monitoring sites.

Results and Discussion

Representative textures and ice contents of shoreline deposits at each monitoring site are given in Table 1. The in situ deposits at site 1 are shown in Fig. 2. The average depth to permafrost at the end of the summer season varied from 0.5 m in thick peats to 1.4 m in clay-till deposits with less than 5 cm of organic ground cover. Permafrost was not encountered in sand and gravel deposits.

Erosion and Deposition Processes

An example of surveyed profiles showing annual erosion and nearshore deposition of frozen lacustrine clay at a relatively high wave energy site is shown in Fig. 3. The erosion of frozen fine-grained materials on shorelines surrounding the larger basins of the lake was observed to proceed in a repeated sequence of melting, slumping, and removal. As the ice pockets and lenses melt, the bank materials become oversaturated with water and form a slurry-like mixture. The partially thawed materials flow out to form a silty-clay beach strewn with

TABLE 1. Textural and ice content analyses of mineral materials at selected shoreline sites. ND, not determined; NP, not present.

Site	Textural analysis of parent materials (%) Sand	Silt	Clay	Ice content (% of dry weight)
1	1	15	84	43
2	0	16	84	ND
3	1	34	65	ND
4	0	15	85	64
5	1	26	73	ND
6	1	34	65	64
7	0	49	51	ND
8	19	16	65	ND
9	1	17	82	ND
10	35	46	19	ND
11	10	45	45	47
12	1	19	80	ND
13	8	39	53	56
14[a]	2	36	62	92
15	98	0	2	NP
16[a]	9	33	58	ND
17[a]	4	37	59	43
18[b]	ND	ND	ND	NP
19	1	34	65	ND
20	ND	ND	ND	ND

[a]Samples analyzed were of backshore lacustrine deposits. To date, erosion at these sites has been predominantly of former sandy beach materials.

[b]Predominantly fine to coarse sand with some silty beds.

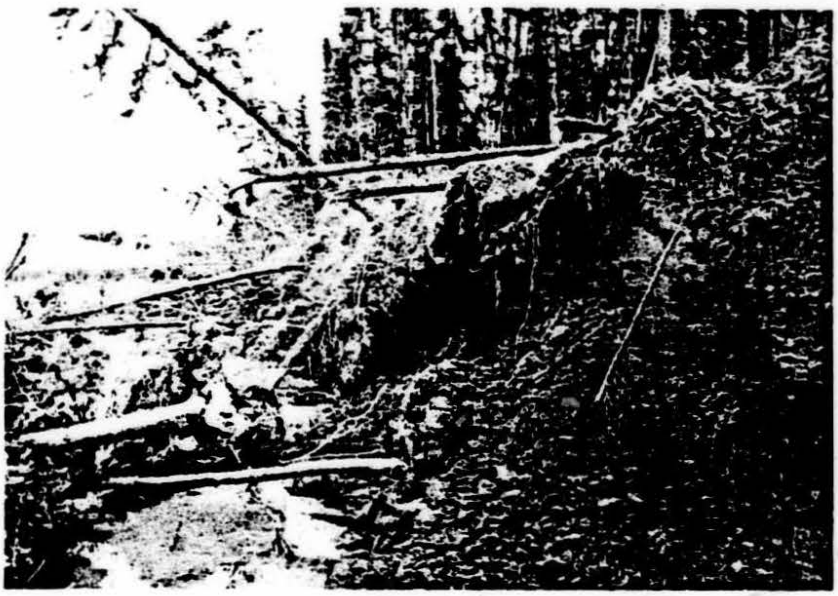

FIG. 2. Erosion in frozen lacustrine clay at site 1.

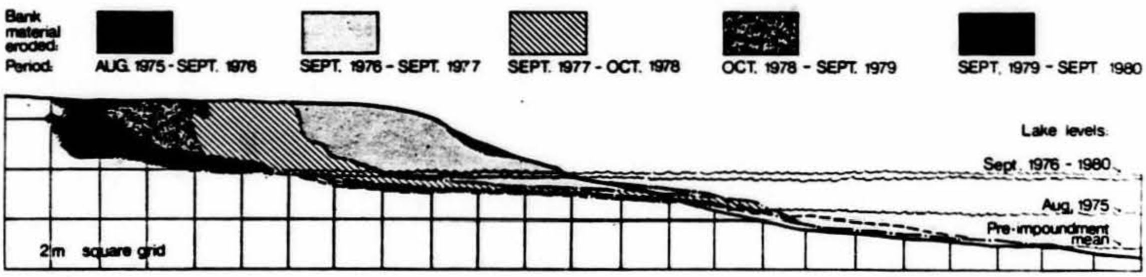

FIG. 3. Consecutive annual shoreline profiles indicating the rate of erosion at a high wave energy site in frozen lacustrine clay, site 1. Note that some nearshore deposition is occurring.

scattered frozen blocks. In regions of high wave energy the newly thawed shoreline materials are completely removed during storms. Where the backshore is covered by an insulating blanket of peat, melting occurs below and slightly above the water surface. In some cases, caverns or melt niches are formed that are up to 1 m in height and extending 3 m into and under the frozen backshore materials (Fig. 4). Depending on the thickness of the overlying materials, the cavern will deepen until the projecting block of shoreline materials splits away from the main land mass and falls onto the foreshore (Fig. 5). The blanket of moss and roots overlying the block often remains intact as slippage occurs in the active layer. The former ground cover drapes over the backshore zone and is slowly broken up and carried offshore during subsequent storm periods. The form of the slumping and eroding shorelines does not change substantially as the backshore moves inland. If bedrock is encountered at the eroding face, erosion at the water level ceases. In the wave-washed zone overlying the bedrock, erosion continues until a bedrock backshore zone is exposed up to the maximum wave uprush elevation.

Observations of the flooded foreshore zone surrounding the lake, made by sounding, coring, and diving, indicate that a large portion of the eroded materials are deposited within 300 m of the shoreline. This evidence is supported by sediment budgets based on samples of the water column taken throughout the lake, which indicate that only a few percent of the eroded materials are carried into the main water mass in suspension (Hecky and McCullough 1984). Deposition of the fine-grained materials often occurs before they are completely broken down into silt and clay sizes. Beginning at the beach as frozen blocks up to 0.5 m in diameter, the aggregates of fine-grained materials are reduced in size as they are abraded and transported farther offshore (Fig. 6). At site 1, for example, clay aggregates up to 6 mm in diameter were found 20 m offshore at a depth of 2 m. At 60 m offshore and 3 m depth, the aggregates were less than 0.5 mm in diameter. At 180 m offshore and 4.8 m depth, the maximum diameter of the freshly deposited materials was 0.1 mm. A sediment core at 20 m offshore indicated that 0.5 m of deposition had occurred on the previously bare bedrock surface in the first 5 yr of impoundment. Five 5- to 10-mm-thick bands of grey clay contained in the core were separated by thicker layers of dark brown agglomerates, suggesting that the depositional sequence corresponded to the five winters of under-ice deposition and six summers of open-water deposition that had occurred since the impoundment.

Erosion and deposition processes observed at two monitoring sites (15 and 18) in coarse-grained unfrozen deposits of sands and gravels agreed with those reported by Bruun (1962) for similar unfrozen materials. The rates of erosion were of the same magnitude as those of the fine-grained materials. The

FIG. 4. Melt niche under bank with 6-m boat.

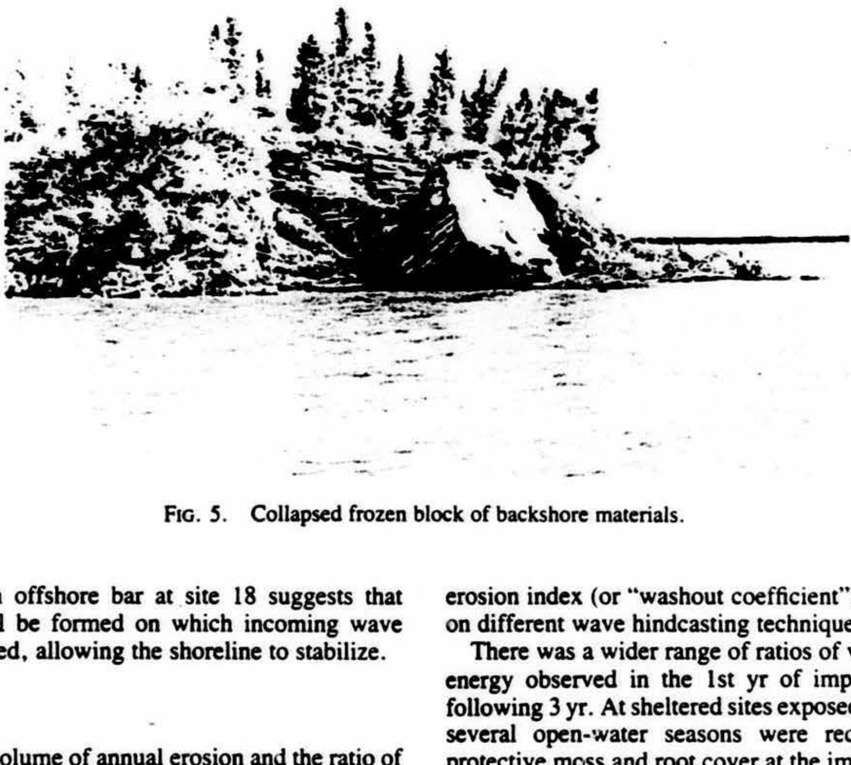

FIG. 5. Collapsed frozen block of backshore materials.

rapid deposition of an offshore bar at site 18 suggests that ultimately a shoal will be formed on which incoming wave energy will be dissipated, allowing the shoreline to stabilize.

Erosion Rates

In Table 2 the total volume of annual erosion and the ratio of volume eroded to incident wave energy at each of the 20 monitoring sites surrounding the lake are compiled for 4 yr of impoundment. Where bedrock was not encountered, the mean of erosion indices for the perennially frozen shorelines was generally one half of that reported by Kachugin (1966) for similar materials in the unfrozen state. Although this suggests that permafrost conditions may retard erosion, there are no data supplied with the USSR observations, and the magnitude of the erosion index (or "washout coefficient") may have been based on different wave hindcasting techniques.

There was a wider range of ratios of volume eroded to wave energy observed in the 1st yr of impoundment than in the following 3 yr. At sheltered sites exposed to low wave energies, several open-water seasons were required to destroy the protective moss and root cover at the impounded water's edge. At more exposed sites, large volumes of peat were easily removed from the flooded foreshore, producing high ratios in the 1st yr. Because of variability of early years, K_e values were calculated using 1978–80 erosion data only. Also excluded from the general K_e determinations were erosion values at sites after bedrock had been encountered at the eroding face (sites 6, 8, 10, and 11). Based on the 16 shoreline sites that extend over the range of materials and fetches encountered on Southern

IG. 6. Laminated clay exposed on the foreshore at a low lake level is shown in the upper photograph. The blocky structure evident in the dark ner bands abrades to pebble-shaped aggregates shown in the lower photograph. (The field book is 175 mm × 115 mm; the glass of the hand lens is 17 mm in diameter).

dian Lake, the K_e value for the higher erosion sites was found least squares analysis to be $0.00035\ m^2 \cdot t^{-1}$ ($r^2 = 0.71$, $n =$ 42) (Fig. 7). The 95% confidence interval for the slope of the regression relationship was $0.00028–0.00043\ m^2 \cdot t^{-1}$ (Neter id Wasserman 1974). The volume of dry mineral material oded was calculated using an estimated average water content of the perennially frozen silty-clays of 58% dry weight and assuming a bulk dry density of $2600\ kg \cdot m^{-3}$ (Table 3). The relationship between eroded mineral volume and incident wave energy indicated a mineral erosion index (K_{em}) of approximately $0.00012\ m^2 \cdot t^{-1}$ ($r^2 = 0.69$, $n = 39$) (Fig. 8) with a 95% confidence interval of $0.00009–0.00014\ m^2 \cdot t^{-1}$. The volume

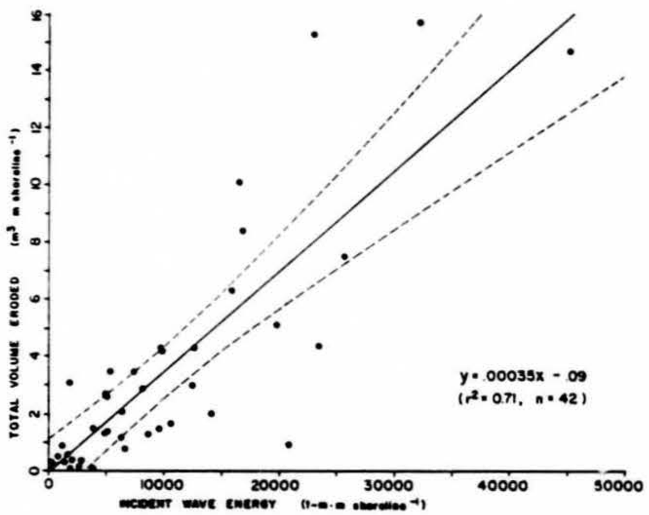

FIG. 7. Total volume of shoreline materials eroded vs. wave energy relationship with 95% confidence band limits, for the 3rd, 4th, and 5th yr of impoundment (1978–80) on Southern Indian Lake.

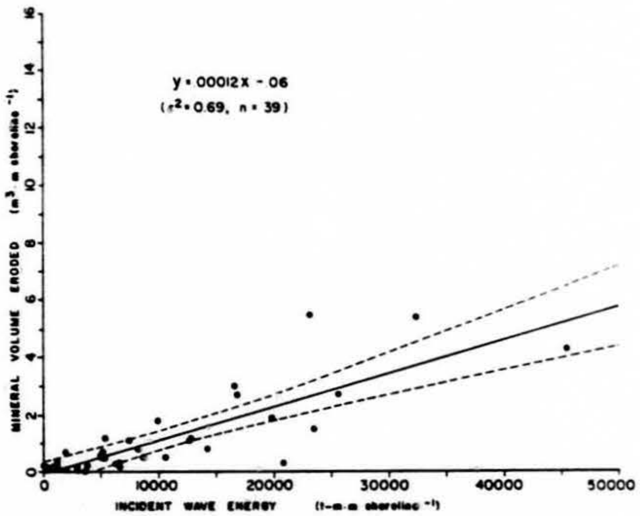

FIG. 8. Volume of mineral materials eroded vs. wave energy with 95% confidence band limits for the 3rd, 4th, and 5th yr of impoundment (1978–80) on Southern Indian Lake.

TABLE 2. Total volumes of material eroded annually from monitored shoreline sites on Southern Indian Lake ($m^3 \cdot m$ shoreline^{-1}). The ratio of volume eroded to calculated wave energy is shown in parentheses for each period ($\times 10^3\ m^2 \cdot t^{-1}$). Values for sites 13 and 16, 1979, are cumulative for 1978 and 1979 erosion years. NS, not surveyed.

Site	1977	1978	1979	1980
1	23.4 (0.77)	15.7 (0.48)	10.1 (0.61)	15.3 (0.66)
2	1.8 (0.18)	1.7 (0.16)	1.3 (0.27)	0.8 (0.12)
3	8.9 (0.35)	7.5 (0.29)	3.0 (0.24)	8.4 (0.50)
4	7.0 (0.67)	4.3 (0.34)	1.3 (0.15)	2.9 (0.35)
5	0.9 (0.71)	0.6 (0.38)	0.9 (0.74)	0.5 (0.56)
6	21.0 (1.32)	14.4 (0.90)	1.6 (0.14)	4.5 (0.28)
7	9.4 (0.48)	4.4 (0.19)	0.9 (0.04)	5.1 (0.26)
8	1.9 (0.84)	0.6 (0.26)	0.2 (0.13)	0.4 (0.17)
9	0.0 (0.00)	1.2 (0.19)	1.5 (0.38)	3.1 (1.70)
10	0.0 (0.00)	0.2 (0.06)	0.1 (0.07)	0.5 (0.21)
11	17.4 (0.54)	4.3 (0.11)	0.7 (0.03)	NS
12	4.8 (0.98)	2.1 (0.33)	2.6 (0.51)	NS
13	1.4 (0.35)	NS	2.0 (0.14)	NS
14	2.0 (0.34)	3.5 (0.47)	1.4 (0.27)	3.5 (0.66)
15	0.0 (0.00)	0.1 (0.56)	0.2 (1.32)	0.1 (0.51)
16	8.6 (0.94)	NS	14.6 (0.32)	NS
17	2.2 (1.35)	2.7 (0.55)	4.2 (0.42)	NS
18	1.0 (0.16)	1.5 (0.16)	6.3 (0.40)	4.3 (0.44)
19	2.1 (0.32)	0.1 (0.03)	0.1 (0.05)	0.2 (0.08)
20	1.3 (0.29)	0.4 (0.14)	0.3 (0.22)	0.4 (0.20)

of the organic layer eroded at each site (excluding site 18) was also correlated with incident wave energy to obtain an organic erosion index of 0.00006 $m^2 \cdot t^{-1}$ ($r^2 = 0.50$, $n = 39$) with a 95% confidence interval of -0.00006–$0.00019\ m^2 \cdot t^{-1}$. The low coefficient of determination and wide confidence interval for the organic erosion index is due to the difference in thickness of the peat layer between sites.

There is a greater variation in erosion rates when individual cross-sections are compared rather than average monitoring site values. The standard error of estimate for the prediction of the eroded volume at a single cross-section is 2.3 $m^3 \cdot m$ shoreline^{-1}, giving a 95% confidence interval of $\pm 4.6\ m^3 \cdot m^{-1}$ (Moroney 1974). The wide confidence interval reflects the high variability in shoreline erosion measured at each cross-section. The variability at a typical monitoring site (3) is illustrated in Fig. 9, where two preimpoundment headlands have been rapidly removed by erosion. The high variability of erosion rates between individual cross-sections suggests that the prediction of eroded volumes using the wave energy index can be applied successfully only to estimate average rates over a reach of shoreline that is sufficiently long to include cusps and headlands as minor features.

Total Annual Erosion

In Table 4, estimates of the total dry weight of mineral materials eroded in the years 1976, 1977, and 1978 are

TABLE 3. Volumes of mineral material eroded annually from monitored shoreline sites on Southern Indian Lake ($m^3 \cdot m$ shoreline^{-1}). Volumes for sites 13 and 16, 1979, are cumulative for 1978 and 1979 erosion years. Sites 15 and 18 are without permafrost and are not corrected for water content. NS, not surveyed.

Site	1978	1979	1980
1	5.4	3.0	5.5
2	0.5	0.5	0.2
3	2.7	1.1	2.7
4	1.1	0.5	0.8
5	0.2	0.4	0.1
7	1.5	0.3	1.9
9	0.3	0.2	0.7
12	0.3	0.7	NS
13	NS	0.8	NS
14	1.1	0.5	1.2
15	0.0	0.2	0.1
16	NS	4.3	NS
17	0.6	1.8	NS
18	1.5	6.2	4.2
19	0.0	0.0	0.0
20	0.1	0.1	0.1

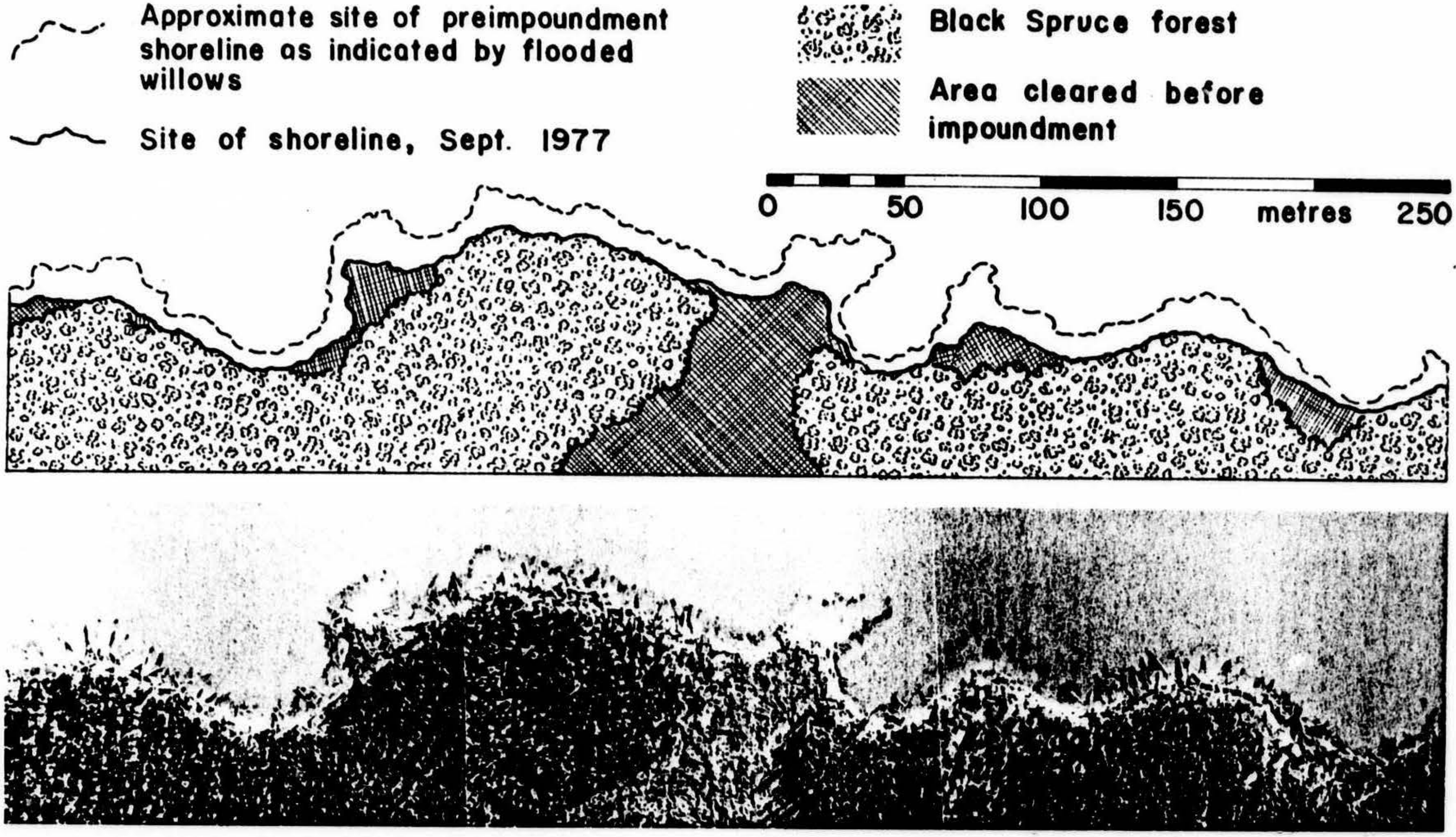

FIG. 9. Local variability in the rate of erosion is illustrated in this aerial photograph at site 3. The photograph was taken in September 1977, 1 yr after impoundment. The preimpoundment shoreline is approximately .elineated by flooded willows. The area was cleared before impoundment to the estimated 850-ft (259.1 m) MSL contour. Since impoundment, the shoreline has retreated into the uncleared forest in several places. Headlands and irregularities along the shoreline have been reduced as well.

TABLE 4. Estimated total dry weight of mineral materials eroded from the shorelines of the major basins of Southern Indian Lake for the period 1976–78 (10^6 kg).

Region	1976	1977	1978
0	122	177	166
1	528	672	615
2	238	311	290
3	478	668	608
4	1594	2099	1916
5	207	275	229
6	190	273	247
Whole lake	3357	4475	4071

summarized for each basin of the lake. The estimates are based on hindcast wave energies derived from wind records and 331 generalized shoreline reaches surrounding the lake. The average length of reach was 6.4 km. Mineral erosion indices of $0.000043\ m^2 \cdot t^{-1}$ in 1976 during impoundment and $0.00012\ m^2 \cdot t^{-1}$ in 1977 and 1978 at the full impoundment level were assumed. The total erosion estimates were not extended beyond 1978, as that was the last year in which a survey was undertaken to determine the portions of the total shoreline in overburden and bedrock materials.

Period of Shoreline Restabilization

Estimating the period required to restore the reservoir shorelines to their preimpoundment condition is a perplexing problem. A study of smaller reservoirs in the region developed for local mines and hydroelectric projects found that standing and fallen trees in the foreshore zone remained in place for periods of at least 40 yr, the age of the oldest reservoir surveyed (R. W. Newbury, G. K. McCullough, S. McLeod, and R. V. Oleson, University of Manitoba, Winnipeg, Man., unpubl. data).

In the first 5 yr of impoundment on Southern Indian Lake, restabilization has occurred only on shorelines where bedrock underlying the backshore zone was exposed at the water's edge by solifluction and erosion. Where bedrock was not encountered, there has been no change in the melting, slumping, and eroding sequence of shoreline migration. The annual erosion indices at monitoring sites in fine-grained materials have shown no diminishing trend following the 1st yr of impoundment. The clearing of shorelines up to the impoundment level did not affect erosion rates.

The period of recovery for most of the permafrost shorelines depends on when bedrock is encountered by the retreating erosion face. Thus, the position of the bedrock surface at the water level underlying the backshore zone must be established to discover the total volume of material to be removed before an estimate of the period of restabilization can be determined. Exploratory drilling of the backshore zone surrounding the lake is a prohibitively large project. Contemporary seismic techniques for indirectly determining the depth of the bedrock were found to be inaccurate in fine-grained, dense permafrost materials.

Without further bedrock information, only a general estimate of the minimum period of shoreline restabilization can be made, if the monitoring sites scattered throughout the lake are considered to be a representative sample of all shoreline conditions. Eighteen of the monitoring sites occur in fine-grained materials that are representative of three quarters (2841 km) of the postimpoundment shoreline. In the initial 4 yr of impoundment, four high wave energy sites have encountered bedrock in the retreating backshore. Assuming that the rate of bedrock encounters is representative of the early years of restabilization and that the rate will decay geometrically as bedrock is exposed at more protected sites, 4/18 of the remaining eroding shoreline will strike bedrock every 4 yr until the preimpoundment condition is restored.

Prior to impoundment, 76% of the shoreline was bedrock controlled. Following impoundment, bedrock was exposed on only 14% of the shoreline. At the recovery rate of the sample shorelines, it would take at least 35 yr to restore 90% of the fine-grained shorelines to their preimpoundment condition. The minimum period of recovery for shorelines in nonpermafrost granular deposits (4% of the flooded shoreline length) is approximately 20 yr, based on the erosion rates observed at site 18 and the volume of deposition required to form a protective offshore shoal similar to that proposed by Bruun (1962). It is apparent that the instability of the Southern Indian Lake shoreline environment and the high rates of sediment input to the lake waters will continue for several decades.

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Ice Management Manual

Ministry of Natural Resources

Ontario

Hon. Alan W. Pope
Minister

John R. Sloan
Deputy Minister

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Preface

Acting on numerous requests for guidance and procedures for dealing with chronic ice problems in Ontario, the Conservation Authorities and Water Management Branch initiated an ice jam program in 1980. The prime objective of the program was to prepare a manual that would include guidelines and procedures for monitoring, predicting and acting on ice break-up and jamming on rivers in Ontario where the history of ice jams is frequent and well-known.

The Branch initiated co-operative projects with the Grand River, Ganaraska Region and Credit Valley Conservation Authorities for the purposes of identifying factors affecting ice jams and collecting other pertinent data such as ice cover, air temperature and streamflow data. In addition, special investigations on specific ice jam problem areas in northern Ontario such as the Goulais River and Town Creek in Timmins were carried out. It was intended that data and information obtained from these activities would be used in the preparation of this manual.

The program also reviewed and documented the history of ice jam problems and remedial measures in the Province, including the documentation of the causes and extent of the events in February, 1981.

This manual is the product of all activities and studies that have been undertaken under this program since 1980. It is hoped that the information provided will be useful in preventing and dealing with the many ice jam related problems that are encountered in the Province.

M.R. Garrett

November, 1984

M.R.Garrett, Director
Conservation Authorities and
Water Management Branch

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Introduction

When rivers become jammed with broken, thawing ice which flows faster than the channel can carry it away, flooding occurs. Ontario's history of ice jams is well known; but the frequency and severity of the problem is increasing. Ice jams caused serious and widespread flooding in Ontario in February, 1981. Dover Township and Port Hope are specific examples of areas that suffered from disastrous flooding also in 1979 and 1980.

There are two sets of problems: the first may occur during freeze-up, when the amount and type of ice forming might impede the flow of water, which then backs up and overflows. The second trouble occurs when ice begins to break up in Ontario's waterways — not only during the annual spring thaw, but also during extreme and prolonged temperature fluctuations earlier in the winter. These abnormal fluctuations, together with a recent trend towards significantly colder winters, have exacerbated the problems associated with ice jams. An increasing number of requests for guidance and procedures to deal with these problems — not only chronic but often now acute problems — resulted in an Ice Jam Program, initiated in 1980 by the Ministry of Natural Resources' Conservation Authorities and Water Management Branch and the Conservation Authorities.

This manual is the result of the program's prime objective: to provide preliminary information and guidelines which can be used to determine the proper procedures for predicting, preventing, and dealing with ice-related flooding.

There are a number of factors generally common to all ice jam situations, and there are many similarities to be found in the conditions prevailing just prior to the jams, wherever they may occur. The studies of these factors and conditions, based on experience since 1960, have provided much of the information for this manual. Other up-to-date information since 1980 has also been obtained through the Ice Jam Program.

Each situation and location is, however, different; each area will have a unique combination of variables in its particular equation. Each area must, therefore, carry out its own data-collection program, which is necessary for determining the correct method of dealing with its own particular problem, as well as estimating the relative costs and benefits involved. It is essential to understand that, without this data (some of which can be collected from existing records, but much of which can only be determined by a monitoring program), the prevention of recurring problems is virtually impossible. The only alternatives are attempts to improve the situation after it has occurred, which are usually unsatisfactory.

Section 1 of this manual outlines briefly the historical patterns and changing trends relating to ice-jam floods in Ontario.

Section 2 summarizes the conditions causing ice-jam floods.

Section3 gives detailed descriptions of the factors involved in freeze-ups and the different kinds of ice formation that generally lead to problems.

Section 4 explains the causes of and techniques for predicting break-ups.

Section 5 lists the essential data to be collected through monitoring programs.

Section 6 assesses the success of remedial measures that have been practiced to date.

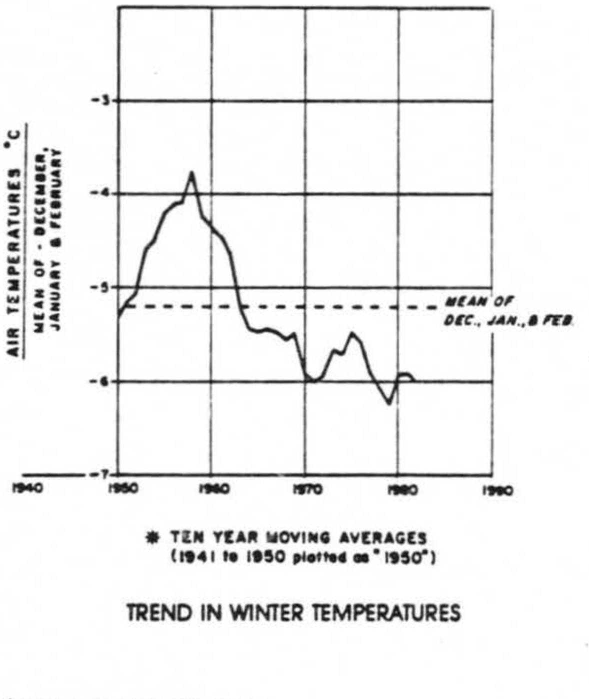
Section 7 summarizes the overall principles of ice control, and is followed, finally, by references and a bibliography (Section 8).

1 Historical Patterns and Changing Trends

Newspapers are the main source of historical records from which the frequency of recurring ice-related floods has been determined. This source has been useful in providing flood dates which can then be compared with weather reports from Environment Canada for the same periods. In this way, it has been possible to determine whether the problems were brought on by a sudden freeze or a sudden thaw.

There was a general decline in the frequency of ice-related floods in the 1950s. This coincided with a considerably warmer trend in winters during that decade. See Figure 1.

Figure 1



TREND IN WINTER TEMPERATURES

* Malton International Airport data, courtesy of T.J. Meyer.

Since then, however, the warming trend has reversed markedly, and there is evidence that this trend towards colder winters will continue for some time. The winter of 1979 and those since have been colder than average during the months of December, January, and February by almost one whole degree Celsius (Figure 1). During these coldest months, however, there have also been occasional and rapid thaws, which continued for several days.

The data show that ice-jam floods have been immediately preceded by rapid changes in weather and by colder than average winters. This trend is expected to contribute to continuing and increasingly frequent ice-related floods in the next ten years. Several improved ice management techniques do exist, however, and this manual indicates how they may be applied successfully to the situations in Ontario.

Typical problem in rural areas.

2
Conditions Causing Ice-Jam Floods

2.1 Two Basic Causes

Ice jams are the result of two basic causes: (i) the freezing up of rivers, and (ii) the break-up of ice during sudden and prolonged changes in weather conditions. In order to predict and prevent floods, an understanding is required of the climatic conditions and the hydraulic factors which can either assist in forming a protective cover or which can cause problems during freeze-up and break-up periods.

2.2 Velocity

Ice jams are caused by the accumulation of too much ice, carried by too much water, and in too short a time. Wherever the rate of supply of ice and water together exceeds the capacity of the river channel to transport it, a jam is imminent.

2.3 Type of Ice

The quality of the ice affects its ability to jam. Hard, blue ice is far more prone to jam than other softer forms, such as slush ice or frazil, which cause freeze-up problems. Slush ice in vast quantities is produced by snow falling and trying to freeze in open water. It has the consistency of wet cotton-batting; it is extremely difficult to handle and may cause severe restrictions of water flow. Frazil is a mass of frozen water particles which stick to each other and everything they touch (Plate 1). Frazil develops when water with no ice cover becomes super-cooled — that is to say, when it cools to below freezing (about -0.05C), but where there is at the same time sufficient air and water movement to: (i) prevent a solid ice cover from forming; and (ii) create an ice-particle blizzard. These particles can build up very rapidly indeed, especially on a rocky bottom, turning into what is called anchor ice. When this happens, the water-carrying capacity of the river can be reduced by 30 per cent within a period of three or four hours, as observed on the Niagara River.

2.4 Likely Sites

Ice jams are most liable to occur when there are:

sudden reductions in the water velocity caused by a widening or deepening of the river channel;

sudden changes in the direction of the flow;

constrictions in the river where it narrows, such as at bridges and their approaches.

2.5 Land Development

Urban development, land drainage, and deforestation all accelerate the rate of run-off — a situation which increases the probable frequency of ice jams.

2.6 Dam Removal

Many streams in conservation areas once had mill ponds. They were formed by dams, which helped control ice jams. Today, however, many of these aids have been removed, filled in or have failed because of poor initial design.

2.7 Freezing from the Bottom Up

Where velocities, flows, and temperatures are all very low, a stream (such as Town Creek in Timmins, for example) will tend to freeze from the bottom up. The channel becomes full of ice, and incoming water flowing over its surface freezes rapidly on top. Given a sudden and prolonged thaw, the increased amount of water from the melting ice has nowhere to go, and floods. As the water softens and erodes the ice, some of it will float and cause jams at constrictions in the river, adding further to the flooding problem.

Plate 1

Frazil particles sticking together.

3 Factors Leading to Flooding During Freeze-up

All rivers in Ontario try to form an ice cover each winter, and this is a good feature, insulating the water from the air. The ability of a river to form an ice cover is dependent on the climate, however, which may fluctuate erratically within a period of only a few days, except in the more northerly rivers where the climate is more constantly cold.

These erratic weather patterns confuse an understanding of the measures required for managing ice jams. For example, two solutions used in the St. Lawrence (International Section) and Niagara Rivers appear, at first sight, to be contradictory. In the St. Lawrence, where a cover once formed will remain all winter, velocities are controlled to make the cover form as quickly as possible. In the Niagara, however, where the climate and storms on Lake Erie may cause frequent break-ups, velocities are controlled to prevent ice-cover formation

A strong, smooth ice cover extending from shore to shore is the most desirable. But it is seldom attained because, while the cover is trying to form, a number of factors can influence the process — such as temperature and wind chill, frazil, anchor ice, velocity, and snowfall. The significance of each is now examined.

3.1 Temperature and Wind-chill

Water temperatures drop more slowly than those of the air. In bays and quiet water areas where velocity is 0.1 metres per second (m/sec) or less, a surface cover will form first. This is important to bear in mind when choosing flood-preventive measures, as the first to freeze is the last to thaw, because of the greater thickness of ice.

Water is also subjected to progressive chilling as it proceeds downstream. Being warmest at the source, flowing waters can, therefore, be several degrees warmer than at the mouth.

With average temperatures of -6C and normal winds of 16 km/hr, river temperatures will drop 0.3C per day. Wind-chill factors, however, can affect these averages markedly, where the water temperature will drop as much as 2C in one day when severe wind-chilling of -30C or more occurs. This combination is reached with the following conditions:

- -18 C air temperature and winds of 19 km/hr (12 mph)
- -12C air temperature and winds of 32 km/hr (20 mph)
- -10C air temperature and winds of 56 km/hr (35 mph)

3.2 Frazil

Once the flowing water at the source has dropped to about 0C, however, wind-chill combinations of -18C or colder will be sufficient to produce super-cooling — and hence frazil, as follows:

- -6C and winds of 28 km/hr (17 mph)
- -10C and winds of 16 km/hr (10 mph)
- -14C and winds of 9 km/hr (5 mph)

Once super-cooling starts, with these wind-chill combinations, the open water area will create about 0.04 cubic metres of ice per square metre per day (7).* During a 50-day period of such weather, one square metre of open water would produce two cubic metres of ice.

When an ice cover forms over flowing water, however, its thickness normally reaches 0.5 metres and, by insulating the water from the air, reduces ice production to only 25 per cent of the amount of open water.

**Numbers in parentheses refer to papers in Section 8 on Selected References.*

3.3 Anchor Ice

Anchor ice begins forming on the bottom of rivers as soon as super-cooling begins. The increased volume of ice and the roughness of its surface raise the water level and impede its flow (frictional resistance). This decreases the flow by as much as 30 per cent in a few hours, which could choke the river and cause severe back-up problems.

3.4 Pancakes

Both anchor ice and frazil may, however, come to the surface and form 'flocs', which freeze together to form a 'pancake'. If the velocity is 0.7 m/sec or less, the pancakes will continue to grow in width and thickness to 100 m or more in diameter, and 40 mm thick in the space of eight hours when there are wind-chills of -18C (7). Eventually, the pancakes will come to rest against an island, bridge-opening or where the current may be slow enough for an ice cover to form. More pancakes following may slide either under or over the first, until they also come to a halt and, if the speed of the current is low enough, they will form the desired ice cover which extends rapidly upstream.

If, on the other hand, there is a long series of rapids upstream, the open water does not form a smooth, strong ice cover but continuously manufactures chunky, uneven ice that does not remain on the surface. This can create hanging dams which cause huge water-level increases. (Plate 2). In the St. Lawrence River, for example, the ice generated in the Long Sault Rapids in the past caused the water level to rise about 15 metres, sometimes flooding the main street of Cornwall. This was prior to the building of the Power Project, which was designed to form an ice cover and eliminate the ice-forming capability of the Rapids.

3.5 Snowfall

Falling directly or blowing into flowing water, snow will accelerate the water's chilling rate, forming masses of slush ice, which tends to cause jamming.

Also, snow falling on top of a layer of insufficiently thick or strong ice will insulate it from the air and may stop further ice growth entirely. This will prevent the formation of a strong ice cover. A weak ice cover will tend to break easily and cause jamming.

3.6 Velocity

The slope of the river affects the velocity and the extent to which sub-freezing air mixes with the water. In other words, the steeper the slope, the greater the increase in chilling rate — and, therefore, the quantities of frazil and anchor ice formed.

3.7 Conclusion

The period when ice is trying to form a cover is a critical one. It should be observed carefully as soon as the water temperature reaches 2C, and watched continuously until the ice extends solidly from shore to shore. Problems will develop with severe wind-chilling conditions and may be compounded by heavy snow. It is possible to have several cycles of freezing and thawing in the watersheds south of Barrie due to the cyclical variations in the weather. Techniques for predicting — and therefore preventing — ice break-up and jams due to these cyclical variations in the weather follow in Section 4.

Plate 2

Heavy surface ridging caused by ice pressures accumulating from ice generated in the rapids on the right of the photograph.

4 Break-up Factors and Predictive Techniques

Ice-jam flooding can be prevented only by first knowing how to predict when break-up is going to occur. Predictive techniques and preventive measures can only be employed, however, if the causes of ice break-ups are understood first.

As stated in Section 2.2, ice jams are caused by the accumulation of too much ice, carried by too much water, in too short a time. But how much is too much, and how short is too short?

Ideally, during the onset of longer days just before the spring equinox, the snow starts melting slowly, gradually increasing the water supply to the river. The increased flow causes the level to rise, increasing velocity; those portions of the river that froze last will lose their ice cover first. If this period continues over a period of several weeks, the main channel of the river opens, leaving a strip of shore ice on either side, which is also eroding and weakening. Because it was thicker to begin with, however, the shore ice takes longer to melt. As the river level rises, some of the shore ice will become detached each day, moving downstream. If the quantity of ice and water mixture is moderate, there will be no problems, even though the river mouths may still be frozen. Two or three weeks of such a weather pattern is enough time both to melt some of the ice and soften the rest. The slowly increasing flow will not, in this way, supply too much ice and water to any constricted sections at one time.

There have been a number of recent break-ups, on the other hand, where the entire process was compressed into several days only. Numerous jams and floods resulted (15). The causes were determined by studying the following factors during a one-week period prior to and a one-week period following the jam.

4.1 Number of Accumulated Degree-days of Melting

One 'degree-day of melting' occurs when the mean of the maximum and minimum air temperatures is +1C. In 1981, an accumulation within a four-day period of sixteen to twenty-three degree-days was enough to cause problems in most watersheds. In others, twenty-four to thirty-two degree-days occurred within a six-day period, which caused problems in all Ontario streams south of a line between Parry Sound and Ottawa (15). The melting rate was four to five degree-days each day. This rate of thawing continued unabated without any more periods of sub-zero weather which normally reoccur in spring.

4.2 Precipitation Likely to Cause Break-up

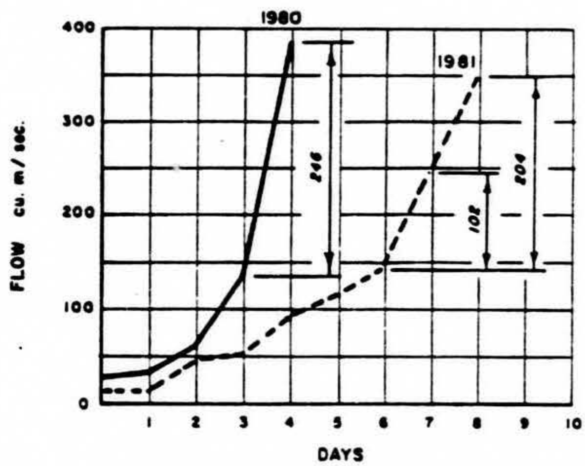
Precipitation generally in Ontario was less than 25 mm over the week prior to ice jams in February, 1981 (except in the Rideau area where it was 27 mm, and in the Moira and Ganaraska regions, where it was 54 mm). Depending on how much melting occurred before the rain, and depending on the water content of any snow already on the ground, the effects will vary. The worst combination is ten degree-days or more of melting, followed by 12 mm or more of rain in a 24-hour period falling on frozen ground or on snow of above-average water content.

Weather data are provided by the Streamflow Forecast Centre, together with a flood advisory, followed by a flood warning if events are likely to be worse than described. Weather forecasting is, however, generally inadequate for more than three days, which gives very little time to prepare for severe changes in weather.

4.3 Rate of Rise in Water Levels Due to Increased Flow

Rate of rise is obtained from the rate of increase in flow. This will vary from one watershed to another, but the factors which cause rising levels are the same and the results similar.

Figure 2

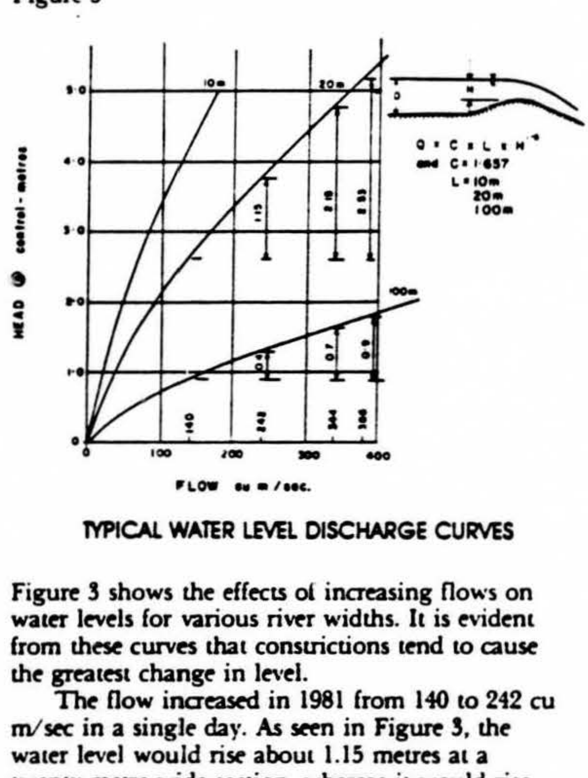


RIDEAU RIVER AT MANOTICK
HYDROGRAPHS DURING ICE BREAK-UPS

In the Rideau system, as an example, there have been two recent violent break-ups — in 1980 and 1981. On both occasions, the flow had been increasing gradually over 3-6 days to about 140 cubic metres per second (Figure 2). Following this gradual increase, however, an increase of 246 cu. m/sec occurred in only one day in 1980; and in 1981, an increase of 204 cu. m/sec in two days.

What must be determined is the lowest rate of increase in flow that has caused break-up. This figure can then be used as the threshold level signalling possible break-up. At this point, the effects on water levels should be examined.

Figure 3



TYPICAL WATER LEVEL DISCHARGE CURVES

Figure 3 shows the effects of increasing flows on water levels for various river widths. It is evident from these curves that constrictions tend to cause the greatest change in level.

The flow increased in 1981 from 140 to 242 cu m/sec in a single day. As seen in Figure 3, the water level would rise about 1.15 metres at a twenty-metre-wide section, whereas it would rise only 0.40 metres at a section 100 metres wide. On the basis of the increase in flow to 344 cu. m sec in two days, the corresponding rises would be 2.15 and 0.74 metres respectively — probably enough to cause a number of problems.

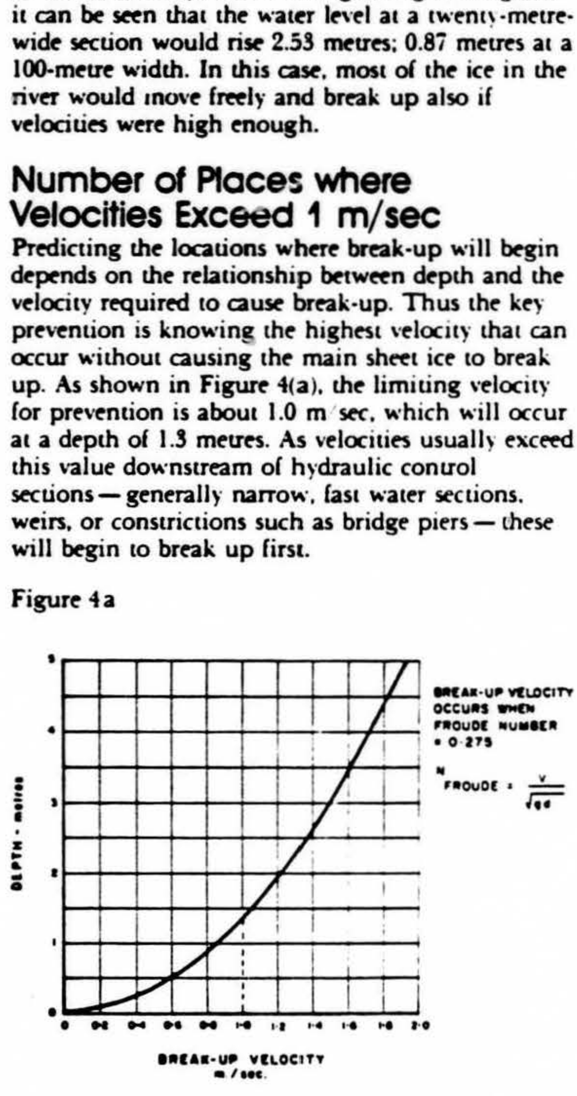
A one-metre rise is usually sufficient to pry the ice cover loose from the river banks in spring; but it would not necessarily move downstream or break up. That action depends on the velocities created or the topography of the river, together with such restraints as islands, bends or ice booms. Such a rate of rise would likely cause problems in narrow

Based on the 1980 increase in flow to 386 cu. m/sec in one day, and referring to Figure 3 again, it can be seen that the water level at a twenty-metre-wide section would rise 2.53 metres; 0.87 metres at a 100-metre width. In this case, most of the ice in the river would move freely and break up also if velocities were high enough.

4.4 Number of Places where Velocities Exceed 1 m/sec

Predicting the locations where break-up will begin depends on the relationship between depth and the velocity required to cause break-up. Thus the key prevention is knowing the highest velocity that can occur without causing the main sheet ice to break up. As shown in Figure 4(a), the limiting velocity for prevention is about 1.0 m/sec, which will occur at a depth of 1.3 metres. As velocities usually exceed this value downstream of hydraulic control sections — generally narrow, fast water sections, weirs, or constrictions such as bridge piers — these will begin to break up first.

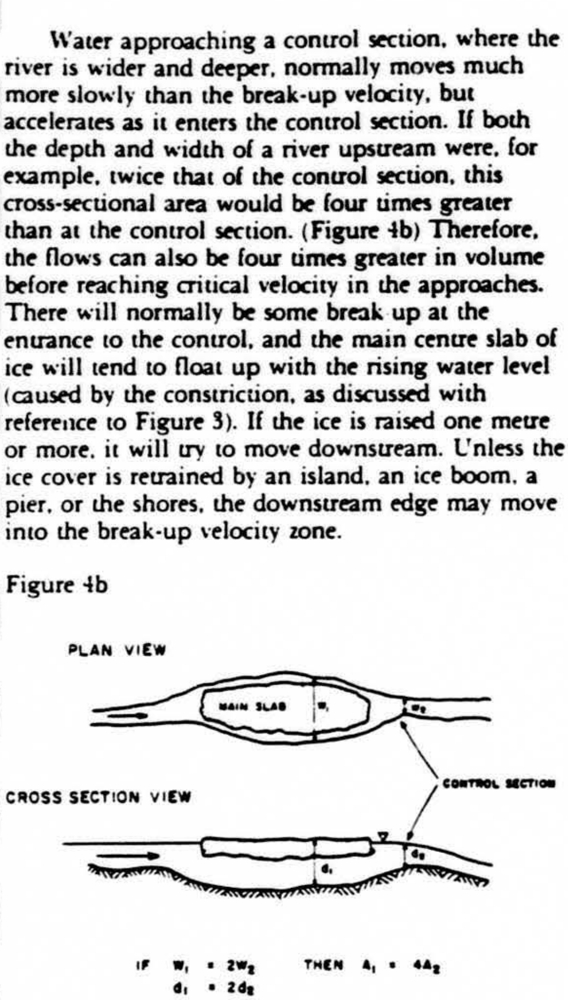
Figure 4a



DEPTH AND BREAK-UP VELOCITY CURVE

Water approaching a control section, where the river is wider and deeper, normally moves much more slowly than the break-up velocity, but accelerates as it enters the control section. If both the depth and width of a river upstream were, for example, twice that of the control section, this cross-sectional area would be four times greater than at the control section. (Figure 4b) Therefore, the flows can also be four times greater in volume before reaching critical velocity in the approaches. There will normally be some break up at the entrance to the control, and the main centre slab of ice will tend to float up with the rising water level (caused by the constriction, as discussed with reference to Figure 3). If the ice is raised one metre or more, it will try to move downstream. Unless the ice cover is retrained by an island, an ice boom, a pier, or the shores, the downstream edge may move into the break-up velocity zone.

Figure 4b

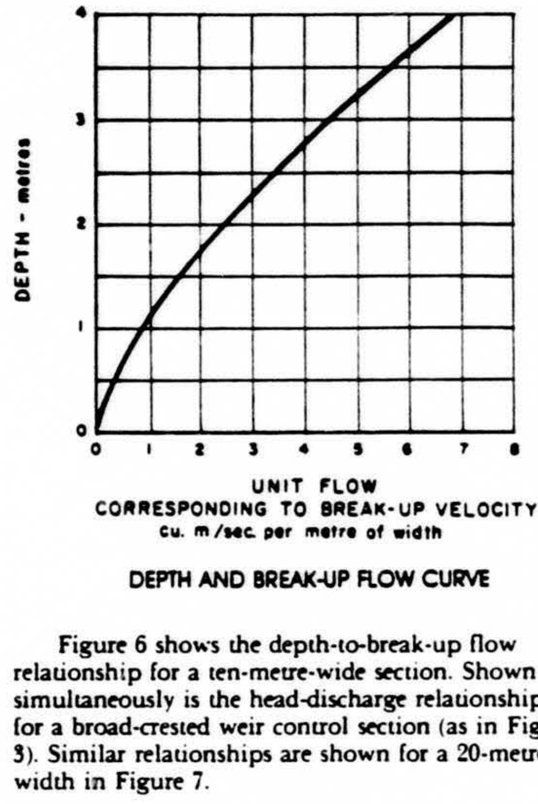


EFFECTS OF CONTROL SECTION

Break-up may also begin at the upstream end of an ice cover if other combinations of depth and velocity shown in Figure 4(a) exist. If the depth is three metres or less and the velocity reaches 1.5 m/sec or more, for example, the water tends to flow over the ice cover, causing pieces to break off and be forced underneath. Nothing can prevent break-up once this happens.

In Figure 5, the depth-to-break-up velocity relationship has been converted for convenience to a depth-to-break-up flow relationship (5). Based on a one-metre-wide section, the break-up flow becomes 1.3 cu. m/sec.

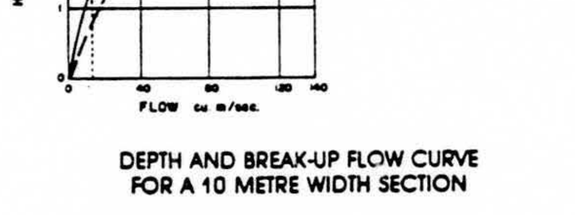
Figure 5



DEPTH AND BREAK-UP FLOW CURVE

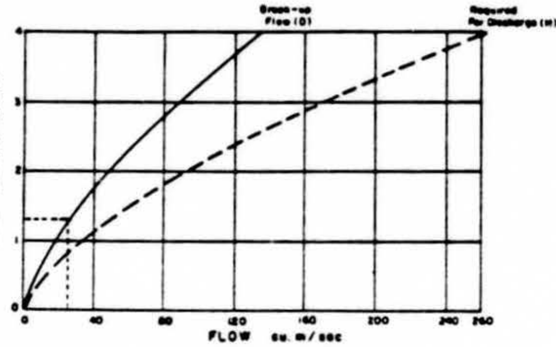
Figure 6 shows the depth-to-break-up flow relationship for a ten-metre-wide section. Shown simultaneously is the head-discharge relationship for a broad-crested weir control section (as in Figure 3). Similar relationships are shown for a 20-metre width in Figure 7.

Figure 6



DEPTH AND BREAK-UP FLOW CURVE
FOR A 10 METRE WIDTH SECTION

Figure 7



DEPTH AND BREAK-UP FLOW CURVE
FOR A 20 METRE WIDTH SECTION

Referring to Figure 6, the flow causing break-up would be 13 cu. m/sec for a 10 m section. It would take place initially at, or downstream from, the control section because the same depth on the control section would indicate a flow of 25 cu. m/sec. As flows increase, however, the velocity at the entrance to the constriction may rise above the break-up value; some ice will break off, and the ice cover will try to move downstream, assisted by the rising levels. As flows continue to increase, there will always be some rate of rise in level and some flow quantity that will cause general disintegration of large portions of the ice cover.

4.5 Partial Break-ups and Local Jams

Rising water levels and increasing velocities cause break-up to begin. Generally, this process takes two to three weeks, and the concentration of ice and water is not sufficient to cause jamming. However, if the process is compressed into a few days, a general disintegration will take place. Between these two extremes, partial break-ups may occur and cause local jams.

Wherever ice pieces accumulate in front of, and/or under, an ice cover, a bending force is created which causes it to fail. Depending on the thickness and quality of the ice, a substantial local jam may result without causing a general disintegration of the entire cover. When this jam breaks, its pieces moving downstream to the next ice-cover may cause a new local jam, and this process may be repeated a number of times before the ice reaches the river's mouth. The time required to complete break-up is extended and there is less severe jamming in any one area.

A likely area for encountering these problems may be selected by following the procedures in Section 5.2.3 on On-site Observations.

4.6 Summary of Predictive Techniques

(a) Problems can be expected if there is an uninterrupted thaw of approximately twenty degree-days of melting during a very short period of time — 3 to 5 days.

(b) Problems are likely in the event of precipitation of 12 mm or more in 24 hours, especially if this follows several days of melting, or if the ground is still frozen, or if the watershed has a large urban area.

(c) Rising water levels and increasing velocities cause break-up — a preliminary indication of possible trouble. A rise of one metre in 24 hours often causes break-up, and will always cause break-up where velocities are greater than 1.0 m/sec.

(d) There is a specific break-up flow for every river, where large portions of the ice cover disintegrate generally. From limited information, it appears that the break-up flow is about 60 per cent of the 100-year flood flow.** Due to variations in ice quality and weather sequences, this percentage may vary from 50 to 70 per cent.

(e) Partial break-ups and local jams may occur due to broken pieces accumulating in front of, and/or underneath, an ice cover, causing it to break by bending.

(f) The worst combination for break-up is a sudden thaw extending over four days, with rain. The resulting jams, however, will depend on how cold the winter has been, whether the ice is hard or soft, thick or thin. In the final analysis, it is the sequence of weather events that is critical.

***Flood levels based on the 1-in-100 year flood flow generally do not take ice jams into account. Thus, due to ice jams, these levels may be equalled or exceeded by a lesser flow (See Section 7.9).*

5 Data Collection and Monitoring of Ice Jam Floods

Before a cost-benefit analysis of remedial measures can be made, certain data need to be gathered. Some information may be obtained from Conservation Authority records; past dates and weather records can often be gathered through newspaper file searches and Environment Canada records.

Often, however, such past records will not, in themselves, provide sufficient information for making a satisfactory cost-benefit analysis, and it will be necessary to implement a monitoring program to gather information for the future. The most important basic piece of data comes from knowing the date the flood began.

5.1 Potential Benefits

Key benefits obviously come from reducing flood damage. The extent of past damage is assessed through answering the following questions:

(a) How many properties were affected, and what were their values?

(b) What was the cost of a clean-up?

(c) How long did the problem last?

(d) How frequent have the problems been?

5.2 Determining the Solution

Effective management techniques vary from one watershed to another — sometimes even within a single watershed. In every case, however, the information required is the same. Before any appropriate and reasonable solutions can be determined, the following information is required:

5.2.1 Weather Information

The crucial two-week period for which information is required spans the week preceding and the week following the date the flood began. During those two weeks:

(a) What were the maximum, minimum, and daily average temperatures?

(b) What was the daily precipitation? Specify snow or rain, and the snow's water equivalent.

(c) How many degree-days of thaw preceded and followed the jam?

5.2.2 Hydraulic Information

Data on the rate of change of inflow and the consequent changes in water level and velocity are particularly important to obtain. In the same two-week period:

(a) What was the variation in the flows and levels in the vicinity of the jam? A graph of these factors must be plotted against time; readings must be taken at least daily, and preferably every six hours.

(b) What was the maximum water level? This will be influenced greatly by river bank height, river width, and ice quality.

5.2.3 On-site Observations

Some of the most useful information of all is simple eye-witness observation of what happened, answering the following questions:

(a) Where did the ice in the jam come from? Was it, for example, a local condition, or did it come from several kilometres upstream?

(b) How thick was the ice?

(c) What kind of ice was it? Hard and blue, soft and white, frazil or slush?

(d) Were there any large lumps, thicker than one metre? These may come from the downstream end of fast-water areas; they are important clues for detecting the solutions.

(e) What was the surface velocity approaching the jam, and how far upstream was it — 1.0 m sec or less? (Timing the ice pieces over a known distance of, say, 30 metres is quite adequate).

(f) Are there any suitable storage locations for ice or water, or both, upstream of the jam?

(g) Observation should be repeated at these locations.

(h) How did the jam dissipate?

5.3 Summary of Monitoring

There are two periods when monitoring should be done:

(a) When the river is trying to form an ice cover (set out in Section 3).

(b) During the week prior to the flood date and the week following — as set out in Section 4.1.

The answers to questions posed in Section 5.1, (a) to (d), will indicate the benefits to be derived from any preventive and remedial techniques.

The cost of these, however, will depend on the solution chosen, based on the on-site observations made and answers given to questions in 5.2.1 to 5.2.3 inclusive.

It is a specialized task selecting the management plan most likely to succeed in any given situation and within any given budget. Success is dependent on accurate information from a good monitoring program.

6
Appraisal of Preventive and Remedial Measures

Prevention of ice jams is more effective than any remedial measures taken after the fact. For cases where the costs of prevention outweight the benefits, or where extraordinary weather conditions overload the system, removal of ice jams is required and the methods are outlined at the end of this section.

There are two basic types of measures for preventing jams:

(a) Weakening and breaking the ice cover into pieces small enough to flow freely.

(b) Controlling the flow of ice and water.

6.1 Weakening and/or Breaking

Before undertaking these activities, it is most important to know where the ice will go when it breaks. There needs to be a safe storage area, such as a lake, swamp or waterfall. Don't just move the problem downstream. This happened in 1982 on the Rideau River, where the Ottawa River normally provides enough storage for its ice. (Plates 3a and 3b). In 1982, however, the Ottawa itself was so ice covered that the Rideau's ice had nowhere to go and tried to flood back over the falls.

Start at the downstream end (normally the mouth), and work upstream so as to provide safety for workers and space for the broken ice to move into, away from the work area.

6.1.1 Dusting

To make the ice easier to break up into small pieces of one or two metres in diameter, it may be weakened first by dusting it with a dark substance like coal, cinders, or sand. Darker substances increase the rate of melting by about 10 mm/day in Ontario (11).

Cost for one application, covering one km of a 30 m wide river, and applied from the air by crop-dusting equipment, is estimated at $1,250 (17).

The effects of dusting may, however, be cancelled out by snowfall, in which case it has to be repeated. Often, also, there is insufficient time for the melting process to be completed sufficiently to break the ice up small enough to flow freely.

Plate 3a

Rideau Falls discharging into Ottawa River.

Plate 3b Looking downstream from Rideau Falls over the Ottawa River ice jam.

6.1.2 Ice Breaking by Blasting

Two essential prerequisites for this job are a properly trained crew and enough time. The time required for ice blasting is one eight-hour day per 1.6 km of ice cover (17).

To produce the desired results, the charge MUST be placed UNDER the ice at the correct depth which will depend on the charge weight (18). For example:

Distance below ice	Weight of charge	Diametre of hole created
1.2 metres	3.6 kg	2.7 metres
1.5 metres	7.3 kg	5.8 metres
1.8 metres	12.3 kg	8.8 metres

Best results are obtained by using ten to twenty charges, spaced one hole-diameter apart, detonated simultaneously.

The explosive used in the example cited was ANFO (Ammonium Nitrate in Fuel Oil). Its low detonation velocity (3,660 m/sec) works better than higher-speed explosives because it causes a wave-action to bend the ice, which is more effective at breaking than is a shattering action (18).

Where streams are more than 15 metres wide, two parallel lines of charges may be necessary in order to break the ice into pieces of one to two metres in diameter. Generally, the smaller the pieces, the more readily the ice will move through constrictions.

Success of blasting is limited by its inherent dangers, the possible lack of a suitable place to store the broken ice, and the fact that it may take too long.

6.1.3 Ice-Breaking by Boat

Normal ice-breakers press down on the cover to break it, and this works well as long as the cover is floating. A fairly recent experiment used a special plow on the front of a tug which forced the ice up and broke it by bending. Neither system works too well, though, if the ice is thick enough to hit bottom — although a regular ice-breaker can at least turn around and back through the ice stern first, provided it has twin screws. Then, the propellers can 'eat' a channel through the jam. The plow variety, however, tends to become a submarine if the ice is grounded.

Most commercial vessels are not satisfactory because they need three metres of depth for safe operation, and even that is not always sufficient, for the bow riding up on the ice drives the stern down, and boats with a two-metre nominal draft may then have problems in a depth of three metres. But small armored tugs specially designed for ice-breaking, which have only a one-metre nominal draft, have been used successfully in the Niagara River since 1964 (12).

An appreciable current to clear the ice from the work area is desirable, as is a safe place for the ice to flow into, of course. Then all that is required initially is to use the ice-breaker to create a narrow channel — or a 'lead' — wide enough for working in the deepest part of the river. Once this is established, the boat's wake widens the channel quite efficiently. It is not essential to clear the area completely, provided the pieces of ice are not larger than two metres in diameter.

Speed is of the essence, readiness is crucial. Often there is a period of only two to four days in which to prevent disaster. The boat should, therefore, be on-site, ready to operate within 24 hours or less. This is achieved by way of a retainer fee for stand-by periods, plus an operational charge for actual use. A captain experienced in ice-breaking is essential too.

Ice-breaking by boat will not be feasible, however, where the river is too shallow, if special vessels are not available, and if there are insufficient overhead clearances at bridges; it may not be feasible either where there is grounded ice.

6.1.4 Combination of Blasting and Breaking by Boat

The City of Ottawa (3) has carried out an effective program for many years, using the following system:

(a) Cut two slots, each 0.3 metres wide, parallel to each other, and generally parallel to the shore, along the edges of the normally-flowing sections of the river.

(b) Blast the slab between the slots — except of course, near bridges or utility crossings. In these places, small outboard-motor boats, with steel-protected hulls, provide the necessary wave action (described in Section 6.1.2).

(c) Timing, again, is crucial here. The flow has to be sufficient to push the ice from the Rideau out into the Ottawa River, but not so rapid as to get it all moving at once.

The disadvantage of this method is that several weeks of preparation are required. Occasionally also, the rate of change in weather is so fast that there is insufficient time to dispose of the ice cover.

6.1.5 Air-Cushion Vehicle (20)

Where the ice cover is hard, but not more than 0.7 metres thick, rapid progress can be made using air-cushion vehicles which create a pressure wave causing the ice to break by bending.

The costs, however, are high; so far only large craft exists, with no evidence of any designs being made for smaller vehicles suitable for Ontario's rivers. Other drawbacks: air-cushion vehicles cannot break grounded ice covers; their effectiveness decreases as the ice becomes softer; their free movement is inhibited by obstacles such as ice ridges and overhead bridges with limited clearance, because they need sloping ramps on the river banks to ascend, circumvent the obstacles, and descend into the river again.

6.2 Control

Controlling the excessive flow of ice and water to prevent flooding is achieved by constructing dams, ice booms, and weirs; or by providing ice-storage areas.

6.2.1 Control Dams

Although costly, control dams are the ultimate solution. By controlling the rate of change in level through controlling flows, the period of time over which break-up occurs can be lengthened. Thus, the amount of ice and water mixture can be controlled.

Control dams in the stream, rather than in the headwaters, are used mainly for controlling ice movement and some of the flow. The mechanism required here it to hold most of the ice coming from upstream, and release only controlled amounts of ice and water as and when the downstream area can accommodate them.

Control dams are expensive but reliable, as evidenced by the St. Lawrence Seaway and Power Project through 24 winters of operation. The system was designed to form an ice cover at velocities of up to 0.7 m/sec over most of Lake St. Lawrence, where the flow can be adjusted to compensate for uncooperative weather conditions. Most of the ice then formed will stay on the surface rather than forming a hanging dam.

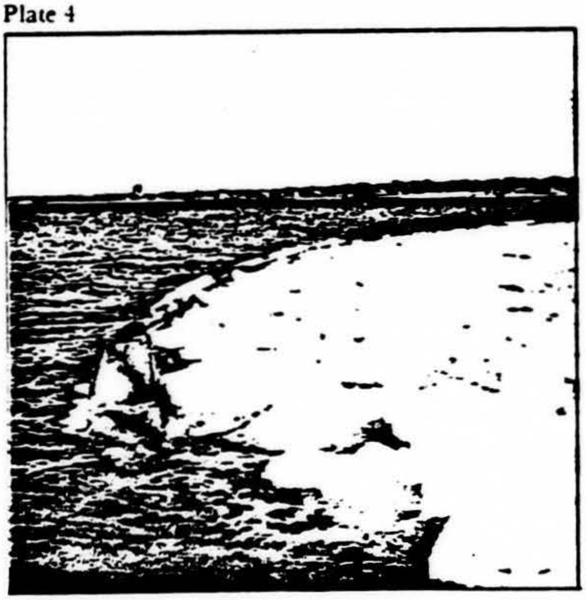
6.2.2 Ice Booms

Installed across a river, ice booms restrict the quantity and prolong the release of ice into potential jamming areas. (Plate 4). They work best when placed at right angles to the direction of the flow, and where velocities are equal to or less than one m/sec — the lower the better.

Ice booms can also accelerate the rate of ice-cover formation as well as helping retain it in the spring so more of the ice will melt in place rather than breaking up.

An ice boom should be designed to float up with the rising water levels, and to submerge also, so that it does not break if the force of the ice becomes too great for it to hold. Some ice will then pass over it; but, as the ice load decreases, the boom's buoyancy will return it to the surface and into effective action again.

Plate 4



Ice boom holding back various types of ice.

6.2.3 Weirs

These help to form ice covers, but are useful only where some form of restraint is provided, such as an ice boom. Without restraint, weirs are liable to spill their entire cover during break-up.

6.2.4 Ice Islands

Man-made or natural, islands provide something for the ice cover to push against after rising levels have removed the restraint formerly provided by the shore. They are not as flexible in their capabilities as ice booms, but have worked well in some instances.

6.2.5 Ice Storage

The Minesing Swamp on the Nottawasaga River and some of the disused channels on the Goulais River are good examples of ice storage areas. Disused channels (oxbows) are often located on the outside of sharp river bends. As ice tends to go straight ahead rather than following river bends, ice will fill these channels, which then act as safety valves for storing a good deal of broken ice.

6.2.6 Ice Removal as a Preventive Measure

Construction equipment can be used as a preventive measure to physically remove at least some of the potentially problem-causing ice. (For more on construction equipment for ice removal, see Section 6.3.1 following).

6.3 Remedial Ice Removal

In some cases, the cost of preventive measures may be too great to warrant the expenditure; in other situations, where prevention works well most of the time, an extraordinary sequence of weather events may, on occasion, still overload the system and cause a jam. Following are measures that may be effective after the jam has already occurred.

6.3.1 Construction Equipment

Ice can be removed physically with bulldozers, back-hoes and draglines. These have all been used successfully on the Ganaraska, Credit and Saugeen Rivers — both in prevention of and removal of jams. They are relatively cheap, but are effective only in locations where the ice cannot affect their operation and where they can actually reach the jam — which is often not possible. While these types of equipment may be effective on small streams and rivers, this is not usually the case in larger waterways.

6.3.2 Blasting Under a Jam

This works best by starting at the downstream end of the jam and working upstream. Placing the charges may be dangerous, and the results will depend on whether the liberated jam has a safe place to go. Ideally, the jam should be released slowly — or else it may result in another jam downstream.

Bombing, howitzer shells, or bazookas are subject to the same drawbacks, and are not likely to work unless they can be detonated under the ice.

Principles of Ice Control

Some of the principles of ice control may be summarized as follows:

7.1 Ice cover protects the water from super-cooling, decreases the total volume of ice produced and prevents the formation of slush ice from snow in the open water, therefore:

CREATE ICE COVER AS SOON AS POSSIBLE OVER AS MUCH OF THE RIVER SURFACE AS POSSIBLE

7.2 Ice cover breaks up because of a rapid rate of change in water level and due to increases in velocity, therefore:

CONTROL ICE MOVEMENT BY CONTROLLING FLOW AND HENCE VELOCITY.

7.3 Ice jams normally last a few days only, therefore:

BY SLOWING THE RATE OF RISE IN LEVELS, AND HENCE THE INCREASE IN VELOCITY, THE AMOUNT OF ICE BROKEN PER UNIT LENGTH OF RIVER CAN BE REDUCED. THERE IS THEN LESS ICE AND LESS WATER COMPETING FOR SPACE IN THE CONSTRICTED SECTIONS AND, THEREFORE, LESS FLOODING.

7.4 When flows cannot be controlled,

PROVIDE ICE MOVEMENT CONTROL SUCH AS ICE BOOMS OR ICE STORAGE AREAS.

7.5 Ice jam floods can be costly and dangerous to break. Therefore:

PREVENTION OF JAMS IS MORE EFFECTIVE THAN ANY OF THE CURES.

7.6 Solving an ice problem in one municipality may merely move it downstream to become someone else's problem. Therefore:

MAKE SURE THE JAMMED ICE CAN BE RELEASED TO A LAKE OR OTHER LARGE STORAGE AREA.

WHEREVER POSSIBLE, ICE BREAKING SHOULD BEGIN AT THE RIVER MOUTH AND WORK UPSTREAM.

7.7 Ice covers break most readily by bending and, once broken, the smaller the piece size the better. Therefore:

WAVE ACTION, EITHER BY BOAT OR AIR CUSHION VEHICLE, WORKS BEST.

7.8 As spring approaches and the days get longer, there is an increase in water supply, a rise in water levels, and an increase in velocity. Therefore:

THE PORTIONS OF THE RIVER WHICH FROZE OVER LAST WILL LOSE THEIR ICE COVER FIRST.

WHEREVER SURFACE VELOCITIES EQUAL OR EXCEED 1 M/SEC, SOME BREAK-UP IS TO BE EXPECTED.

7.9 Every watershed has a critical flow which will cause a major break-up. Based on preliminary data, it appears that:

THE CRITICAL FLOW IS ABOUT 60% OF THE OPEN WATER 100-YEAR FLOW, WITH VARIATIONS OF BETWEEN 50% AND 70% BEING LIKELY DUE TO ICE QUALITY AND WEATHER SEQUENCES.

8
Selected References

There are many papers relating to design problems and research projects but few met the criteria for this manual, which were that:

(a) They must be relevant to the climatic conditions of Ontario;

(b) They must have similar topographical conditions to those in Ontario;

(c) They must have direct application to operational ice problems for use by Resource Managers.

Relevant references are arranged by subject.

8.1 Key to Abbreviations in Succeeding Paragraphs 2 to 5

I.M.S.P.
: Ice Management Seminar Proceedings — January 30, 1980 — Produced by the Ontario Ministry of Natural Resources, Southwestern Region, London

W.H.R.R.I.
: Proceedings of the Workshop on Hydraulic Resistance of River Ice — September, 1980 Produced by the National Water Research Institute (formerly C.C.I.W.), Burlington, Ontario — edited by G. Tsang and S. Beltaos

N.R.C.
: National Research Council, Canada.

I.A.H.R.
: International Association of Hydraulic Research Ice Symposia
: — Iceland, 1970; Hanover, New Hampshire, 1975;
: — Sweden, 1978; Quebec, 1981

C.C.R.E.L.
: Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire

C.J. of C.E.
: Canadian Journal of Civil Engineers

A.S.C.E.
: American Society of Civil Engineers

I.H.D. — W.M.O.
: International Hydrologic Decade combined with the World Meteorological Association, Banff, 1972

8.2 Case Studies of Chronic Problem Rivers in Ontario

REFERENCE NUMBER

THAMES RIVER FLOOD — MARCH, 1979. I.M.S.P., 1
by B. Bennett, LOWER THAMES VALLEY C.A.
History of major ice-jam related floods in 1951, 1965, 1968 and 1979 in detail. Comments on the use of dynamite and an ice-breaking tug. Solutions required upstream of Chatham.

MOIRA RIVER — BELLEVILLE, I.M.S.P. 2
by K. Lathem, Crysler and Lathem Ltd.
Chronic problems in Belleville. Large quantities of sheet ice move into lower velocity areas where "hanging dam" conditions prevail, i.e., the supply of ice and water is much faster than channels can discharge it into the Bay of Quinte. Ice storage works constructed in 1977-78 for about $2.5 million. Preliminary results are mostly satisfactory, but evaluation is still lacking.

"RIDEAU RIVER" — OTTAWA 3
by W. Frietag, City of Ottawa
The City of Ottawa undertakes an annual ice-breaking program to alleviate flooding on the Rideau River at a cost of approximately $125,000.

Keys are cut in the ice at bridges and other locations where blasting is prohibited. Broken ice is flushed downstream by increased flows from an upstream reservoir. Mechanical saws, dynamite, boats and up to 30 men are involved and careful planning is essential.

8.3 How and Where Ice Jams Form and Collapse

8.3.1 During Freeze-up

REFERENCE NUMBER

WINTER REGIME OF RIVERS AND LAKES. CCREL, APRIL, 1971 4
MONOGRAPH III — Bla
by B. Michel — Laval U., Sept. 1980

This is the most comprehensive publication for understanding ice formation processes in their various forms and the behaviour of rivers and lakes during freeze-up and break-up.

RIVER ICE HYDRAULICS, W.H.R.R.I. 5

A discussion of resistance of ice covers, ice jams at break-up and ice cover formation. An important missing element was cited as "lack of ice jam thickness measurements". Of particular value is Figure 3 on Page 189, which shows the "universal stability diagram" for ice covers during freeze-up, break-up (jam) and break-up (solid cover).

ST. LAWRENCE POWER PROJECT ICE MANUAL 6
ONTARIO HYDRO — 1968, UNPUBLISHED DOCUMENT
by D.M. Foulds

Design criteria and operating performance are compared for a period of about 20 years. In general, the 0.7 m/sec average velocity for ice cover formation has worked well. Some years the ice cover was smoother than expected and in a few years the opposite, depending on weather conditions at the time of formation. Operating instructions are given for recognizing likely jamming conditions.

STUDIES OF RIVER AND LAKE ICE, Volumes 2, 3 and 4 7
ONTARIO HYDRO, Unpublished — I.H.D. Data

Three years of weather and ice data are available for the Niagara River between Fort Erie and the Falls. Extensive information is provided on frazil formation, anchor ice and ice volumes, but there are few conclusions.

8.3.2 During Break-up

REFERENCE NUMBER

"MECHANICS OF ICE JAMS" I.M.S.P. 8
by D.B. Hodgins, J.F. MacLaren Ltd.

This is a good summary paper of the state of the art and contains an extensive bibliography (57 pages), some of which may be misleading. Unresolved problems relate to the strength of unconsolidated ice jams, their thickness and roughness.

"FROUDE CRITERION FOR ICE BLOCK STABILITY" 9
GLACIOLOGY JOURNAL, VOL. 13, No. 68, 1974, by G.D. Ashton — CCREL

Stability analysis on floating ice blocks are well set out here, and it is really an extension and improvement on the work of Uzuner and Kennedy.

"RIVER ICE JAMS", THEORY, CASE STUDIES AND APPLICATIONS — January, 1982 10
by S. Beltaos, NATIONAL WATER RESEARCH INSTITUTE

A "State-of-the-art" summary of what can be done to predict flood stages at this time. The limitations are clearly set out and it seems unlikely that these will be resolved in the near future.

"BREAK-UP AND CONTROL OF RIVER ICE", I.A.H.R. 11
by C.P. Williams — N.R.C.

Good paper for the Ontario scene. Cites the importance of variable weather sequences, the added problems of northward flowing streams. Information on "dusting" to accelerate break-up.

"NIAGARA RIVER ICE MANUAL", Ontario Hydro, 1964 12
by D.M. Foulds, Unpublished.

Historical summary of ice problems and the weather sequences which caused them. Worst problems caused by weather variability from cold to warm and back again with the attendant storms being the most significant. Operating instructions for monitoring ice movements in order to recognize major problems developing, preventative and remedial measures.

REFERENCE NUMBER

"NIAGARA RIVER ICE CONTROL", EASTERN SNOW CONFERENCE, February 1967, by D.M. Foulds. 13

Operating experiences during some horrendous storms, ice removal by ice breakers and by manipulation of river levels and velocity, success of ice boom.

DYNAMICS OF ICE FORMATION IN THE UPPER NIAGARA RIVER 14
I.H.D. — W.M.O., Banff 1972, by R.S. Arden and T.E. Wigle

Results of part of the studies referred to in reference 7 above. Excellent pictures of frazil and a good description of the difficulties in ice research, especially in relation to trouble-free operation of instruments. Recording water temperature thermometer and net radiometer instruments give good estimation of onset of frazil formation and anchor ice.

"SOUTHERN ONTARIO ICE JAM STUDIES", O.M.N.R., C.A.W.M.B. 15
by D.M. Foulds

Report on the very unseasonal break-up of Southern Ontario streams in February, 1981.

8.4 Effects of Ice Jams on Water Levels

Caution is advised in reading the literature and in trying to apply results from one area to another. There is a great variation in effects, depending on such variables as the slope of the water surface, the steepness and height of the river banks, the supply of ice and the prevailing climate and its variability. For example, W.L. Knowles speaks of rises of 1 to 2 metres on the Thames, whereas D.M. Foulds at Niagara speaks of 20 metres (13).

"EFFECTS OF RIVER ICE ON STAGE", I.M.S.P. 16
by W.L. Knowles, J.F. MacLaren Ltd.

In rivers with very mild gradients, ice accumulation as far as 5 to 10 miles downstream can have an effect on river stage at a gauging station. Illustrations are presented for Dresden, Thamesville and Chatham. Utilization of frequency data for predicting 1-in-100 year stage levels or delineating the 1-in-100 year floodway must receive careful evaluation.

8.5 Remedial Measures

REFERENCE NUMBER

"METHODS OF REMOVING ICE JAMS", I.M.S.P. 17
by S.L. Denhartog, C.C.R.E.L.

A good summary paper on the appropriate times for trying to remove ice jams, as well as the possible consequences. Methods for removal are given together with some costs for using machinery, dusting, ice breaking, ships and blasting. Advantages and disadvantages are discussed.

"USE OF EXPLOSIVES IN REMOVING ICE JAMS 18
C.C.R.E.L., M.P. 1021, 1970
by G.E. Frankenstein

Excellent practical information on how to blast ice covers for maximum cracking. Optimum depths for and size of charges, speed of explosive (low) and resulting hole sizes. Break-up of about 1.6 km crew/day is maximum.

"BLASTING SHEET ICE AND ICE JAMS", I.M.S.P. 19
by D. Mairs, C.I.L., Toronto

Methods of blasting, advantages of various types of explosives and detonation, proper handling and safety techniques.

"AIR CUSHION VEHICLES" (ACVs), I.M.S.P. 20
by R. Wade, Canadian Coast Guard

"Rates of 25 km/hr break-up of 1 metre thick ice were obtained with a 50-tonne vehicle 20 m long by 10 m wide"; i.e., good for big rivers but not small ones. Current models are too large and too expensive to build and operate unless they can be used for other purposes throughout the year.

"SPECIFICATIONS FOR ACVs", I.M.S.P. 21
by D. Jones

Good proposal for smaller vehicles, but none available or in production.

"USE OF ICE BREAKING BOATS", EASTERN SNOW CONFERENCE, 1967 22
by D.M. Foulds

Used on the Niagara River and also used in the Lower Thames for many years.

The Southern Indian Lake Impoundment and Churchill River Diversion[1]

R. W. Newbury, G. K. McCullough, and R. E. Hecky

Department of Fisheries and Oceans, Freshwater Institute, 501 University Crescent, Winnipeg, Man. R3T 2N6

Newbury, R. W., G. K. McCullough, and R. E. Hecky. 1984. The Southern Indian Lake impoundment and Churchill River diversion. Can. J. Fish. Aquat. Sci. 41: 548–557.

The 242 000-km^2 Churchill River basin extends across the northern half of Alberta, Saskatchewan, and Manitoba. In 1976, hydraulic control structures were completed to divert 75% of the natural river flow of 958 m$^3 \cdot s^{-1}$ across the drainage divide separating the Churchill and Nelson river basins in northern Manitoba. The diversion flows follow 300 km of tributary valleys southward to the Nelson River channel where a 30-yr, 10 000 MW hydroelectric scheme is being developed. The diversion was accomplished by damming the northern outlet of Southern Indian Lake, a 1977-km^2 riverine lake on the Churchill channel (latitude 57°N, longitude 99°W). The dam caused a 3-m impoundment above the historical lake levels, which flooded 414 km^2 of the backshore zone. Permafrost, or permanently frozen ground, is widespread in the uplands surrounding the lake. As bedrock occurred on only 14% of the postimpoundment shoreline, severe erosion of the frozen backshore deposits is now underway. A long period of instability is anticipated on lake shorelines and in river valleys affected by the altered hydraulic regime. Although the whole-lake water exchange time was increased by only 41% by the impoundment, the circulation patterns and exchange times in individual basins of the lake were changed dramatically when the Churchill waters were diverted at the southern end of the lake. The effects of the changing regimes on the aquatic habitats and fisheries of Southern Indian Lake have been investigated in pre- and post-impoundment studies undertaken by the Freshwater Institute of the Department of Fisheries and Oceans.

Le bassin de la rivière Churchill, d'une superficie de 242 000 km^2, traverse la moitié septentrionale de l'Alberta, de la Saskatchewan et du Manitoba. En 1976, étaient terminées les installations de contrôle hydraulique pour détourner 75 % du débit naturel de 958 m$^3 \cdot s^{-1}$ de cette rivière à travers la ligne de séparation des eaux des bassins de la rivière Churchill et du fleuve Nelson, dans le nord du Manitoba. Les eaux ainsi détournées suivent 300 km de vallées tributaires vers le sud en direction du chenal du fleuve Nelson, où l'on a mis en œuvre un programme hydroélectrique de 30 ans, d'une production de 10 000 MW. La dérivation a été réalisée à l'aide d'un barrage bloquant l'émissaire nord du lac Sud des Indiens, un lac fluvial du chenal de la rivière Churchill (57° de latitude N et 99° de longitude O). Le barrage a causé une élévation du niveau de l'eau de 3 m au-dessus des niveaux historiques du lac, avec inondation de 414 km^2 de la zone d'arrière-plage. Le pergélisol, ou terrain gelé en permanence, est étendu dans les terres hautes entourant le lac. Comme la couche rocheuse n'est présente que sur 14 % seulement de la ligne de rivage après retenue des eaux, il se produit actuellement une importante érosion des dépôts de l'arrière-plage congelés. On s'attend à une longue période d'instabilité sur les rives du lac et dans les vallées fluviales affectées par un régime hydraulique modifié. Bien que le temps d'échange de l'eau de tout le lac n'ait augmenté que de 41 % à la suite de la retenue, les caractéristiques de la circulation et les temps d'échange dans les bassins individuels du lac subirent des changements dramatiques quand les eaux de la rivière Churchill furent détournées à l'extrémité sud du lac. Les effets de régimes changeants sur les habitats aquatiques et les pêches du lac Sud des Indiens ont été étudiés avant et après la retenue des eaux par l'Institut des eaux douces du Ministère des Pêches et des Océans.

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Southern Indian Lake lies in a shallow bedrock basin on the Churchill River in northern Manitoba (latitude 57°N, longitude 99°W, Fig. 1). In 1976, a dam built across the lake outlet raised the lake level 3 m to facilitate the diversion of the Churchill River southward to hydroelectric generating stations on the Nelson River. Preliminary studies of the effects of the impoundment and diversion were previously undertaken by the University of Manitoba (unpubl. data), by private consultants (Van Ginkel and Associates, Winnipeg, Man., unpubl. data; Underwood-McLellan and Associates, Winnipeg, Man., unpubl. data), and several components of the federal–provincial Lake Winnipeg, Churchill and Nelson Rivers Study Board (LWCNR 1975). The Freshwater Institute (FWI) of the Department of Fisheries and Oceans was the lead agency for the fisheries and limnology impact assessment under the LWCNR (1975). In 1976, the FWI began a long-term study of the Southern Indian Lake impoundment to assess the current predictive capability as expressed in the LWCNR and to generate new predictive capability that would increase quanti-

[1]This paper is one of a series on the effects of the Southern Indian Lake impoundment and Churchill River diversion.

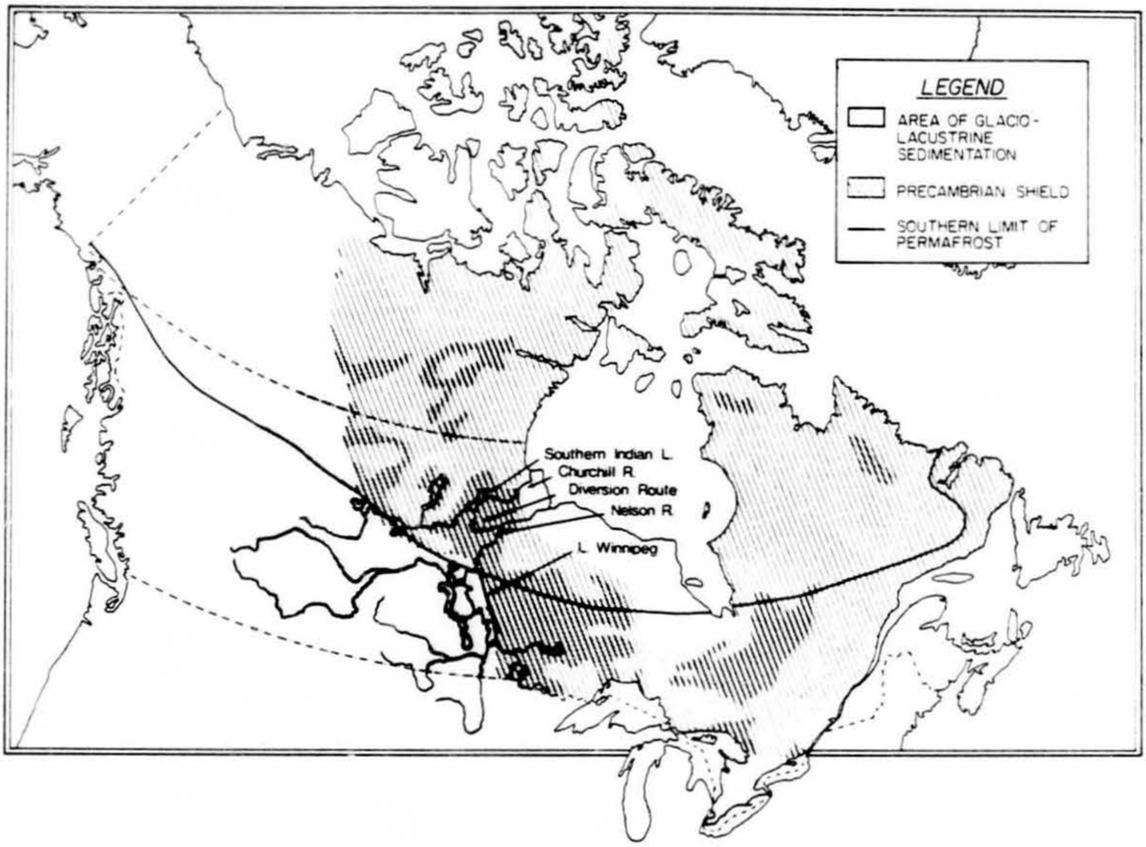

FIG. 1. Geographical setting of Southern Indian Lake and the Churchill River Diversion.

ative precision of future reservoir and diversion assessments. Hecky et al. (1984) summarized how well the follow-up studies have fulfilled these purposes to date.

A brief summary of the hydroelectric development, geography of the Southern Indian Lake region, and changes in the hydraulic regime of the Churchill River and Southern Indian Lake is presented in this paper as background to the FWI studies.

The Churchill–Nelson Hydroelectric Development

The steep granitic and gneissic bedrock river channels of the Precambrian Shield in central Canada provide many attractive sites for hydroelectric development. Over three quarters of Canada's hydroelectric energy is produced on the Shield in generating stations with a total installed capacity of 41 000 MW (Government of Canada 1980). The Churchill–Nelson generating stations are located on the western arm of the Shield in northern Manitoba (Fig. 1). The Churchill and Nelson rivers gather water from over 1.4 million km^2 of the interior plains of western North America before flowing northeasterly to Hudson Bay through a heavily glaciated trough that lies along the boundary of the Churchill and Superior bedrock provinces of the Shield. The Nelson River falls from elevation 218 m (MSL) at Lake Winnipeg, dissipating 5406 MW of hydraulic power over [illegible]80 km of channel with a mean discharge of 2480 $m^3 \cdot s^{-1}$. Prior to diversion the portion of the Churchill River involved in the Manitoba development fell 255 m between Southern Indian Lake and Hudson Bay, dissipating 2702 MW of power over 460 km of channel with a mean discharge of 1011 $m^3 \cdot s^{-1}$ (LWCNR 1975).

The first generating station on the Nelson River was completed in 1961 to supply power to the isolated International Nickel Company mine and refinery at Thompson, Manitoba. In 1964, federal–provincial studies were initiated to examine the feasibility of developing further generating stations on the Nelson and Churchill rivers for markets in southern Canada and the northern United States with the assistance of a 927-km direct-current transmission line sponsored by the Government of Canada. In 1966, nine dam sites were identified on the Nelson River, including the Jenpeg site below Lake Winnipeg, to regulate levels of this 24 400-km^2 lake for dependable midwinter flows (Fig. 2). Generating stations on the Churchill River were not recommended. Instead, several diversion schemes were proposed that would combine the Churchill and Nelson River systems to increase flows through the Nelson River dams. Power demand projections by Manitoba Hydro predicted that the Nelson plants with the Churchill diversion would be fully developed by the early 1990's. The decision to proceed with the transmission facilities and full Nelson development was announced in 1966 in conjunction with a public campaign to stimulate electrical consumption and develop power exports to the United States.

In 1976, the mean level of Southern Indian Lake was raised

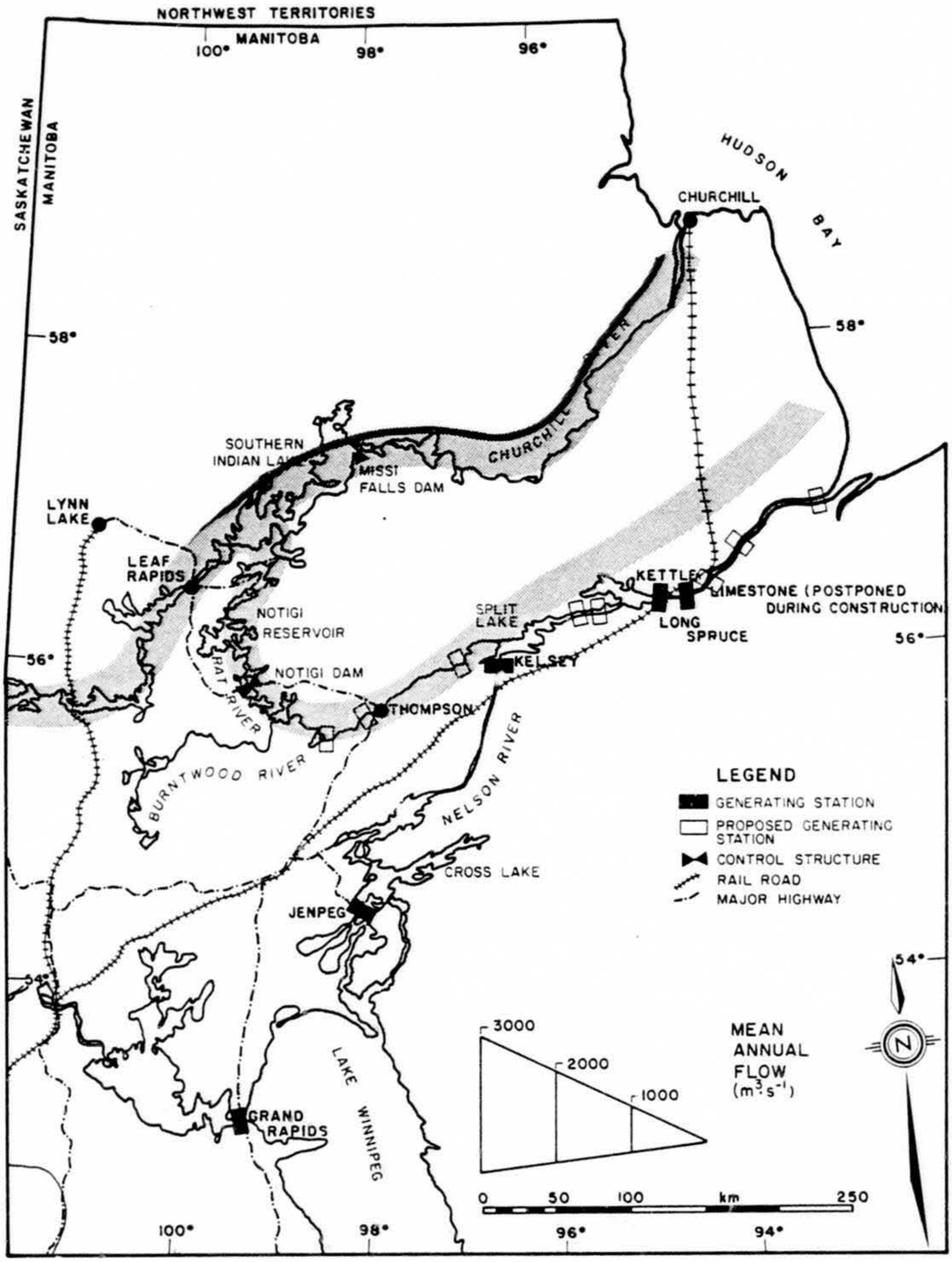

FIG. 2. Churchill and Nelson rivers hydroelectric development, indicating the altered flow regime of the rivers. Dark tone indicates relative magnitude of lower Churchill River discharge remaining after diversion; mid-tone indicates portion of Churchill River discharge diverted at Southern Indian Lake; light tone indicates Nelson River discharge.

3 m, diverting 75% of the Churchill River flows southward into the lower Nelson River valley (Fig. 2 and 3). The Jenpeg power and regulation dam was completed in 1977, and generating stations were constructed above and below the second crossing of the Hudson Bay railway line at Kettle Rapids (1272 MW) and at Long Spruce Rapids (980 MW). A third partially constructed Nelson River dam at Limestone Rapids was abandoned in 1979 because the predicted growth in electrical consumption and power exports did not occur. In 1980, the Nelson River construction program was suspended indefinitely.

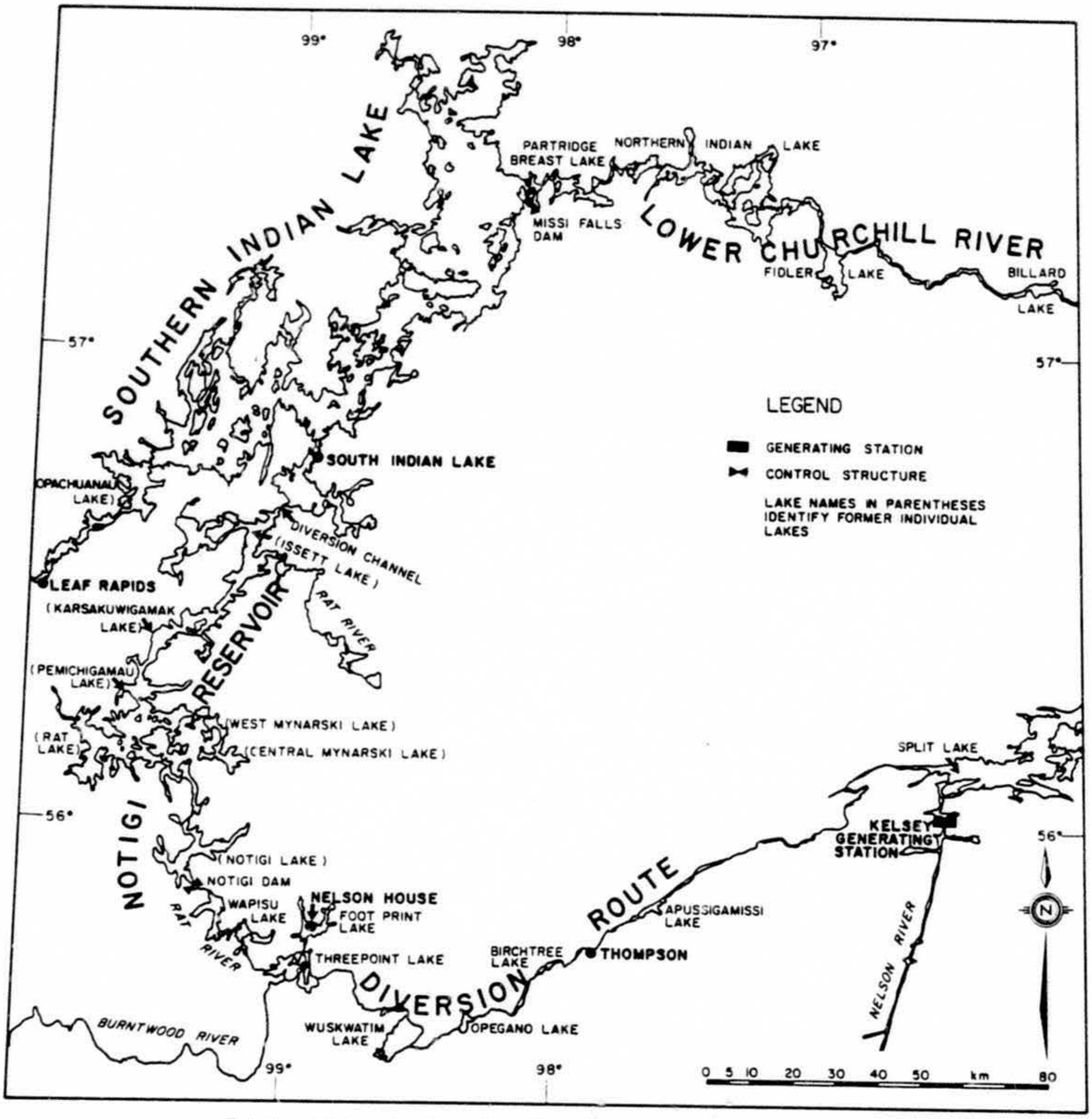

FIG. 3 Lakes and communities affected by the Churchill River diversion.

The Southern Indian Lake Region

The Southern Indian Lake region is underlain by the rugged bedrock surface of the Precambrian Shield. Innumerable lakes and wetlands connected by small meandering streams cover approximately one third of the land surface. Southern Indian Lake is composed of several irregularly shaped basins separated by narrow channels and islands. For the FWI studies, the basins have been designated as regions 0–7 (Fig. 4).

Massive granitic and gneissic rocks extend generally throughout the Southern Indian Lake region, with narrow belts of meta-volcanic rocks rich in sulphides on the western shore of regions 1 and 3. Several small outcrops of meta-sedimentary rocks occur along the western shore of region 4, the southern shore of region 6, and at isolated sites in regions 1 and 2. Mineralized zones associated with pyrrhotite and pyrite minerals occur infrequently throughout the lake (Davies et al. 1962; Pohlinger 1972).

During the most recent period of continental glaciation, the Southern Indian Lake region was covered by ice which advanced southward from the Keewatin center west of Hudson Bay. A thin layer (<10 m) of dense basal tills was deposited in isolated pockets of the heavily glaciated bedrock surface surrounding the lake. After several periods of re-advance and withdrawal, the final retreat from the lake basins occurred between 8000 and 10 000 yr ago. During the retreat, extensive areas of glacio-fluvial sands and gravels were deposited on the uplands surrounding region 5 and the northern half of region 4. The granular deposits form a ribbed and rolling upland of kames and eskers. Varved silty clays up to 20 m thick fill depressions and cover much of the uplands surrounding the basins of the southern two thirds of the lake. The clays were deposited under a northern arm of glacial Lake Agassiz that extended down the Churchill valley into region 4. The local relief is greater in the southern areas where knolls and ridges of bare bedrock outcrop are separated by poorly drained wetlands that formed on the flat glacio-lacustrine deposits. Forest and *Sphagnum* moss peat deposits up to 3 m in depth have accumulated since deglaciation.

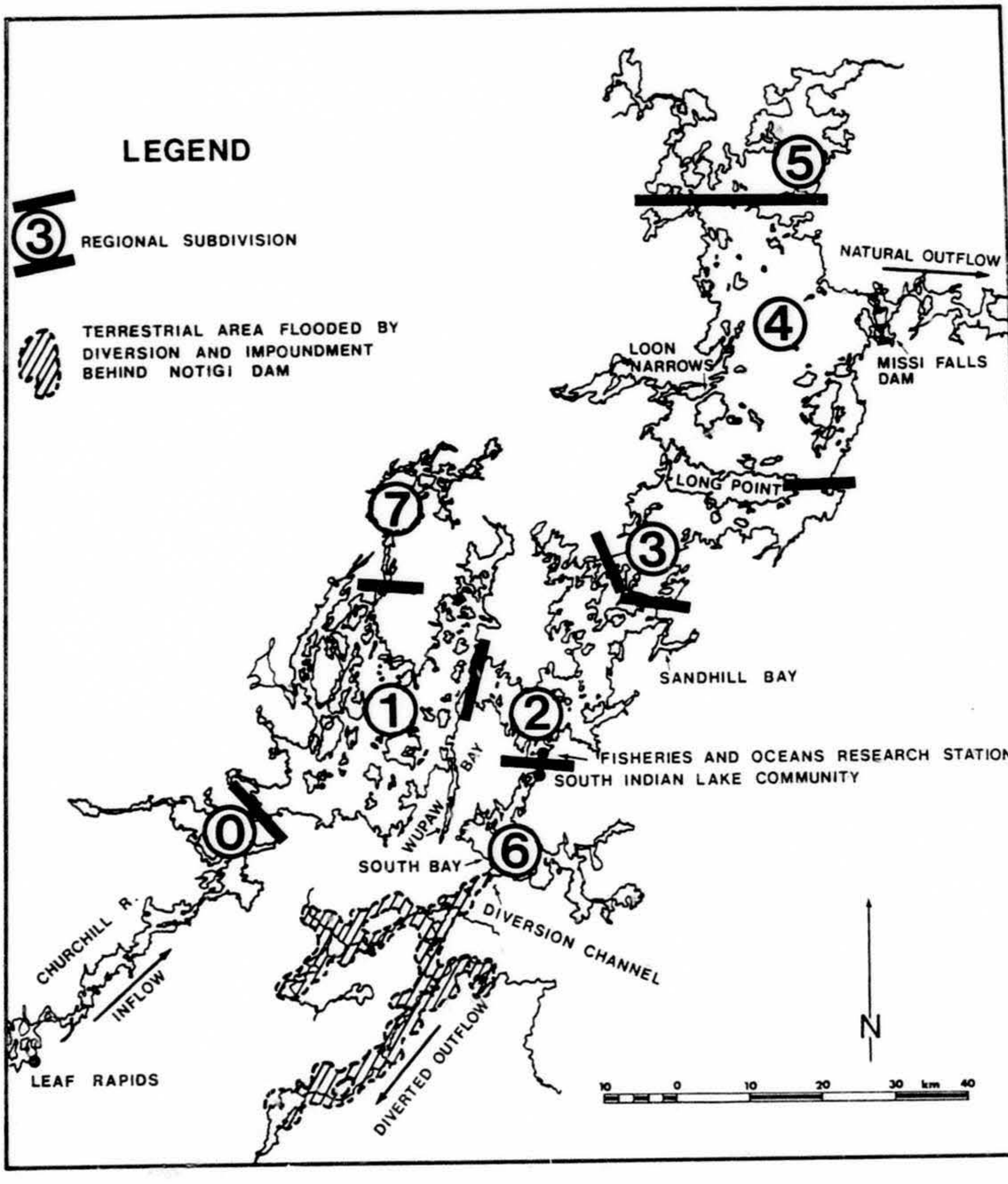

FIG. 4. Regional divisions of Southern Indian Lake.

The climate of the region is continental, consisting of long cold winters and short cool summers. In winter, severely cold waves of polar continental air move southeastward across the region. In summer, the weather pattern is characterized by frequent cool periods following eastward-moving cyclones. The mean annual temperature at the community of South Indian Lake is −5°C. Average monthly temperatures vary from −26.5°C in January to +16°C in July. The annual precipitation of 430 mm is generally associated with frontal weather systems. One third of the precipitation occurs as snow during the average 200-d mid-October to late May snow cover period. The average accumulated depth of snow is 60 cm. The period of ice cover on Southern Indian Lake lasts from early November to late May. An ice cover up to 1.5 m in thickness develops in areas removed from the Churchill River flow that are blown clear of snow.

The Southern Indian Lake region lies in the wide band of boreal forest or taiga that crosses midlatitude Canada. Black spruce (*Picea mariana*) is the predominant tree species in most areas. It forms pure, closed stands of 10- to 15-m-tall trees on the sloping uplands, and open to sparse stands of stunted 3- to 6-m-tall trees on the poorly drained lacustrine deposits. Pure stands of tamarack (*Larix laricina*) occur in most wetlands. Jackpine (*Pinus banksiana*) is abundant on the well-drained granular deposits in the northern third of the basin. Areas of deciduous species are interspersed in the conifer forests, particularly where recent fires have occurred. Common species are

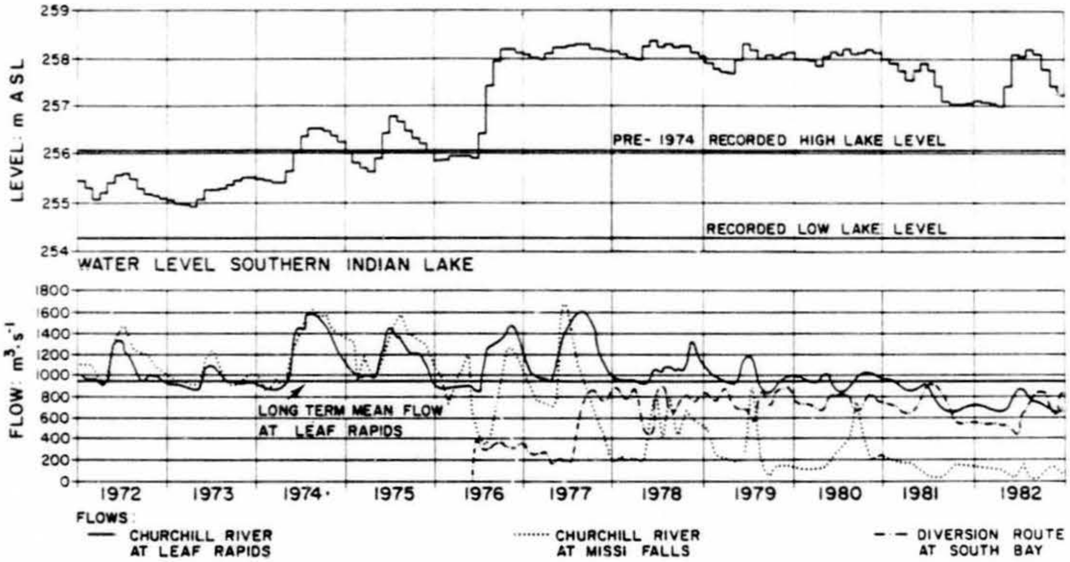

FIG. 5. Southern Indian Lake levels, Churchill River flows, and diversion flows from 1972 to 1982.

aspen (*Populus tremuloides*), balsam poplar (*Populus balsamifera*), paper birch (*Betula papyrifera*), willow (*Salix* spp.), and alder (*Alnus* spp.). A mixed deciduous–coniferous ecotone commonly occurred along the preimpoundment shorelines.

Permafrost is widespread in all terrain types surrounding the lake, with the exception of the glacio-fluvial deposits in the northern half of region 4 and region 5. The depth of permafrost exceeds 10 m in upland areas (Brown 1978) but it does not exist under the main lake basins or under the narrow valleys of major tributaries because of the thermal influence of the water bodies. The temperature of the permafrost ranges from −0.2 to −0.8°C. The thickness of the active layer varies from 0.5 to 2 m, depending on local terrain conditions. Landforms associated with permafrost conditions that occur in the regions of glacio-lacustrine deposits include palsas and collapse scars, raised peat plateaux, and black spruce islands in lowland bogs.

There is evidence of a major movement of people into the region about A.D. 700 who were probably early representatives of the present population of Swampy Cree, one of the Algonkian tribes that occupy the boreal forest zone of the Precambrian Shield. Archaeological investigators of isolated campsites in regions 4 and 5 and on the diversion route south of region 6 have reported artifacts that date from before 4000 B.C. (LWCNR 1975). The Danish expedition led by Jens Munck overwintered in the Churchill estuary in 1619–20, leading the Swampy Cree to name the river "Mantawesepe" or the "river of strangers" (Faries and Watkins 1938). In 1686, fur traders of the Hudson's Bay Company renamed the river the "Churchill" in honor of the Duke of Marlborough, a senior officer of the Company. The present community of South Indian Lake is located on the eastern shore of the narrow channel leading from region 2 to region 6. Many of the local people moved to Southern Indian Lake from the Nelson House Reserve on the Rat River early in this century. Previously, the lake had been used only seasonally as part of the traditional hunting and gathering region of the Nelson House Band. In 1922, a trading post on the lake was established by the Hudson's Bay Company. In 1942, a commercial fishery and registered trapline system were organized. For many years prior to the impoundment, the lake was the largest producer of export-grade lake whitefish (*Coregonus clupeaformis*) in northern Manitoba, with annual catches in excess of 1 million pounds (Bodaly et al. 1984b). Following the announcement of the Churchill River diversion scheme, the 600-member community was relocated to the eastern shore of the South Indian narrows near a newly constructed airstrip. A road between the mining town of Leaf Rapids and the now-abandoned construction camp at the diversion channel on the south shoreline of region 6 was built in 1974.

The Southern Indian Lake Impoundment

The 3-m impoundment of Southern Indian Lake was accomplished by blocking its main outlet channel at Missi Falls, in region 4, with a rock-fill dam and constructing a concrete spillway on a smaller nearby outlet channel. Water may be released into the lower Churchill River during periods of high inflow, which would otherwise cause the lake to exceed the licensed regulation limit of 258.17 m (MSL) while the diversion is operating at a licensed maximum capacity of $850\ m^3 \cdot s^{-1}$. Minimum releases to the lower Churchill River are $14.2\ m^3 \cdot s^{-1}$ during the open-water season and $42.5\ m^3 \cdot s^{-1}$ during the ice-cover period.

Lake Levels before and after Impoundment

Lake levels, Churchill River flows, and diversion flows for the period 1972–82 during which the diversion and impoundment occurred are summarized in Fig. 5. The recording of open-water lake levels began in 1956 but there are several ensuing years in which incomplete records were obtained. The maximum and minimum daily lake levels based on the 1956–76 period record were 256.08 m (MSL) and 254.31 m (MSL). Simulated monthly lake levels based on flow records for the 1912–67 period using the natural elevation–discharge relationship at Missi Falls indicate that the long-term mean level has been 254.93 m (MSL) and with maximum and minimum levels of 256.79 m (MSL) and 253.82 m (MSL) (LWCNR 1975).

The Missi Falls dam and the outlet control structure were built during the 1973–76 period. Initially the smaller southern outlet channel was blocked to dewater the control structure site,

TABLE 1. Preimpoundment and postimpoundment areas (km^2) and volumes (10^9 m^3) of regions of Southern Indian Lake (after McCullough 1981).

	Region								Total
	0	1	2	3	4	5	6	7	
Postimpoundment area	92	521	279	252	741	307	139	60	2391
Preimpoundment area	77	475	223	200	625	211	120	46	1977
Postimpoundment volume	0.63	5.27	2.45	2.42	9.64	1.81	0.81	0.35	23.38
Preimpoundment volume	0.38	3.78	1.70	1.74	7.59	1.04	0.42	0.19	16.84

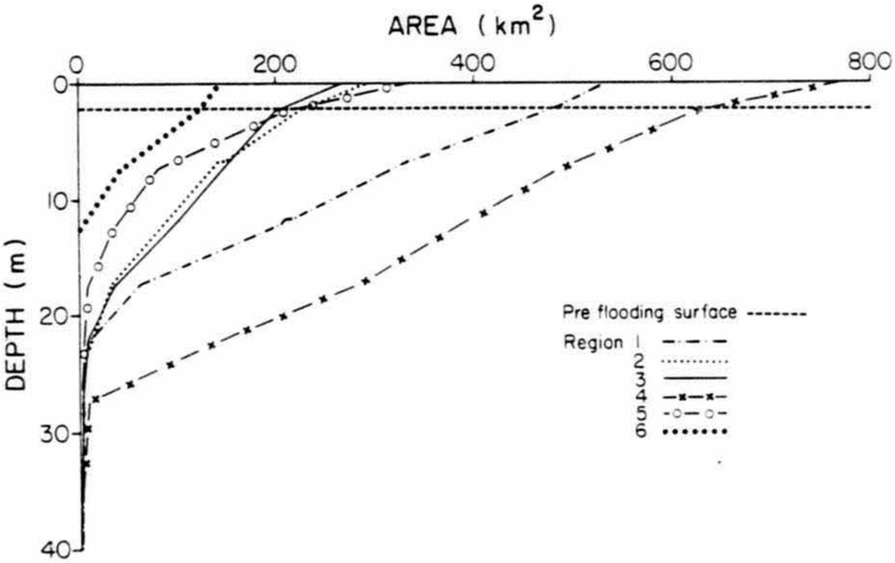

FIG. 6. Hypsometric relations of the major basins of Southern Indian Lake.

TABLE 2. Shoreline lengths and types around Southern Indian Lake regions before and after impoundment (map scale 1:63 360). Postimpoundment estimates are for the immediate postflooding condition before significant erosion has occurred (LWCNR 1975).

	Whole lake	Region						
		0	1	2	3–4	5	6	7
Preimpoundment								
Shoreline length (km)	3665	155	1067	615	906	314	280	328
Shoreline type (%)								
R1 Steep outcropping bedrock	14	11	8	11	8	0	5	60
R2 Outcropping bedrock	62	72	80	64	75	36	47	28
R3 Bedrock with overburden beach	12	8	11	15	10	17	37	6
O1 Proglacial sands and gravels	4	2	0	6	3	14	0	0
O2 Alluvial silty clay	7	6	1	3	4	28	3	5
O3 Lacustrine silty clay	1	1	0	1	0	5	8	1
Postimpoundment								
Shoreline length (km)	3788	194	1030	512	1037	551	256	208
Shoreline type (%)								
R Bedrock	14	9	8	13	7	0	5	95
O Overburden	86	91	92	87	93	100	95	5

TABLE 3. Volumes, depths, and exchange times of Southern Indian Lake regions before and after impoundment and river diversion. Preimpoundment mean elevation of the lake is 255 m (MSL), and postimpoundment mean elevation is 258 m (MSL). Exchange times for regions are based on the net interbasin transfers necessary to balance the water budget of the lake and do not allow for wind-driven circulation (McCullough 1981).

	Whole lake	Region							
		0	1	2	3	4	5	6	7
Volume (km^3)									
Pre	16.84	0.38	3.78	1.70	1.74	7.59	1.04	0.42	0.19
Post	23.38	0.63	5.27	2.45	2.42	9.64	1.81	0.81	0.35
%	39	66	39	44	39	27	74	93	84
Mean depth (m)									
Pre	8.5	4.9	8.0	7.6	8.7	12.1	4.9	3.5	4.1
Post	9.8	6.8	10.1	8.8	9.6	13.0	5.9	5.8	5.8
%	15	39	26	16	10	7	20	66	41
Exchange time (yr)									
Pre	0.51	0.012	0.12	0.053	0.055	0.23	1.5	4.2	0.39
Post	0.72	0.021	0.17	0.078	0.40	1.4	2.8	0.031	0.73
%	41	67	42	47	730	610	190	−99.3	87

forcing all of the lake outflow to pass over Missi Falls. Reducing the outlet to one channel caused the lake level to rise above the recorded high level to 256.52 m (MSL) during the summer high-flow period in 1974. In 1975, the outlet capacity was further reduced by coffer dams at Missi Falls, causing the lake levels to reach 256.76 m (MSL), again exceeding the recorded high levels. The final impoundment of the lake and the beginning of the diversion southward began in June 1976. By October, the lake had reached the licensed operating range of 257.25–258.17 m (MSL), but diversion flows were limited to less than 400 $m^3 \cdot s^{-1}$ until the following year (Fig. 5). The levels have been maintained near 258 m (MSL) since 1976, creating a 3.07-m impoundment above the long-term mean lake level.

Effects of Flooding on Areas, Volumes, and Shoreline Lengths

The surface areas and volumes of the Southern Indian Lake regions before and after impoundment were compiled by McCullough (1981) from various maps and hydrometric surveys. The total lake area increased from 1977 to 2391 km^2 and the total volume increased from 16.84×10^9 to 23.38×10^9 m^3. The regional areas and volumes are summarized in Table 1. Hypsometric distributions of area with depth for regions 1–6 are shown in Fig. 6. Although all regions had the same increase in lake level, the relative increases in flooded area and volume vary from region to region because of the shapes of the individual basins. The greatest relative increase in area occurred in region 5; the least occurred in region 6.

Shore and backshore zones surrounding the lake were mapped prior to impoundment in accordance with a classification system developed for lakes and river channels in the Churchill and Nelson valleys (Newbury et al. 1973; LWCNR 1975). The classification and shoreline lengths for regions 0–7 and the whole lake are summarized in Table 2. Estimates of the postimpoundment distribution of bedrock and overburden at the shoreline were derived from preimpoundment mapping. Immediately after flooding, bedrock was exposed at the water's edge only where high bedrock outcrops occurred in the preimpoundment state.

The net length of shoreline increased from 3665 to 3788 km (measured at 1:63 360 map scale) following the impoundment. In regions 0, 3, 4, and 5 the length increased, but in regions 1, 2, 6, and 7 the length decreased, as the highly crenulated shoreline and islands were submerged. Raising the lake level above the natural wave-washed zone caused a marked change in the type of materials at the eroding face of the shoreline. Prior to flooding, 88% of the shoreline was bedrock controlled. Immediately following impoundment, bedrock outcrops occurred on only 14% of the shoreline (Table 2).

Effects of Flooding and Diversion on Water Exchange Times

Water budgets for the Southern Indian Lake basins for pre- and post-impoundment and diversion conditions showed that

TABLE 4. Lake elevations, areas, mean depths, and water renewal times of major lakes in the lower Churchill River valley before and after Churchill River diversion. Mean pre- and post-diversion flows of Churchill River are estimated at 1011 and 25 $m^3 \cdot s^{-1}$, respectively (LWCNR 1975).

	Partridge Breast Lake	Northern Indian Lake	Fidler Lake
Elevation (m MSL)			
Pre	245.0	236.0	232.6
Post	242.0	232.8	229.0
Annual level fluctuation (m)			
Pre	1.0	1.3	1.0
Post	2.8	2.7	2.2
Area (km^2)			
Pre	23.8	144.7	38.8
Post	14.5	87.8	9.3
Volume (km^3)			
Pre	0.151	0.818	0.115
Post	0.107	0.415	0.034
Mean depth (m)			
Pre	6.3	5.7	3.0
Post	7.4	4.7	3.7
Water renewal time (d)			
Pre	1.7	9.0	1.3
Post	4.8	16.4	1.3

TABLE 5. Lake elevations, areas, volumes, mean depths, and water renewal times under mean diversion flow conditions of major lakes in the Rat River valley amalgamated by impoundment at Notigi Lake (Bodaly et al. 1984a; LWCNR 1975; Underwood-McLellan and Associates, Winnipeg, Man., unpubl. data). NA, data not available.

	Issett Lake	Karsakawigimak Lake	Pemichigamau Lake	Central Mynarski Lake	West Mynarski Lake	Rat Lake	Notigi Lake	Notigi Reservoir
Elevation (m MSL)								
Preimpoundment	250.6	248.1	247.8	251.1	249.0	247.8	242.0	
Postimpoundment	258.0	258.0	258.0	258.0	258.0	257.9	257.2	
Change	7.4	9.9	10.2	6.9	9.0	10.1	15.2	
				Predevelopment				*Postdiversion*
Area (km^2)	3.7	18.8	19.3	11.5	6.2	78.4	15.1	584
Volume (km^3)	NA	0.038	0.042	0.291	0.031	0.010	0.080	4.1
Mean depth (m)	NA	2.0	2.2	3.7	2.7	1.7	5.3	7.8
Water renewal time (d)	NA	37	39	136	213	60	30	62

TABLE 6. Changes in the power distribution of the Churchill and Nelson rivers following hydroelectric development, based on mean annual flows and average open-water conditions (LWCNR 1975). Post-development distributions are subdivided between energy dissipated at hydraulic structures and energy dissipated throughout channels and shorelines in the affected systems.

	Prediversion power (MW)	Postdiversion power (MW)		Change in power
		Concentrated at dams	Spread throughout system	
Rat River	2	109[a]	12	6×
Burntwood River	85	0	697	8×
Lower Churchill River	2702	8	686[b]	
Lower Nelson River	3969	1730[a]	3552[b]	
Southern Indian Lake	0.2		21.5[c]	108×
Total hydraulic power	6758.2	6815.5[d]		

[a]Wave power on the Notigi and Nelson River reservoirs was not included.

[b]Power remaining in stable lower Churchill or Nelson River channels.

[c]Disruptive power on Southern Indian Lake is estimated as the product of average annual wave power on the lake multiplied by the proportion of shorelines in erodible backshore materials.

[d]Net increase in power of 57.3 MW is due to the 3-m impoundment on Southern Indian Lake and the net increase in water levels in the system.

the water exchange time for the whole lake increased from 0.51 to 0.72 yr under average flow conditions because of the increased volume of the lake (Table 3) (McCullough 1981). In regions 0, 1, 2, 5, and 7 the exchange times were affected by the impoundment only. In regions 3 and 4 the exchange times were increased 7.3 and 6.1 times, respectively, as the major portion of the Churchill River waters no longer flowed through these basins. The major portion of the flow of the Churchill River now passes through region 6, decreasing the exchange time from 4.2 to 0.03 yr.

Downstream Effects of the Churchill River Diversion

Before entering Southern Indian Lake, the Churchill River receives water from 242 000 km^2 of the western interior of Canada. Local tributary streams directly tributary to the lake drain an additional 14 000 km^2. In combination, the drainage areas produce a long-term potential outflow from the lake of 1010.7 $m^3 \cdot s^{-1}$ (LWCNR 1975). With the diversion operating, the mean outflow at Missi Falls was estimated to be 251 $m^3 \cdot s^{-1}$ (LWCNR 1975), a reduction to 25% of the natural condition. Approximately 75% of the flow, or 760 $m^3 \cdot s^{-1}$, is diverted into the small Rat River and Burntwood River valleys enroute to the lower Nelson River (Fig. 3).

The effects of the reduced flows on lakes of the lower Churchill River valley are summarized in Table 4. Under the minimum postdiversion discharge conditions, the lake levels are lowered because of the reduced depths of flow at the lake outlets. An increased range of lake elevations is anticipated as periodic flood flows are released at Missi Falls.

Diversion flows to the Nelson River are controlled by a dam and gated regulation structure installed at the outlet of Notigi Lake in the Rat River valley. The elevation of Notigi Lake was raised 15.2 m by storing Rat River discharges over a 3-yr period prior to the diversion. By June 1976, the lake was impounded to the level of Southern Indian Lake, creating the 584-km^2 Notigi reservoir in the upper Rat River valley contiguous with Southern Indian Lake. The predevelopment morphometric characteristics of the upper Rat River valley lakes incorporated in the Notigi reservoir are summarized in Table 5.

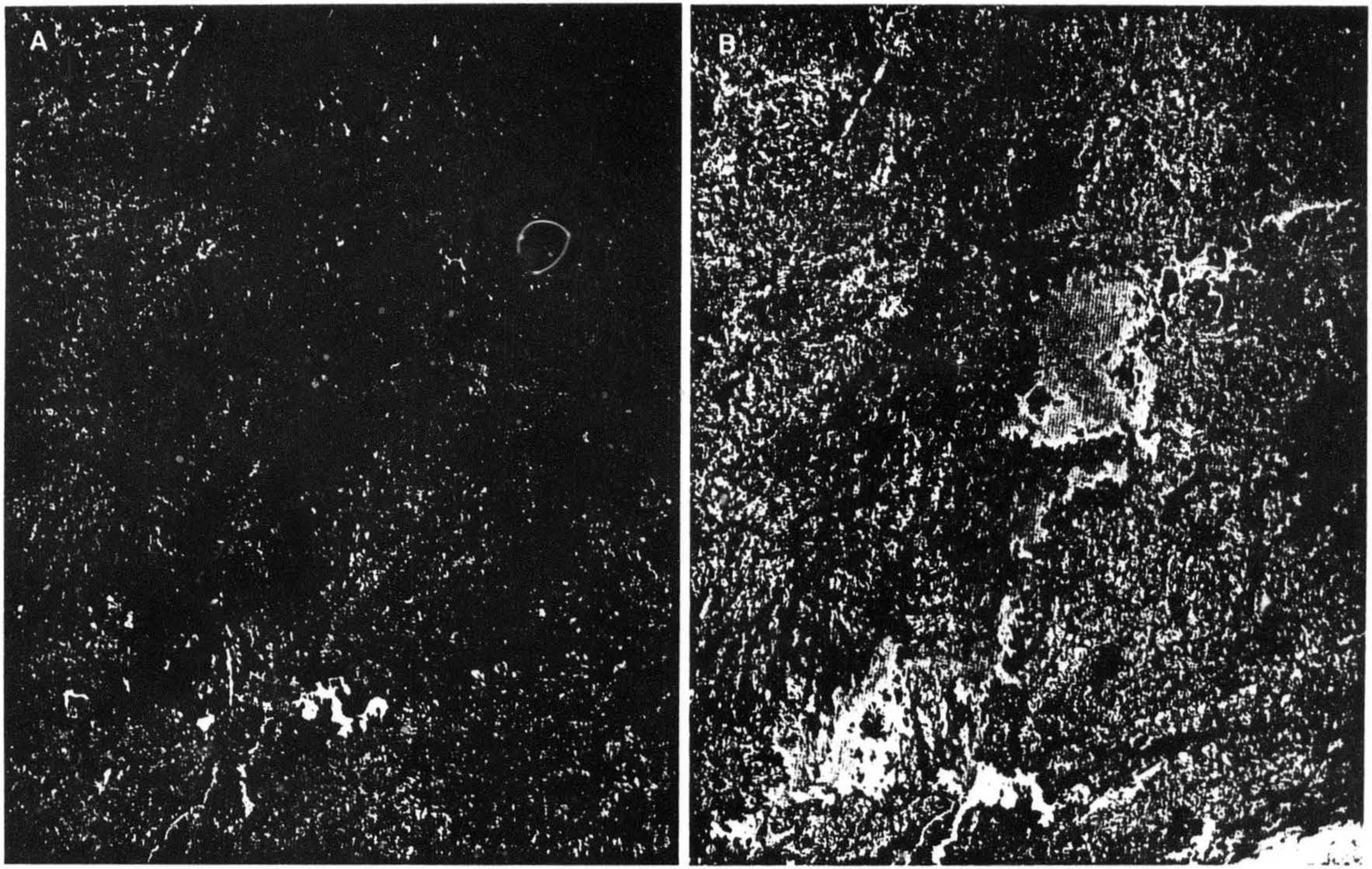

FIG. 7. Landsat satellite images of Southern Indian Lake taken on (A) 29 July 1973 before impoundment and (B) 24 June 1978 after impoundment. Lighter blue tones in Fig. 7B indicate higher reflectivity from the water surface because of the increased turbidity of the lake water. The turbidity increased in regions 0, 1, 2, 3, 4, and 6 because of the erosion of flooded fine-grained shoreline materials. The turbidity of waters in regions 5 and 7 was not affected by the impoundment because the flooded shoreline materials were coarse-grained or bedrock. A decrease in turbidity after impoundment occurred in shallow bays (e.g. Wupaw Bay and the east end of region 6) because the bottom sediments were no longer resuspended by wave action. The change in Southern Indian Lake from a sediment trap to a sediment source after impoundment is apparent by comparing the turbidity of the inflowing and outflowing waters of the Churchill River.

Effects of Diversion on the Hydraulic Regime of the Churchill and Nelson Rivers

Impoundment and diversion of the Churchill River was a sudden and drastic relocation of hydraulic forces in the established landscape. The magnitude of the relocation is summarized in Table 6 by comparing the hydraulic power (megawatts) expended on the landscape before and after the impoundment and diversion under average river flow and wave conditions. The degree of change or instability brought about by impoundment and diversion depends on the landforms encountered by the redirected forces of the new configuration. For example, the repositioning of wave energy by flooding a stable bedrock cliff caused no instability because the landform encountered before and after the project was unaffected. In contrast, flooding into backshore zones composed of frozen unconsolidated materials created shorelines that can remain unstable for decades. Most of the 21.5 MW of wave energy of Southern Indian Lake is being expended in eroding new shorelines in the flooded periphery of the lake during the open-water season. The eroded materials have increased the turbidity of the lake waters dramatically, as shown in the satellite images of Southern Indian Lake taken before and after impoundment in Fig. 7. Similarly, the 8 times greater power of the diverted flows has begun to form a new "lower Churchill River" in the Rat and Burntwood valleys wherever erodible materials form the riverbanks.

The natural forces redirected by the Churchill River diversion scheme are generally too large and too dispersed to be mitigated by further construction. As a result, the instabilities created in the environment are essentially beyond control. Subsequent papers in this issue document the effects of this disruptive power and instability on the physical, chemical, and biological components of the constantly changing aquatic ecosystems of Southern Indian Lake.

Acknowledgments

Studies at Southern Indian Lake are now entering their eleventh year. The longevity of research on the lake, and the contents of this issue, attest to the farsighted judgment of those individuals involved with the original formulation of the problem and the study design. Many originators have moved on to other tasks. In particular, Gordon Koshinsky and Andrew Hamilton were responsible for the formulation and execution of the original design of the LWCNR (1975) study, aspects of which are still being pursued today. Without their foresight, and the continued support of Paul Campbell, manager of the fish habitat research group, the scientific opportunity presented by the Churchill–Nelson development would have been lost. Studies funded by the Department of Fisheries and Oceans began under the directorship of G. H. Lawler and have continued under his overview as Director-General of the Western Region. Without his continuing support and the support of the administrative services of the Freshwater Institute, the long-term Southern Indian Lake studies reported in this volume would not have been possible. Finally, all the contributors to this series of papers must thank Dr Winston Billingsley, Special Editor, for greatly improving each manuscript as well as ensuring the overall coherence of the series.

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Effect of Impoundment and Diversion on the Sediment Budget and Nearshore Sedimentation of Southern Indian Lake[1]

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Hecky, R. E., and G. K. McCullough. 1984. Effect of impoundment and diversion on the sediment budget and nearshore sedimentation of Southern Indian Lake. Can. J. Fish. Aquat. Sci. 41: 567–578.

Shoreline erosion added an annual average of 4×10^6 t of mineral sediment per year to Southern Indian Lake (postimpoundment area, 2391 km^2) during the first 3 yr of impoundment. This erosion increased sedimentary input to the lake by a factor of 20. The lake retained 90% of this eroded material within its basin, and 80–90% of the retained material was deposited nearshore. Despite the production of extremely fine constituent particle sizes, eroding shorelines generated predominantly large clay aggregates, initially transported offshore as bed load. During bed load transport, abrasion of clay aggregates produced fine particles that became suspended. Over 80% of the suspended load is lost to outflows from the lake because the suspended load is primarily fine silt and clay-sized particles, most of which do not settle even under winter ice cover. The extensive nearshore clay aggregate deposits are temporary, and net deposition in these areas will change to net erosion when input of sediment from eroding shorelines ceases. The effects of shoreline erosion on the lake's sediment regime will persist for decades.

L'érosion de la ligne de rivage, pendant les trois premières années de retenue des eaux, ajouta en moyenne $4 \times 10^6\ t \cdot a^{-1}$ de sédiments minéraux dans le lac Sud des Indiens (superficie après retenue de 2391 km^2). Les apports de sédiment dans le lac augmentèrent, du fait de cette érosion, d'un facteur de 20. Le lac a retenu dans son bassin 90 % de ce matériel d'érosion, et 80–90 % du matériel retenu a été déposé près du rivage. En dépit de la taille extrêmement fine des particules produites, l'érosion de la ligne de rivage donna naissance en grande partie à de gros agrégats d'argile, initialement transportés vers le large comme charge de fond. Au cours du transport de cette charge, il se produisit, par abrasion des agrégats d'argile, de fines particules qui devinrent en suspension. Plus de 80 % de cette charge en suspension disparaît dans les émissaires du lac, car les particules sont surtout de la vase fine et de l'argile, la plupart demeurant en suspension même sous la couverture de glace en hiver. Les abondants dépôts d'agrégats d'argile à proximité du rivage sont temporaires et, quand cesseront les apports de sédiment des rives soumises à l'érosion, la déposition nette dans ces zones se transformera en érosion nette. Les effets de l'érosion de la ligne de rivage sur le régime sédimentaire du lac persisteront pendant plusieurs décennies.

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In the summer of 1976, Southern Indian Lake (SIL) was raised 3 m above its natural mean level initiating extensive shoreline erosion (Newbury and McCullough 1984). We have examined the sediment budgets of the lake and its constituent basins, before and after impoundment, to determine how changes in basin configuration, water flows, shoreline typology, and sediment-generating erosional processes affected concentrations of suspended solids and depositional patterns.

SIL is a large, shallow, multibasin lake on the Churchill River in northern Manitoba, which was impounded as part of the Churchill–Nelson rivers hydroelectric development (Newbury et al. 1984). A dam was built at Missi Falls, the natural outlet of SIL (Fig. 1), to raise the lake and effect gravity-flow diversion of water from SIL through an excavated channel at South Bay (region 6, Fig. 1). Diversion into the Nelson River basin began in June 1976 but did not exceed 400 $m^3 \cdot s^{-1}$ until August 1977. Since September 1977 the diversion flow has averaged 75% of the natural outflow from SIL (Newbury et al. 1984). Impoundment increased the surface area of the lake by 21% to 2391 km^2. With flooding, SIL underwent a major change in the nature of the land–water contact. Prior to flooding, 80% of the lake's perimeter was stable, bedrock-controlled shoreline, but after flooding, 86% was unconsolidated overburden that was generally permafrost affected (Newbury et al. 1984).

Before the flooding of SIL, the Churchill River was the main source of sedimentary material entering the lake, transporting $2 \times 10^5\ t \cdot yr^{-1}$ (termed "external loading"). After flooding, an additional 4×10^6 t of mineral sediment (Newbury and McCullough 1984) was added annually to the lake from surrounding shorelines (termed "internal loading"). Internal loading of material from flooded areas is a prominent feature of new reservoirs, and the distribution of newly added material, within and downstream from a new reservoir, will determine the nature, degree, and distribution of dependent ecological effects.

Large changes in the internal loading of sedimentary material can be demonstrated both by measurements of shoreline erosion (Newbury and McCullough 1984) and by the use of pre- and

[1]This paper is one of a series on the effects of the Southern Indian Lake impoundment and Churchill River diversion.

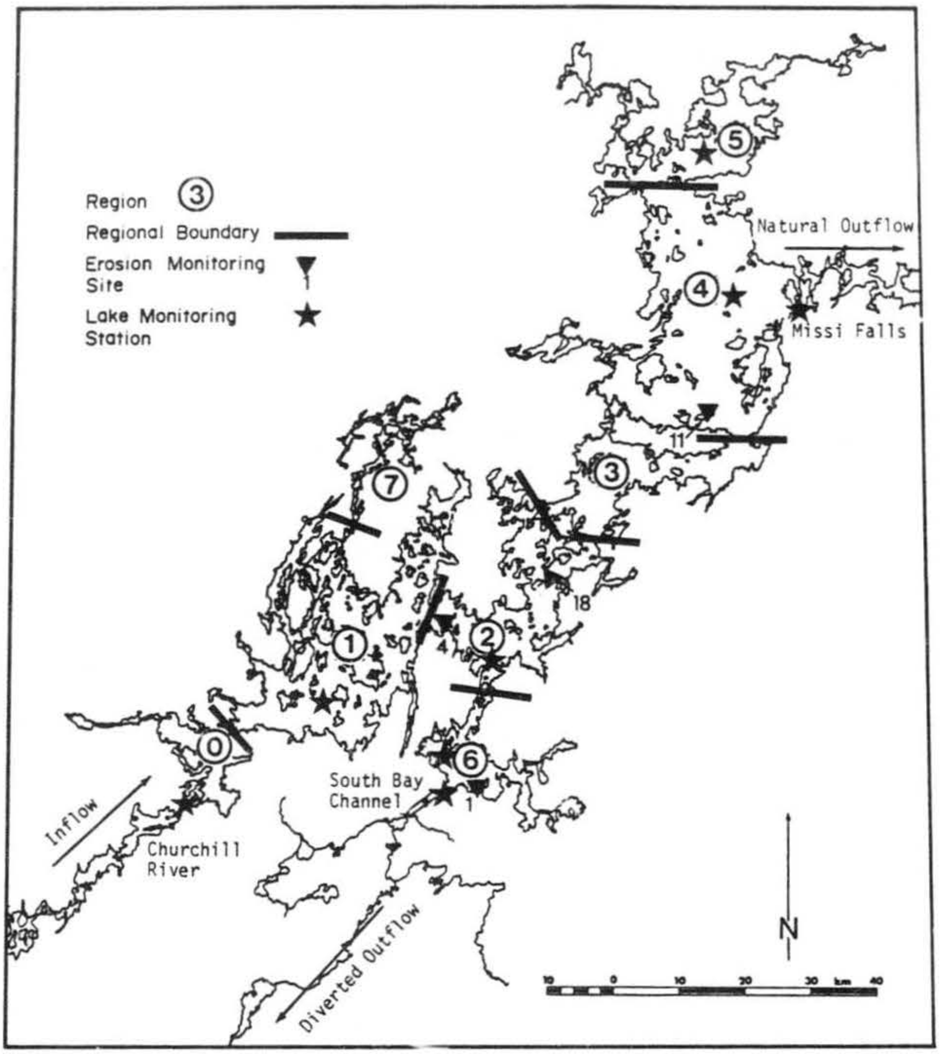

FIG. 1. Location of erosion-monitoring sites referred to in text and lake stations contributing to construction of the sediment budgets for regions of SIL.

post-impoundment and diversion sediment mass balance budgets, which use the continuity equation

(1) $I + E = O + D + M$

where I is inflowing sedimentary mass from tributaries (external loading), E is mass input by erosion (internal loading), O is outflowing mass in effluents, D is mass deposited on the lake bottom, and M is change in mass stored in the water of the lake.

SIL poses some complexities for analysis of its sedimentary regime. Previous studies of sedimentation in reservoirs have focussed largely on retention of externally loaded sediments, i.e. E in equation 1 is assumed to be zero or negligible (Rice and Simons 1982; Cyberski 1973). Retention of externally loaded sediment can be empirically predicted from data on water retention period, flow velocity, and sediment particle-size distribution (Churchill 1948). In SIL, internal loading of sediments eroded from flooded shorelines overwhelmed external loading after impoundment. Other reservoir studies have emphasized the formation of new, stable banks and shoals and their effects on reservoir morphometry (e.g. Everdingen 1968); however, stable shoal formation is the exception in SIL. Therefore, the major objectives of this study have been to determine not only the gross retention of eroded sediment but also to provide a first approximation as to where the retained material is sedimented. To do this, sediment budgets have been constructed for SIL and its individual regions from 1975 before full impoundment through 1978 after full diversion. Investigations of nearshore sedimentation and some physical characteristics of suspended sediments were conducted over the period 1977–82 to confirm independently some of the results of the sediment budget analyses.

Methods

Suspended Solids Measurements

Filterable suspended solids (FSS) are those that are retained by a Whatman GFC filter using the method of Stainton et al. (1977). The nominal pore size of these filters is 1 μm. Total suspended solids (TSS) were operationally defined as those recovered by centrifugation using a Sorvall RC2-B centrifuge with a GS 3 rotor (angle head accepting 500-mL polyethylene

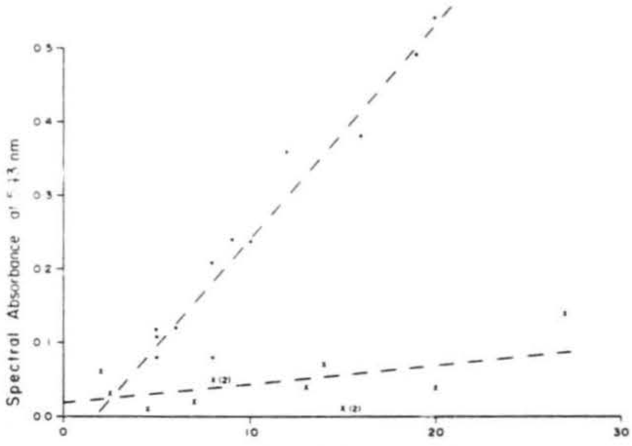

FIG. 2. Effect on absorbance of filterable (high slope line; $Ab_s = 0.029W - 0.048$, $r^2 = 0.94$) and nonfilterable (low slope line; $Ab_s = 0.0025W + 0.02$, $r^2 = 0.27$) suspended material, where W is the dry weight of the sediment. Nominal diameter for filterable solids is 1 μm.

bottles) run at 8000 rpm for 2 h at temperatures of 20–28°C. This centrifugation should theoretically settle all particles greater than 0.06 μm nominal diameter. Duplicate water samples, 1–1.4 L in total volume, were centrifuged in successive 350- to 400-mL portions, and the collected pellets were then oven-dried at 105°C and weighed. The difference between TSS and FSS, determined on paired samples, was considered to be nonfilterable suspended solids.

FSS were found to be highly correlated with several light measurements. Absorbance measurements at 543 nm (in the middle of the visible spectrum) were made on water samples in a 10-cm cell on a Bausch & Lomb Spectronic 70 spectrophotometer before and after filtration and centrifugation. Absorbance at 543 nm was strongly correlated with FSS but was weakly correlated with nonfilterable suspended sediment concentration (Fig. 2). Most importantly, absorbance was 10 times as sensitive to an increase in FSS concentration as it was to an equivalent increase in nonfilterable suspended material. Kullenberg (1974) also found that particles >1 μm in diameter are much more effective in scattering light than smaller particles. The scattering coefficient (S_s, based on vertical extinction and backscattering of incident surface light), the horizontal beam attenuation coefficient (α), and the logarithm of the inverse of Secchi disk depth were all strongly correlated with FSS (Table 1). These measurements were routinely made in SIL by Hecky (1984) and provided a means of rapidly estimating FSS in the field. They also offered the advantages of integrating over a significant portion of the upper water column (1–5 m) in the case of the scattering coefficient and Secchi disk measurement or of yielding detailed profiles of the whole water column in the case of the horizontal beam transmissometer. During the open-water season, measurements with the transmissometer indicated that the water column was nearly always uniform in α (Hecky et al. 1979). Consequently, S_s was used to estimate FSS because of the greater number of measurements of S_s during open-water seasons. Horizontal beam attenuation was used during ice cover to estimate FSS concentrations. Hecky et al. (1979) reported the estimated FSS concentrations for all SIL stations from 1974 through 1978. Observation of concentrations was at least monthly during the ice-free period, June through

TABLE 1. Linear regression relationships between filterable suspended sediments (FSS) and scattering coefficient (S_s), horizontal beam attenuation coefficient (α), and Secchi disk depth (SD). The number of samples (n), correlation coefficient (r), and range of FSS values included are also given.

Relation	n	r	Range (mg·L⁻¹)
$FSS = 7.7(S_s) + 0.5$	18	0.93	1–30
$FSS = 0.9(\alpha) - 0.1$	16	0.94	1–20
$FSS = 9.8(\ln 1/SD) + 7.6$	27	0.89	1–30

October, but less frequent, 2–3 times, during the ice-covered period.

Sediment Budgets

Sediment budgets from SIL were analyzed in two stages. Because FSS could be related with greater confidence than TSS to available light measurements, FSS budgets were constructed in the initial stage. Budgets for FSS were constructed using only inflow, outflow, and lake concentration data. These budgets have a net flux term (see below) that sums erosional input and depositional output (E and D, equation 1). In the second stage of budget analysis, total sediment budgets were derived from the FSS budgets, and the net flux term was separated into its erosional and depositional components.

Budgets for Filterable Solids

FSS budgets were calculated for the individual regions of the lake (Fig. 1) except that regions 0 and 1 and regions 3 and 4 were combined and treated as single regions. Budgets were calculated for the period of January 1975 through December 1978. The continuity equation

$$(2) \quad I_f = O_f + S_f + M_f,$$

where I_f = mass of FSS entering the lake or region from all inflows, O_f = mass of FSS leaving the lake or region in all outflows, S_f = mass of filterable material entering or leaving suspension within the lake or region, and M_f = change in total mass of FSS in the lake or region in a given time period (i.e. change in storage), was solved for each region on a monthly basis for S_f. This term, S_f, represents the balance between measured gains and losses of FSS. Water fluxes and storage were from the water budget of McCullough (1981). Linear interpolation was used between FSS observations. Month-end FSS concentrations were multiplied by month-end regional volumes to calculate M_f.

Because Churchill River FSS concentrations were found to be discharge related ($r = 0.69$, $n = 20$, flow range 870–1630 $m^3 \cdot s^{-1}$), a linear regression equation was used to estimate FSS concentrations in the inflowing river. Mean FSS concentrations for local inflowing streams and rivers were assumed to be the mean concentration for the Churchill River, 3.2 $g \cdot m^{-3}$. The Churchill River is 20 times larger than the next largest inflowing river and accounts for 90% of total inflows into SIL. The assumed mean concentration in local inflows is likely somewhat high for these small inflows; however, even using this estimate, sedimentary mass contributed by local inflows is only 5% of the Churchill River contribution. FSS concentrations in outflowing water from the basins were taken to be those measured at the central basin station in regions 4 and 6. Mean monthly FSS concentrations were multiplied by outflow volumes to calculate O_f.

TABLE 2. Flushing rates (mo^{-1}) for various basins of SIL during ice-covered (IC) and ice-free (IF) seasons (from McCullough 1981).

Season		Region				
		0–1	2	3–4	6	5
1975–76	(IC)	0.56	1.3	0.27	0.02	0.07
1976	(IF)	0.58	1.4	0.15	1.3	0.01
1976–77	(IC)	0.51	1.2	0.20	0.84	0.03
1977	(IF)	0.70	1.1	0.23	1.8	0.05
1977–78	(IC)	0.44	1.1	0.06	2.4	0.02
1978	(IF)	0.54	1.3	0.13	2.5	0.14

Total Sediment Budgets

The calculation of a total sediment budget (equation 1) required correction of several terms of the FSS budget to account for nonfilterable suspended solids. Filtration of suspended solids samples throughout the lake and covering a large range of concentrations (from 9–47 $mg \cdot L^{-1}$ TSS) removed 56% of the TSS on average (range 38–89%, SD = 15%, n = 11). This yields a mean weight ratio of 1.8 for TSS:FSS. Churchill River TSS was 53% FSS.

The mean factor of 1.8 for TSS:FSS was applied to the quantities I_f and O_f (equation 2) to estimate I and O (equation 1). The quantity of total material entering or leaving suspension was calculated as $S = 1.8S_f$ for negative values of S_f (material entering suspension from within the basin; equation 2) and $S = S_f$ for positive values (material leaving suspension within the basin). Negative values for S_f were corrected for the submicrometre fraction because filterable material entering suspension would be accompanied by submicrometre particles. Positive values of S_f were not corrected for a submicrometre fraction, as the submicrometre fraction is not likely to sediment within SIL even under ice. Six months are required to clear a 10-m still-water column of 1-μm particles (Tanner and Jackson 1947). Only regions 3–4 and 5 were flushed slowly enough (Table 2) for submicrometre particles to settle in substantial amounts; and even there, observed sediment profiles, under ice cover when turbulence is lowest, were unchanged from mid-January through the end of March (see Results below). Values of E are from Newbury and McCullough (1984), and the depositional flux (D) is calculated as $D = S + E$.

Shoreline Erosion and Nearshore Deposits

Annual surveys to determine rates of shoreline erosion were done by Newbury and McCullough (1984). Twenty shoreline sites were chosen to represent a range of erodible backshore materials with high and low exposures to the lake. The developing offshore profiles at these sites were monitored by annual depth soundings with a survey rod along profiles perpendicular to the shoreline. Additional investigation of the proportion of eroded materials that was deposited near to eroding shorelines was begun in 1982 by scuba diving. Cores of the bottom sediments were taken using 73-mm-diameter plastic tubes along transects perpendicular to the shore at erosion monitoring sites 1 and 11 (Fig. 1). Both sites chosen had been preimpoundment sand beaches; hence, newly deposited fine mineral and organic sediments were easily discriminated from preimpoundment coarse deposits or rocky bottom. Mechanical analysis of sand, silt, and clay fractions of shoreline overburden samples was reported by Newbury and McCullough (1984).

Samples of recently deposited nearshore bottom sediments were also analyzed for sand, silt, and clay content by standard mechanical analysis (McKeague 1976), which included drying, pulverizing with a mortar and pestle, and dispersing with sodium hexametaphosphate. Duplicate subsamples were mechanically analyzed without preparatory pulverization and dispersion to determine the natural size distribution of the silt and rounded aggregates of clay that formed the sediments. Suspensions of wet, unpulverized bottom sediment were sedimented into a suspended pan, and the accumulated weight was recorded using a Cahn RG electrobalance and chart recorder.

Results

Shoreline Material

The inorganic shoreline materials of southern regions of the lake are predominantly permafrost-affected glacio-lacustrine deposits with greater than 80% clay and 0–5% sand. Discontinuous, thin deposits of clayey till (19–65% clay, 10–35% sand) commonly lie on the bedrock surface under the glacio-lacustrine deposits. The till is most extensive in region 4 and much less common in regions 0, 1, and 6. Region 5 shorelines are predominantly composed of coarse-grained glacio-fluvial and pro-glacial deltaic deposits. Sand, silt, and clay content of selected samples of shoreline materials were reported by Newbury and McCullough (1984).

Concentrations of Filterable Solids

The effect of impoundment on the concentrations of FSS varied from region to region (Fig. 3). Most of the impoundment of SIL was accomplished in 1976 with the raising of the lake level by 0.3, 0.8, 0.7, and 0.3 m in June, July, August, and September, respectively (Newbury et al. 1984). During 1976, the June and early July concentrations in the three southern regions 1, 2, and 6 were similar to those in 1974 and 1975. However, in 1976, by the end of July in region 6 and the end of August in regions 1 and 2, FSS concentrations were clearly greater than in the preimpoundment years. In 1977 and 1978 these three southern regions all had concentrations higher than before impoundment, but concentrations of FSS did not increase from 1977 to 1978. The exceptionally high values observed in October 1976 in regions 1 and 6 did not recur. Mid- to late-summer and fall concentrations in 1975 were somewhat higher than in 1974 because of shoreline erosion due to high water levels. However, the range of concentrations observed in these southern regions prior to impoundment was small compared with the postimpoundment changes.

Prior to impoundment, the two northern regions 4 and 5 had similar, low concentrations of FSS. After impoundment, region 5 did not change significantly, but region 4 showed the marked increase in FSS observed in southern basins in August 1976. FSS concentrations increased in each subsequent year, especially in the June–July period. The relative difference in FSS between region 4 and the more southerly basins of SIL was substantially reduced after impoundment.

Under ice cover the FSS concentrations of the rapidly flushed southern regions (0–1, 2, 6, Table 2) declined substantially from open-water values. In the more slowly flushed region 4, winter reductions were less marked. For example, the winter profile of α (the coefficient of horizontal light attenuation) in region 2 showed a substantial decline (60%) from October to January and a continuing reduction through March (Fig. 4) to

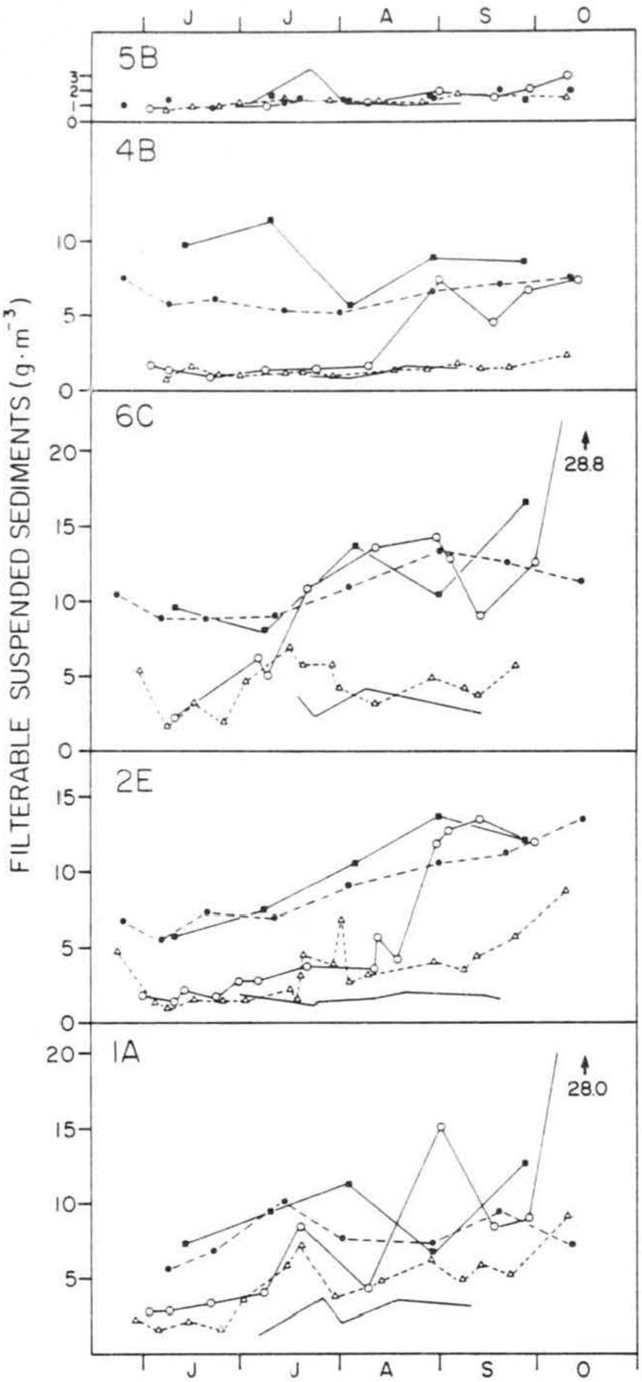

FIG. 3. Concentrations of FSS at index stations in the various regions during the open-water seasons of 1974 (—), 1975 (△), 1976 (○), 1977 (●), and 1978 (■). Plotted data are from Hecky et al. (1979).

20% of the October value. In region 4 the upper water column became somewhat clearer (30% decline) by January as sediments accumulated at greater depths. However, there was a negligible change from January to March (Fig. 4). If the vertical water column at the station in region 4 was assumed to be a closed system, all the material that was to settle from the water column had done so by mid-January. The more rapid decline in α in region 2 and the continuing decline after January were due

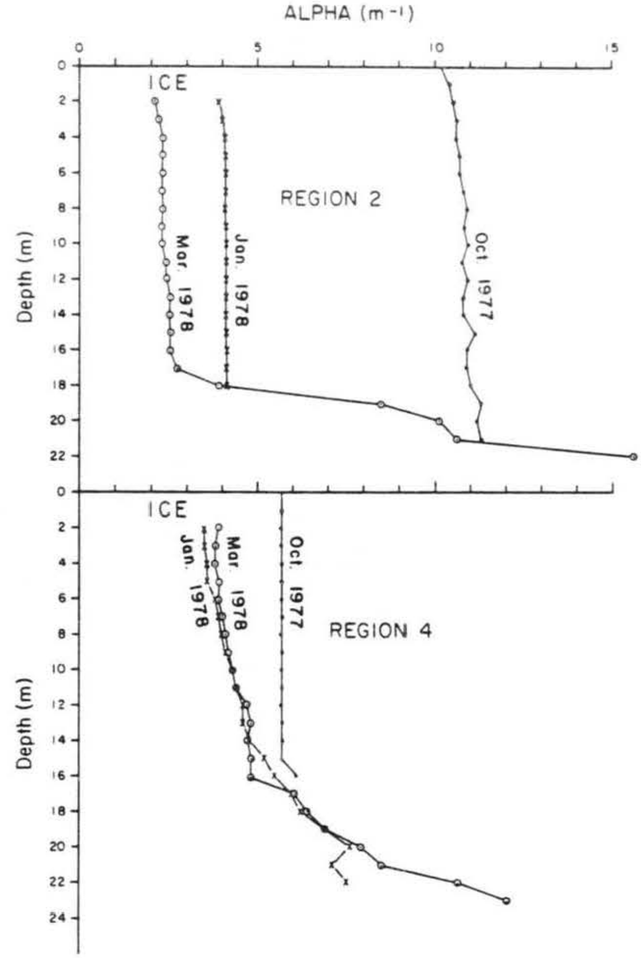

FIG. 4. Depth profiles of α in regions 2 and 4 for October 12, 1977, January 13, 1978, and March 23, 1978, from Hecky et al. (1979).

to the more rapid flushing during this winter period as compared with region 4 (Table 2). Rapid flushing allowed the southern regions to come quickly to a balance between the inputs and outputs of sediment. For example, the seasonal concentrations during 1977 and 1978 were similar within these regions. In contrast, region 4 had not achieved a new steady state, and sediment concentrations in all seasons continued to rise through 1978 (Fig. 3).

Sediment Budgets

Filterable suspended sediment budget

Impoundment and diversion changed SIL from a basin of deposition for FSS (positive S_f) to a basin of export (negative S_f) (Table 3). Export of FSS at outflows increased by a factor of 4–5 after impoundment and diversion. Substantial changes in FSS concentrations, especially in regions 1 and 6 (Fig. 3) in 1976, resulted in a large storage term for that year, but much of that initial flush of sediment was lost the following winter (1977). Subsequent changes in storage (1978) have been smaller but positive as FSS concentrations in the largest region, 3–4, have continued to increase. These changes in output and storage of FSS occurred because eroded input increased the internal loading of FSS.

TABLE 3. Calendar year filterable sediment budgets (10^6 kg) for SIL. Mass fluxes are riverine input (I_f), riverine output (O_f), lake suspended sediment storage (M_f), and the material entering (negative) or leaving (positive) suspension from within the lake (S_f).

	I_f	O_f	M_f	S_f
1975	123	60	2	61
1976	113	201	112	−200
1977	147	297	−91	−59
1978	100	278	21	−199

Total sediment budget

The negative S_f values in the filterable suspended sediment budget (Table 3) after impoundment do not represent the total amount of material actually removed from shorelines because eroded material can sediment nearshore and be effectively invisible to a budget based on FSS concentrations at centrally located stations within a region. Total sediment budgets were constructed to allow comparison of suspended sediment fluxes with the erosion inputs calculated by Newbury and McCullough (1984), as their estimates were based on total sedimentary material eroded. Preimpoundment shoreline mapping indicated that less than 1% of the shore was characterized by significant shoreline erosion (Water Resources Branch 1974). From the evidence of Landsat satellite imagery (Hecky and McCullough 1984) and from observations recorded from the first shoreline erosion survey in August 1975 (R. W. Newbury, Department of Fisheries and Oceans, pers. comm.), it is apparent that some minor shoreline erosion, especially in regions 0–1, 2, and 6 ($E > 0$ in Table 4), occurred when the lake level was temporarily raised above its historic high level by preliminary construction work in the summer of 1975. Annual erosion surveys begun in August 1975 documented greatly increased extent and rate of erosion after impoundment (Newbury and McCullough 1984). Seasonal total sediment budgets incorporating the eroded influxes (E) from shorelines are given in Table 4. E was subdivided seasonally by assuming that shoreline erosion that is wind energy dependent (Newbury and McCullough 1984) did not occur under ice cover (i.e. $E = 0.0$). Also in Table 4 are estimates of seasonal depositional fluxes for each region. Region 3–4 had the largest erosional and depositional fluxes because of its long, actively eroding shorelines and long fetches. Even when the depositional flux is expressed per unit area (Table 5), this largest region had the highest sedimentation rates after impoundment.

Total sediment export at outflows from SIL increased after impoundment (4–5 times) but not nearly in proportion to the increase in sediment input ($I + E$), as the lake retained at least 90% of the eroded input (Table 6). After impoundment, E and D were highly seasonal (Table 4), as shoreline erosion occurred by wave action during the open-water period. The relative losses of suspended solids to outflow and deposition are best compared during the ice-covered period, when generation of suspended solids from shoreline erosion ceases. The relative losses were, in part, a function of water flushing (Fig. 5). All the regions, except 3–4, follow a common trend before and after impoundment, including region 6, which changed from a very slow flushing rate to a very high rate. Above a flushing rate of $0.5 \cdot mo^{-1}$ the ratio of loss by outflow to total loss was nearly constant at 0.8. The aberrant behavior of region 3–4 may mean

TABLE 4. Fluxes in seasonal total sediment budgets (10^6 kg) for ice-free (IF) (June 1–Oct. 31) and ice-covered (IC) (1 Nov.–31 May) periods for individual basins of SIL during years of observation. Mass fluxes are riverine input (I), erosional input (E), riverine output (O), mass entering (negative) or leaving (positive) suspension within region (S), and $S + E$ = depositional flux (D). Quantitative estimates of E are from Newbury and McCullough (1984). Estimates of E are not available for the open-water period of 1975, but values for E were observed to be negligible in regions 2, 3–4, and 5.

Region and season		I	E	O	S	D
Region 0–1						
1975	(IF)	132	>0	180	−100	>0
1975–76	(IC)	80	0	108	8	8
1976	(IF)	112	650	352	−464	186
1976–77	(IC)	132	0	218	50	50
1977	(IF)	180	849	308	−148	701
1977–78	(IC)	90	0	106	4	4
1978	(IF)	84	781	322	−322	459
Region 2						
1975	(IF)	180	>0	140	11	11
1975–76	(IC)	108	0	102	14	14
1976	(IF)	352	238	256	16	254
1976–77	(IC)	220	0	208	27	27
1977	(IF)	308	311	238	19	330
1977–78	(IC)	106	0	110	17	17
1978	(IF)	324	290	352	−62	228
Region 3–4						
1975	(IF)	142	0	54	21	21
1975–76	(IC)	104	0	68	33	33
1976	(IF)	184	2072	92	−128	1944
1976–77	(IC)	210	0	282	7	7
1977	(IF)	240	2767	174	38	2805
1977–78	(IC)	54	0	46	−12	−12
1978	(IF)	132	2524	130	−26	2498
Region 6						
1975	(IF)	0.6	>0	0.8	−0.2	>0
1975–76	(IC)	0.2	0	0.8	1.1	2.2
1976	(IF)	72	190	104	−68	122
1976–77	(IC)	62	0	72	7	7
1977	(IF)	156	273	168	−12	261
1977–78	(IC)	142	0	110	9	9
1978	(IF)	204	247	238	−42	205
Region 5						
1975	(IF)	2.8	0	1.0	0.5	0.5
1975–76	(IC)	1.6	0	0.8	0.8	0.8
1976	(IF)	6.2	207	0.4	−1.0	206
1976–77	(IC)	2.2	0	0.8	3	3
1977	(IF)	2.8	275	1.4	−0.4	275
1977–78	(IC)	1.6	0	0.6	0.6	0.6
1978	(IF)	5.0	229	2.8	1.8	227

that this large region was not uniformly mixed by the end of the open-water season, as was assumed in the budget.

If O is assumed to bear the same relation to total losses ($O + D$) during the open-water season as during the ice-covered season, then offshore sedimentation (loss of material from suspension) can be estimated for the open-water season. Using the approximation $O/(O + D) = 0.8$ (or $D = 0.25(O)$ after rearranging terms) for suspended solids losses in rapidly flushed basins observed during ice cover, offshore sedimentation of suspended material (D_o) during open water can be estimated as $D_o = 0.25(O)$ in regions 0–1, 2, and 6. This offshore sedimentation of material transported in suspension was

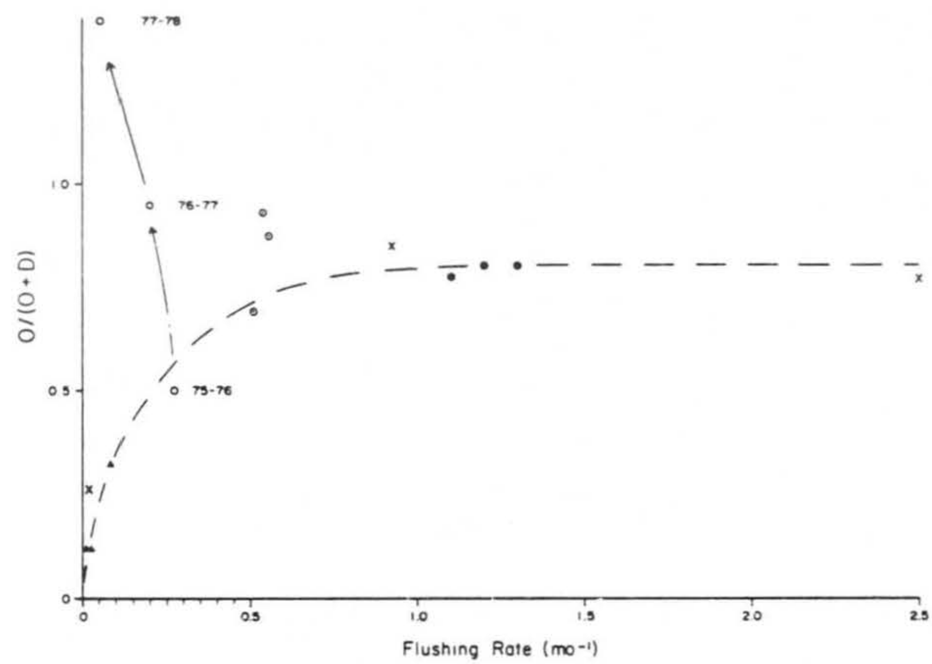

FIG. 5. Relative losses of suspended solids to outflow as a proportion of total losses compared with flushing rates during the ice-covered period for the various regions of SIL. Broken line is drawn by eye. Aberrant behaviour of region 3–4 is shown for successive years after impoundment. ⊙, region 0–1; ●, region 2; ○, region 3–4; ▲, region 5; ×, region 6.

TABLE 5. Regional sedimentation rates ($g \cdot m^{-2} \cdot mo^{-1}$) for SIL during ice-free (IF) and ice-covered (IC) periods determined by dividing the depositional flux (D) by water area of the region.

Season		Region 0–1	2	3–4	6	5
1975	(IF)	>0	10	5.1	>0	0.5
1975–76	(IC)	2.1	9	5.7	2.6	0.5
1976	(IF)	64	200	428	188	159
1976–77	(IC)	12	15	1.0	7.2	1.4
1977	(IF)	196	262	565	376	179
1977–78	(IC)	0.9	10	—[a]	9.3	0.3
1978	(IF)	150	181	503	295	148

[a]Negative value of D rate not calculated.

TABLE 6. Comparison of natural riverine sediment loading (10^6 kg) (I), erosive inputs (E), and export (O) from SIL. Percentage of eroded input retained (R) is calculated assuming the fraction of I retained in each year is the same as in 1975.

	Flux I	E	O	R
1975	246	>0	120	100
1976	226	3357	402	91
1977	294	4475	594	90
1978	200	4071	556	89

between 6 and 25% of total sedimentation, D (Table 7), during the open-water period. On average, 87% of D occurred nearshore in these rapidly flushed regions. Even higher proportions of nearshore deposition were likely occurring in region 5, where eroded mineral material was overwhelmingly sand and where new stable beaches and offshore shoals were forming. The proportion of nearshore deposition may have been lower in region 3–4, which had higher mean fetch (Table 8) and more energetic shoreline conditions. Movement of material off shorelines and concurrent abrasion of eroding clay materials may have been more rapid under these high-energy conditions.

Nearshore sedimentary deposits

Between July 1977 and September 1980, $13\ m^3 \cdot m$ shoreline^{-1} was eroded from the sand deposits at site 18 (Fig. 6). In the same period, $11\ m^3 \cdot m^{-1}$, or 85% of the eroded volume, was deposited at the edge of the nearshore shoal. The eroding bank consists of a 12-m height of fine to coarse sand and silty sand beds overlain by 1- to 2-m-thick clay and silty clay beds. Most of the fine materials were brought into suspension and carried farther offshore. The eroding sand was deposited in accordance with a model originally proposed by Bruun (1962). The deposited sand created a new offshore profile parallel to the preimpoundment profile but separated by a thickness of new deposits similar to the change in mean lake level.

At erosion monitoring site 1 in South Bay, in thick glaciolacustrine clay, 50 000 kg of mineral sediments was eroded per metre of shoreline between August 1975 and August 1982. Over the same period approximately 37 000 kg of mineral sediments, 75% of the weight eroded from the banks, was deposited within 300 m of the water's edge at depths of 2–7 m. Profiles of the bank surveyed in 1975 and 1982 are superimposed in Fig. 7.

At erosion-monitoring site 11 and region 4, measurements indicated that nearshore deposits comprising such a high proportion of the eroded material were diminished after bedrock was encountered on the eroding shoreline. Between September 1975 and July 1982, 17 000 kg of mineral sediments per metre of shoreline was excavated and a new bedrock shoreline was

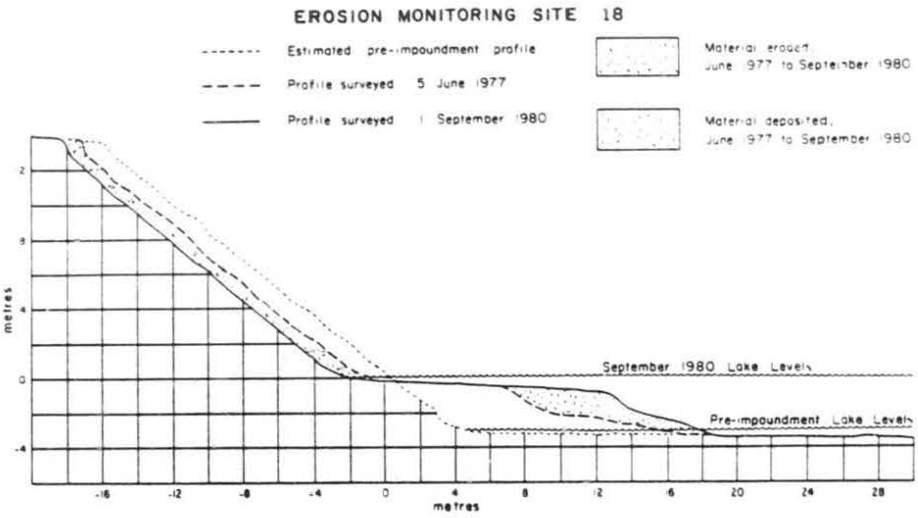

FIG. 6. Shoreline cross-section at site 18, where a sand bank is being eroded.

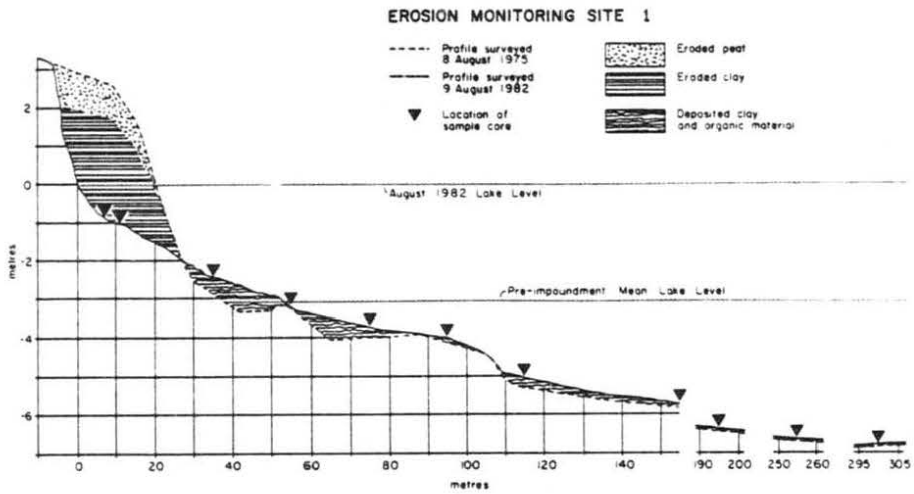

FIG. 7. Shoreline cross-section at site 1, where a fine-grained, laminated clay bank is being eroded.

attained. However, only 3300 kg, less than 20% of the eroded material, remained as deposits in the nearshore zone in 1982. Approximately 90% of these newly deposited sediments was igneous sand washed out of till deposits in the eroded banks. A few millimetres of loose clay coated cobbles and boulders in a wide zone beyond 90 m offshore in 3- to 5-m water depth. Most of the material eroded at site 11 had been carried beyond this cobble zone.

In the summer of 1983, nearshore bottom sediments were measured at an additional 28 sites. Initial inspection of this data indicates that while high deposition rates, such as at site 1, are common, there is a wide range in the fraction of eroding bank material that is deposited in the nearshore zone.

TABLE 7. Offshore deposition ($D_o = 0.25(O)$) and total deposition (D) (10^6 kg) and proportion of offshore to total sedimentation during open-water seasons for regions 0–1, 2, and 6 after impoundment.

Region	Season	D_o	D	D_o:D
0–1	1976	44	186	0.24
	1977	39	601	0.06
	1978	40	459	0.09
2	1976	32	254	0.13
	1977	30	330	0.09
	1978	44	228	0.19
6	1976	13	122	0.11
	1977	21	261	0.08
	1978	30	205	0.15

TABLE 8. Regional mean fetches (km) perpendicular to shorelines in open areas (fetches >1 km) (F_o) of the regions and for the whole region (including fetches <1 km) (F_T). Also given are maximum fetches within each region (F_{max}).

Region	F_o	F_T	F_{max}
0–1	3.7	2.4	12
2	3.4	2.3	10
3–4	8.6	6.4	35
6	3.7	3.1	11
5	5.1	2.5	20

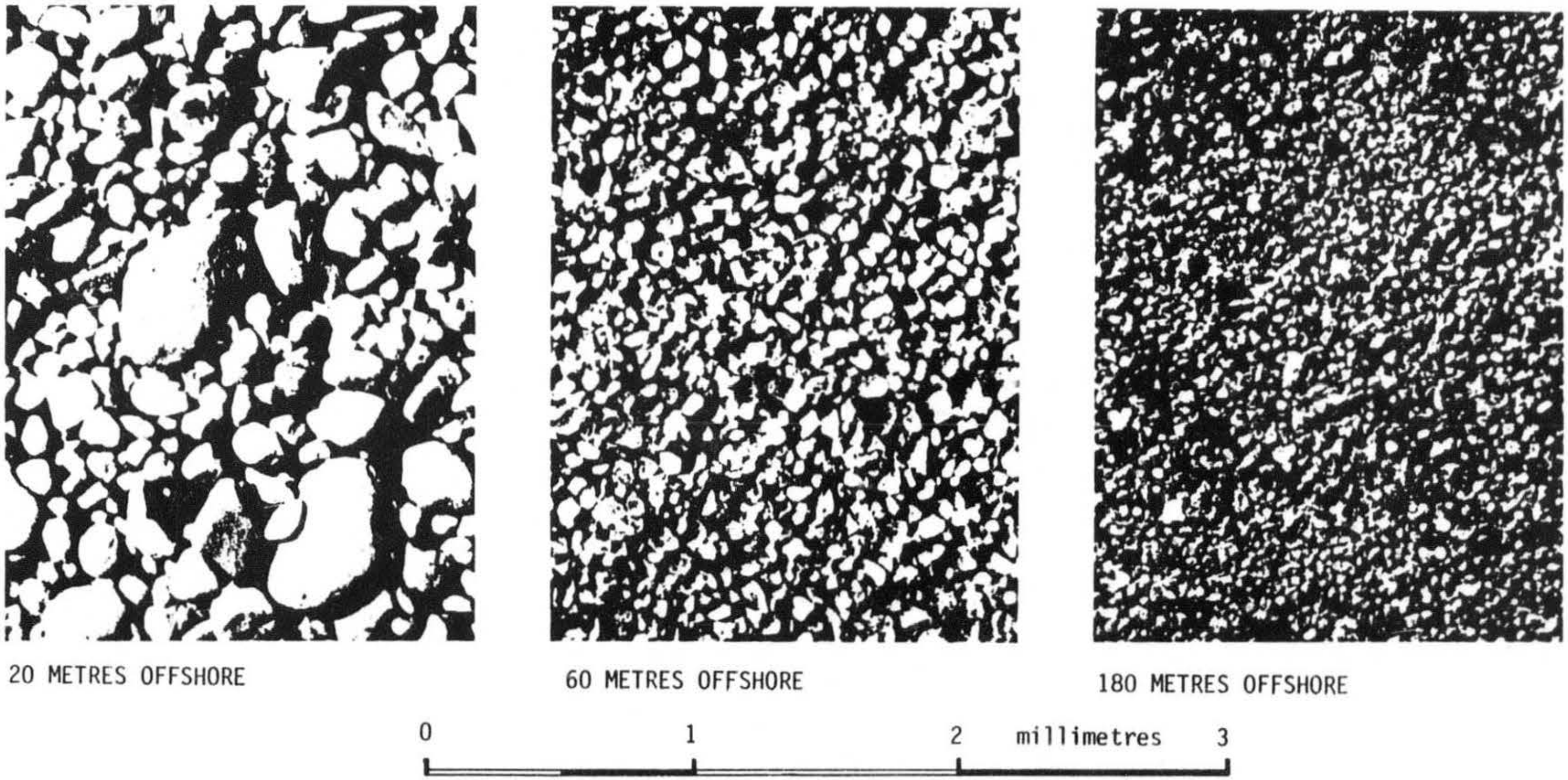

FIG. 8. Clay aggregates along a perpendicular transect from site 1. Note sorting with distance and depth. Water depths at 20, 60, and 180 m offshore are 2, 3, and 4.8 m. These clay aggregates are the coarsest in their respective samples, as finer particles have been removed by decantation.

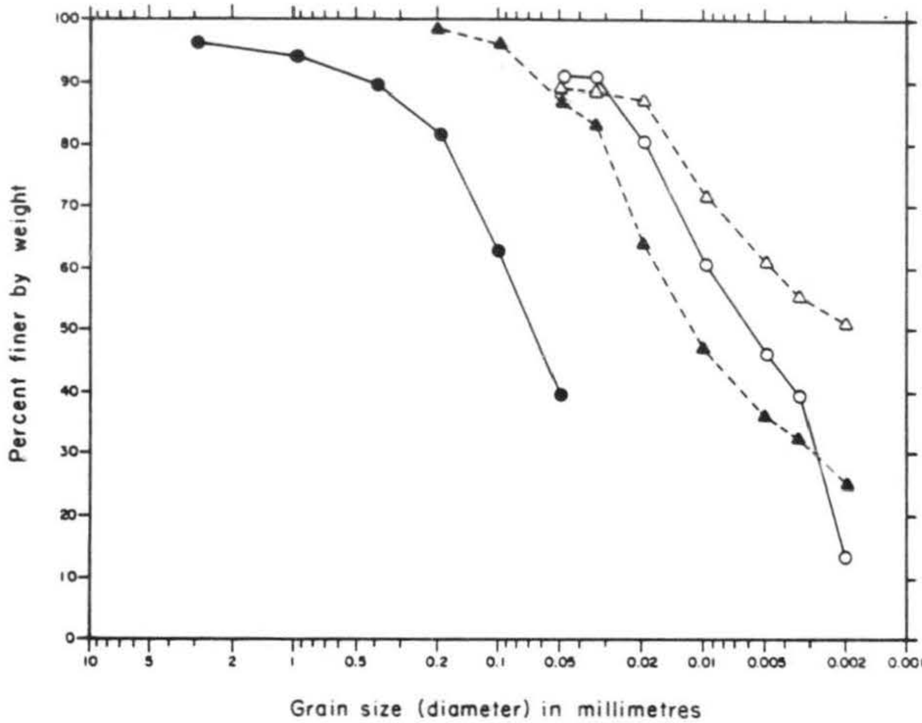

FIG. 9. Grain size distributions offshore of site 1. The size distribution of clay aggregates (closed symbols) is substantially coarser than their dispersed constituent grains (open symbols). 20 m offshore: ●, unaltered size distribution; ○, ground and dispersed with Calgon; 100 m offshore: ▲, unaltered size distribution; △, ground and dispersed with Calgon.

Visual inspection of beach (fig. 6 in Newbury and McCullough 1984) and nearshore bottom materials showed that clay fragments are rolled in the surf to form rounded, pebble-like aggregates. These continue to abrade to smaller sizes as they are carried offshore as bed load. A series of clay aggregates from samples of the bottom sediments at 20, 60, and 180 m from the water's edge is shown in Fig. 8. The aggregates are deposited in a matrix of very fine sediment that has been removed by gentle swirling and decantation from the samples illustrated. The sample at 20 m offshore has a mean diameter of the clay aggregates of 70 μm, whereas the mean particle diameter of the material after dispersion is 6 μm. Similarly, the aggregates farther offshore are smaller than nearshore, but they are still much larger than their constituent grains (Fig. 9).

Discussion

Erosion, transportation, and deposition are the primary processes affecting the concentration and distribution of sediments in a lake. The predominance of any one of these processes in space or time is largely determined by the distribution of mechanical energy within the lake and the nature of the land–water interface. This energy distribution is manifested in water circulation and turbulence (Hakanson 1977; Sly 1978). Zones of sediment erosion, transport, and accumulation occur in all lakes of substantial size, and the distribution of these zones reflects the available energy at the sediment–water interface on a lake's bottom. Hakanson (1977), although recognizing the complexities of energy distribution within the lake, found that two factors, effective fetch and water depth, determined to a large extent the distribution of sediments within the lakes he examined. Sly (1978) similarly emphasizes these factors. Effective fetch at any site sets a physical limit to the transfer of energy from wind to waves, and water depth dissipates the energy before it is transferred to the bottom.

The impoundment of SIL altered the morphometry of the lake's regions in a relatively minor way compared with many new reservoirs. Although the areal increases were on the order of 10–50% (Newbury et al. 1984) for the different regions, much of the increase in area occurred along bays and inlets. The open-water fetches were negligibly changed by impoundment. The mean depth of the lake increased by 15% (Newbury et al. 1984), so there was more water through which to dissipate the wave energy. Diversion did markedly increase the flushing rate of region 6 (Table 1), reducing the time available for sedimentation of suspended particles, but flushing rates declined or were relatively unchanged in all other regions. The changes in depth and flushing rate could have led to an increase in the areas of sediment accumulation, making SIL a better sediment trap after impoundment, as has occurred in many other reservoirs (Baxter and Glaude 1980). However, impoundment changed the nature of the land–water interface around the lake. After impoundment, wave energy could be expended in excavating the highly erodible overburden instead of being dispersed on resistant bedrock shorelines. Internal loading by erosion grossly altered the reservoir's sediment input (Table 4), but there were relatively minor changes in the processes affecting transport and deposition of sediment within the reservoirs.

Erosion

Before impoundment, 80% of the shoreline of SIL was characterized by wave-washed bedrock and sand or pebble beaches. Relatively unstable shoreline forms had persisted over the 7000-yr life span of the lake only in sheltered positions or where glacio-lacustrine deposits fronting the shore were unusually extensive, as in a few areas in region 6 (Newbury and McCullough 1984). Outlet constriction due to the construction of the Missi Falls control structure in the summers of 1974 and 1975 caused unusually high lake levels. Shoreline profiles and observations from the summer of 1975 indicate that the maximum water level overtopped established beaches in regions of low to moderate fetch but did not reach tops of beaches in regions of high fetch. Although some erosion did occur under high water levels in 1975 prior to full impoundment, it was limited to the southern regions of the lake, and even there it was minor compared with changes following full impoundment. Using 1975 as a base year in the sediment budget analysis nonetheless underestimates the change from the natural condition in regions 1, 2, and 6.

With full impoundment in the summer of 1976 the lake level was raised above established beaches and bedrock outcrops into the unconsolidated overburden that comprised 86% of the new shoreline. High rates of shoreline erosion occurred in all regions of moderate to high fetch. Shoreline rates of erosion were found to be largely energy dependent (Newbury and McCullough 1984). Negligible material was removed from shorelines in protected bays with low fetches (<1 km). Rather, new shoreline profiles in such areas were the product of permafrost melting and subsequent slumping. The erosion estimates of Newbury and McCullough (1984) and E in Table 4 specifically exclude these areas. Maximum eroded input occurred in region 3–4 (Table 4), which has the largest mean fetches and extensive actively receding shorelines.

Although numerous studies have found cohesive clays to be less erodible than sands under flowing water conditions (e.g. Hjulstrom 1939; Terwindt et al. 1968), in SIL, unfrozen sand banks and clay banks with permafrost eroded at similar rates under similar energy regimes. The excavation process on SIL shorelines produced clay aggregates of a wide size range. Subaerial exposure of undercut collapsing clay blocks produced a highly fractured appearance in the laminated clay. The fracture lines may have followed planes of dehydration created by permafrost and surface drying. On disturbance, these blocks crumbled along fracture planes into chunks of a variety of sizes. Fractionation of the blocks was further assisted by preferential erosion of the silty laminae (fig. 6 in Newbury and McCullough 1984). Clay aggregates could also have been detached from submerged clay layers under turbulent pressure variations (Terwindt et al. 1968) caused by breaking waves. Although some suspended material entered the lake directly as a slurry where permafrost melting was widespread, the great bulk of material entered as aggregates of various sizes.

Transport

Upon submersion of eroded shoreline material, bed load transport initially predominated. Sorting of the blocky aggregates excavated from shorelines began as smaller sized blocks were transported to greater depths than large blocks. Transport along the bottom caused rounding of the blocks to typical beach pebble shapes (Fig. 8). Because the clay aggregates were easily abraded, they were worn smaller each time they were entrained and continually moved to lower energy regimes farther offshore.

Deposition

It was estimated that 90% of eroded material was deposited within SIL by analysis of the suspended sediment budgets for the first 3 yr of impoundment. Of this deposited material, 70–80%, depending on the region, was deposited nearshore. The observed portion of nearshore accumulation off an eroding sand shoreline (site 18) was 85%. Similarly, at an energetic clay shoreline (site 1) with a maximum fetch of 11 km, 75% of eroded material was deposited within 300 m of the shoreline. The agreement of observed and expected deposition at these sites supports the general conclusions of the sediment budget. The exceptional situation at site 11, where most of the eroded material was removed from the nearshore zone, may be typical of the ultimate fate of all the nearshore deposits after the shoreline has stabilized on bedrock, and there is no supply of new sediment. At site 11 the high wave energy from a 22-km fetch had partly reexposed the bedrock foreshore by 1979.

Nearshore deposition rates were initially rapid off eroding clay shorelines (up to $10 \mathrm{cm \cdot yr^{-1}}$ at site 1). Rapid nearshore deposition is frequently observed in new reservoirs (e.g. Everdingen 1968) where sediments are resistant to abrasion and these deposits persist through time. However, in SIL these nearshore clay deposits will be unstable in the long term. Entrainment of clay aggregates leads to particle abrasion, particle size reduction, and particle movement farther offshore. At site 1 even the largest aggregates observed at offshore sites were substantially smaller than those that could be moved by waves generated by winds whose high strengths are not infrequent (Table 9). Deposits at site 1 have accumulated nearshore because rates of sediment input have greatly exceeded rates of offshore transport. When new stable shorelines occur, sediment input will decrease (Newbury and McCullough 1984) and the nearshore deposits will eventually be removed by storm waves.

The comparative stability of sand and clay beaches is evident in the cross-sections of the deposits. At the sand site (Fig. 6) where well-sorted proglacial sands were eroding, a discrete sand wedge accumulated. The surface of the new deposit was parallel to, but 3 m higher than, the preimpoundment offshore profile. In contrast, the new deposit off the clay shoreline was thickest at 2 m water depth and diminished slowly over several hundred metres (Fig. 7) as aggregate sizes diminished and the total mass of material loss to suspended transport increased. At site 11 where bedrock had been exposed by shoreline erosion, only 20% of the eroded volume remained nearshore and of this, over 90% was igneous sand, which will likely be a stable deposit at this site. Any clay sediments that may have been deposited immediately after impoundment had been transported farther offshore by 1982.

If it were assumed that the whole eroded mineral volume entered the lake as bed load and that the 20–30% annual loss to suspended sediment transport, suggested by the sediment budget analysis, was generated by abrasion of bed load, a mean annual erosion constant of $0.25 \cdot \mathrm{yr^{-1}}$ might be applied to the nearshore clay deposits in SIL. A logarithmic time decay model applied to erosion of these nearshore clay deposits, like that applied by Newbury and McCullough (1984) for stabilization of bank erosion, would estimate that 10 yr would be required for 90% removal of these clay deposits to offshore waters and outflow. However, significant net erosion of these deposits would not occur until the input of material from shorelines decreased substantially. Consequently, the high turbidities in SIL (Hecky 1984) caused by impoundment will continue for significant lengths of time after shoreline excavation ceases. Newbury and McCullough (1984) estimated that 35 yr may be

TABLE 9. Comparison of maximum grain diameter observed along a nearshore depth profile at site 1 and the size of particle that could be entrained by a strong wind (sustained wind of 45 km·h^{-1}, 0.9-m wave with 3.8-s period; for method see Komar and Miller 1973). Wind speeds >45 km·h^{-1} were sustained for an average of 30 h in each open-water season from 1977 to 1979.

Distance offshore (m)	Depth (m)	Max. grain diameter observed (mm)	Max. grain diameter entrained (mm)
20	2	6	30
60	3	0.5	9
180	4.8	0.1	2

required for 90% of the fine-grained shorelines to return to their preimpoundment condition. Erosion of unstable nearshore deposits may require an additional decade before suspended sediment concentrations return to preimpoundment levels and the Churchill River is once again the major sediment source for SIL.

Uncertainty is attached to our estimates for subaqueous erosion of nearshore clay deposits. Rates of abrasion for any particular site must be related to eroding particle size and shape, hydrodynamic energy, and perhaps sand content of the deposits before a better model can be developed. Future research on this process should emphasize the generation of suspended sediment from abrading clay aggregates.

Impact

Only basins whose postimpoundment shorelines were largely bedrock (region 7 of Newbury et al. 1984) or sand (region 5) suffered little effect on their suspended sediment budgets and offshore suspended sediment concentrations (Hecky 1984; Patalas and Salki 1984). The immediate effects of bank erosion are most dramatic nearshore where high turbidities and high net sedimentation rates occur. This deposition is unstable and is occurring in areas that before impoundment had bedrock, cobble, gravel, and sand bottoms. Substantial changes in the utilization of these areas by biological communities have occurred (Fudge and Bodaly 1984; D. Rosenberg, Department of Fisheries and Oceans, pers. comm.). In the long term, these deposits will be removed; but, in the process of removal, the offshore effects of suspended sediments on light transmission (Hecky 1984) and planktonic communities (Hecky and Guildford 1984; Patalas and Salki 1984) will persist even after shorelines have stabilized. As the nearshore deposits are eroded, most of the suspended sediment will be removed at the outflow in the rapidly flushed regions 0–1, 2, and 6. Even in the large and slowly flushed region 3–4, loss at outflow can still predominate because much of the suspended material is fine grained (<1 μm nominal diameter). Although in this region material may be removed from shorelines and nearshore deposits more rapidly than from the other regions because of its greater fetches, its loss rate of suspended material is lower because of its slow flushing rate. The return of region 3–4 to a preimpoundment state with respect to suspended sediment concentration may be as slow as the basins that have smaller fetches and lower wave energies.

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Environmental Impact Prediction and Assessment: the Southern Indian Lake Experience[1]

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Hecky, R. E., R. W. Newbury, R. A. Bodaly, K. Patalas, and D. M. Rosenberg. 1984. Environmental impact prediction and assessment: the Southern Indian Lake experience. Can. J. Fish. Aquat. Sci. 41: 720–732.

The impoundment of Southern Indian Lake (SIL) and diversion from the lake of the Churchill River in northern Manitoba, Canada, were the subjects of two independent environmental impact statements. Subsequently, a case study measured change in the limnological and biological characteristics of the lake after development. Comparison of pre- and post-impoundment observations allows an assessment of the predictive capability that was applied to the lake by the preimpact statements. Predictions related to the physical environment, e.g. increased shoreline erosion, littoral sedimentation, higher turbidity, and decreased light penetration and visibility, were qualitatively correct; however, an unpredicted decrease in water temperature also occurred. Increased phosphorus availability and light limitation of primary production were also correctly forecasted in a qualitative manner. These aspects will be quantitatively predictable in future reservoirs because of studies at SIL and elsewhere. Biological responses above the primary trophic level were mostly not predicted or predicted incorrectly. Unpredicted changes that were especially significant to the fishery were rapid declines in the quantity and quality of whitefish (*Coregonus clupeaformis*) catch, increases in mercury concentrations in fish, and the need for extensive compensation programs to keep the fishery economically viable. Testable hypotheses to explain all unpredicted events have been formulated but require experimental verification. The paradigm of reservoir ecosystem development that is present in the literature requires reformulation if future environmental impact analyses of reservoirs are to be improved.

La retenue des eaux du lac Sud des Indiens et a dérivation de la rivière Churchill, dans le Manitoba septentrional (Canada), ont été l'objet de deux évaluations environnementales. Dans une étude de cas subséquente, les changements de caractéristiques limnologiques et biologiques du lac après la montée des eaux ont été mesurés. Une comparaison des observations d'avant et d'après retenue permet d'évaluer la capacité prévisionnelle, qui a été appliquée au lac à l'aide des évaluations environnementales d'avant la retenue. Les prédictions touchant l'environnement physique, p. ex. érosion accrue de la ligne de rivage, sédimentation du littoral, turbidité plus forte et pénétration de la lumière et visibilité réduites, ont été qualitativement exactes; cependant, il s'est produit une diminution non prédite de la température de l'eau. Une plus grande accessibilité du phosphore et la limitation par la lumière de la production primaire ont été également correctement prédites de manière qualitative. Grâce aux études menées au lac Sud des Indiens et ailleurs, il sera possible de prédire ces aspects quantitativement dans de futurs réservoirs. Les réactions biologiques au-delà du niveau trophique primaire ont été pour la plupart non prédites, ou encore prédites incorrectement. Parmi des changements non prédits, et particulièrement importants pour la pêche, on note de rapides déclins de la quantité et de la qualité des prises de grand corégone (*Coregonus clupeaformis*), des augmentations de concentration de mercure dans les poissons et le besoin de programmes compensatoires intensifs afin de maintenir la rentabilité de la pêche. Nous avons formulé des hypothèses vérifiables pour expliquer tous les événements non prédits, mais ces hypothèses devront être vérifiées expérimentalement. Il faudra formuler de nouveau le paradigme de développement des écosystèmes des réservoirs contenu dans les travaux publiés, si l'on veut améliorer les analyses d'évaluation environnementale des réservoirs.

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Any major industrial or resource development today will likely be required to undergo an environmental impact assessment, i.e. consideration and evaluation before development of probable effects, either by law, regulation or public demand. Impact assessments have become so ubiquitous and bureaucratized that it is easy to forget that they are recent phenomena. Public awareness of environmental problems grew rapidly through the 1960's (Parlour and Schatzow 1978; Beanlands and Duinker 1983), and many governments responded by institutionalizing environmental impact assessments in the early 1970's. Although environmental impact assessments are now commonplace, strong misgivings have been registered about their purpose, structure, conduct, utility, and effectiveness (e.g. Schindler 1976; Rosenberg et al. 1981; Beanlands and Duinker 1983). This is as true of reservoir

[1]This paper is one of a series on the effects of the Southern Indian Lake impoundment and Churchill River diversion.

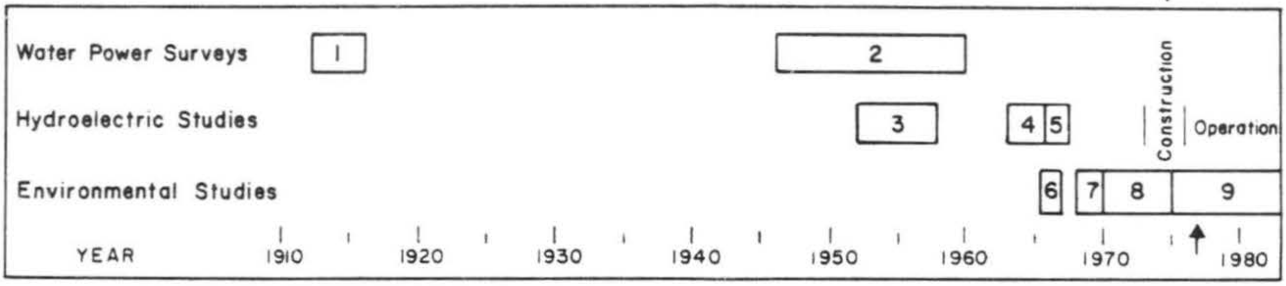

FIG. 1. Time frame of Churchill River Development. Sources: 1, Denis and Challies (1916); 2, Ramsay (1947) and Godfrey (1957); 3, Lake Winnipeg and Manitoba Board (1958); 4, Nelson River Programming Board (1967); 5, Gibb, Underwood-McLellan and Associates Ltd. (1968); 6, University of Manitoba (unpubl. manuscr.); 7, Underwood-McLellan and Associates Ltd. (1970d); 8, Lake Winnipeg, Churchill and Nelson Rivers Study Board (1975); 9, Freshwater Institute Studies (in this issue). Arrow marks signing of Northern Flood Agreement, which has a mechanism for handling compensation claims.

developments as of other developments (Rosenberg et al. 1981) even though man's experience with reservoirs began with the earliest organized civilizations (Wittfogel 1957).

The long association between man and reservoirs might lead one to think that all the effects of reservoir formation would be well known. This opinion has been recently expressed through statements such as "it seems unlikely that subsequent impoundments in the temperate regions will give rise to any large-scale surprises" (Baxter 1977, p. 277). Others such as Efford (1975, p. 197) have stressed the opposite view that "Our ability to measure the impact of hydro-dams on biological systems is not well developed." This apparent divergence of opinion is understandable when the paucity of postimpact environmental data for reservoirs is considered (Geen 1974). In a review of environmental impact assessment (Rosenberg et al. 1981), postdevelopment monitoring and analysis were identified as the most frequent deficiencies of the six necessary components of an "ideal" scientific impact assessment. The scientific method requires both the posing of testable hypotheses (environmental prediction) and hypothesis testing (postdevelopment analysis). Any evaluation of the utility of environmental impact assessments must consider the predictive capability of environmental science and consequently the comparison of prediction and results. The objective of this paper is to examine these two aspects of environmental impact assessment using the impoundment of Southern Indian Lake (SIL) and accompanying diversion of the Churchill River as an example.

Historical Background

Federal water power surveys in the early 1900's (1, Fig. 1) identified the hydroelectric potential of the Churchill River in northern Manitoba, Canada. Extensive provincial surveys beginning in the 1940's (2, Fig. 1) confirmed this potential, and hydroelectric feasibility studies commenced. By the mid-1950's (3, Fig. 1) the possibility for diversion of the Churchill River flow into the Nelson basin had been discovered. In the 1960's feasibility studies by Manitoba Hydro (4 and 5, Fig. 1) indicated that a high-level impoundment of SIL (i.e. increasing its level by 10 m to 265 m ASL) and diversion of water from the Churchill River into the Nelson River would optimize electric generation benefits relative to all other possible system configurations (Dickson 1975). There was public resistance to this scheme primarily because the feasibility studies had not considered the effect of such a development on the existing and future utilization of natural resources in northern Manitoba. As often happens in such developments, natural resource impacts were addressed late in the planning process (6, Fig. 1). In response to public concern, Manitoba Hydro retained a consulting engineering firm to assess the impact of various diversion configurations on natural resource utilization (Underwood-McLellan and Associates Ltd. 1970a). This was the first published predevelopment impact assessment (7, Fig. 1). Subsequently, Manitoba Hydro opted for a low-level impoundment and diversion scheme that would flood SIL by 3 m to a maximum elevation of 258 m ASL. Continuing public concern about the effects of this option led to the establishment of a second predevelopment assessment (8, Fig. 1), a Federal–Provincial study to examine the environmental effects of a low-level impoundment (Lake Winnipeg, Churchill and Nelson Rivers Study Board 1975). This study proceeded concurrently with construction of the development. The Freshwater Institute of the Department of Fisheries and Oceans subsequently conducted a program of research on SIL, which continued after impoundment and diversion (9, Fig. 1). These follow-up studies can be used to assess the predictions made by the preimpoundment impact analyses.

Strategy

In this paper the impoundment of SIL and the diversion of the Churchill River is treated as a large-scale experiment conducted to test hypotheses made by predevelopment impact assessments, in order to evaluate the predictive capability of scientific theory being applied to new reservoirs during the early 1970's when the SIL assessments were done. We will then discuss how the theory might be modified based on the SIL experiment to achieve better predictability, especially in a quantitative sense, in future reservoir developments. In so doing, we will enumerate several new or revised hypotheses generated by consideration of the SIL experiment and suggest how they might be tested.

Methods

Sources of Predictions

The Underwood-McLellan and Associates Ltd. (U-M) study (1970a) and the Lake Winnipeg, Churchill and Nelson Rivers (LWCNR) study (1975) each had different terms of reference, purposes, and time frames for assessing systems with slightly different project configurations. To understand how we derived testable predictions from these studies, it is necessary to briefly review the objectives and methodologies of each.

The U-M study

It had "...the overall objective...to determine the costs and benefits of various schemes of diversion of the Churchill River" (Underwood-McLellan and Associates Ltd. 1970a, p. 8). The study was primarily an office study based on natural resource information already available. Virtually no new data were collected from the study area. Consequently, predictions were based on experiences in other reservoirs as derived from the scientific literature. The study (Underwood-McLellan and Associates Ltd. 19′ 0b) did not have a defined operating regime for any of the various diversion configurations considered, but it specifically gave predictions for a high-level diversion from SIL (maximum elevation 265 m ASL) and a low-level diversion (maximum elevation 256 m ASL) with several possible ranges of drawdown. The minimum range of drawdown considered was 1.5 m. In the configuration eventually developed (Newbury et al. 1984 and below), the maximum elevation has not exceeded 258 m. The annual drawdown is less than 2 m, which is similar to the preimpoundment annual range of water levels. Consequently, predictions from U-M based on drawdown effects have been eliminated from our analysis. We consider that if the U-M study had specifically made predictions for a 258-m elevation it would have interpolated between the severity of its expected effects at 256 and 265 m.

The LWCNR study

This study began after Manitoba Hydro had fixed the configuration, operating regime, and timing of construction. Churchill River water was to be diverted at up to $875\ m^3 \cdot s^{-1}$ from SIL into the Nelson River basin. Diversion would be by gravity flow, and the lake was not to be raised above 259 m ASL. Drawdown on SIL was to be similar to the natural range of water level fluctuation. Construction of control works and the diversion channel proceeded contemporaneously with the study. The objective of the study was, therefore, limited to making "...recommendations for enhancing the overall benefits with due consideration for the protection of the environment" (LWCNR 1975, p. 61). Specifically, the terms of reference directed that the study "...must be adapted to provide reliable data on present natural conditions and the anticipated and actual conditions arising from the operation of the controls as designed and constructed" (LWCNR 1975, p. 61). Field studies were conducted on SIL in 1972 and 1973; thus, predictions were based on a large amount of descriptive data. The natural state of SIL and predicted changes were summarized in Hecky and Ayles (1974).

Source of Results

An emphatic recommendation of the LWCNR study was that a long-term ecological monitoring and research program be conducted to establish the impact of the development. Subsequently, the Freshwater Institute initiated a case study of the SIL reservoir; studies funded by the Department of Fisheries and Oceans began in 1974 and have continued to the present. The operational regime for SIL over the period of postimpoundment studies has differed only slightly from that considered by the LWCNR study, i.e. although elevation of the lake has not exceeded 258 m ASL, diversion flows have exceeded $875\ m^3 \cdot s^{-1}$ from time to time during hydraulic capacity studies of the diversion channel undertaken by Manitoba Hydro (Newbury et al. 1984). The results of this case study are published as a series of 17 papers in this issue of the *Canadian Journal of Fisheries and Aquatic Sciences*. The major results of these studies are briefly reviewed here to characterize the natural state and to establish observed changes that occurred with impoundment and diversion.

Summary of Impacts

During the summer of 1976 the level of SIL was raised to 3 m above its long-term mean elevation. The lake was impounded by a control dam on the natural outlet at Missi Falls (Fig. 2) causing the water of the Churchill River to flow by gravity into the Nelson River basin via an excavated channel at South Bay (region 6, Fig. 2). Since September 1977, the diverted flow from the Churchill River has averaged 75% of the river's long-term mean flow of $1011\ m^3 \cdot s^{-1}$ out of SIL (Newbury et al. 1984). Impoundment increased the area of the lake by 21% to $2391\ km^2$. Preimpoundment assessment of the multibasin lake recognized eight regions defined by channel constrictions (Fig. 2; Newbury et al. 1984). These regions, in their natural state, differed in their flushing rates and water chemistry (Cleugh 1974), primary productivity (Hecky and Guildford 1984), biological communities (Hecky 1975; Patalas and Salki 1984; Wiens and Rosenberg 1984), and commercial fishing effort (Bodaly et al. 1984b). The impounded lake also had regional differences in extent of flooded area, change in mean depth, and nature of flooded banks (Newbury et al. 1984). The quantitative effects of impoundment on selected limnological variables in four major regions of the lake are summarized in Table 1.

Physical Changes

After impoundment, SIL was deeper and cooler, and these effects were apparent in all major regions (Table 1). Hecky (1984) concluded that the greater mean depth of all the regions in the unstratified lake had the effect of diluting incoming heat. Increased back-scattering of solar irradiance in regions of the lake with higher suspended sediments added to the general cooling effect as well. In regions north of the diversion point (regions 4 and 5 in Table 1), decreased riverine heat input in the spring delayed ice-out (Hecky 1984). Delayed ice-out reduced the length of the heating season and lowered the maximum temperatures achieved relative to the regions upstream of diversion. Patalas and Salki (1984) found that the temperature decreases in the upper 5 m (2–3°C) were even greater than the declines in whole water mass temperatures given in Table 1. Region 6 (South Bay), which before impoundment was a warm, shallow, isolated bay off the main axis of Churchill River flow, was cooled by the introduction of relatively cool diverted water from the deeper region to the north (Hecky 1984). The postdiversion flushing rate for this bay was too rapid to allow significant warming of the diverted water.

The postimpoundment water surface intersected glacial and organic deposits on 86% of the new shoreline (Newbury et al. 1984). Onshore waves initiated substantial erosion on all shorelines exposed to more than 1 km of offshore fetch (Newbury and McCullough 1984). In regions 1, 4, and 6, the predominant backshore material was permafrost-affected, glacio-lacustrine clays and fine-grained tills. The shoreline erosion introduced large volumes of these materials to the lake (Table 1). As much as 80% of this eroded material was initially deposited nearshore (Hecky and McCullough 1984). The remainder went into suspension and significantly increased the offshore sediment concentrations by 2–5 times (Table 1). In region 5, the shoreline was composed of sandy eskers, kames,

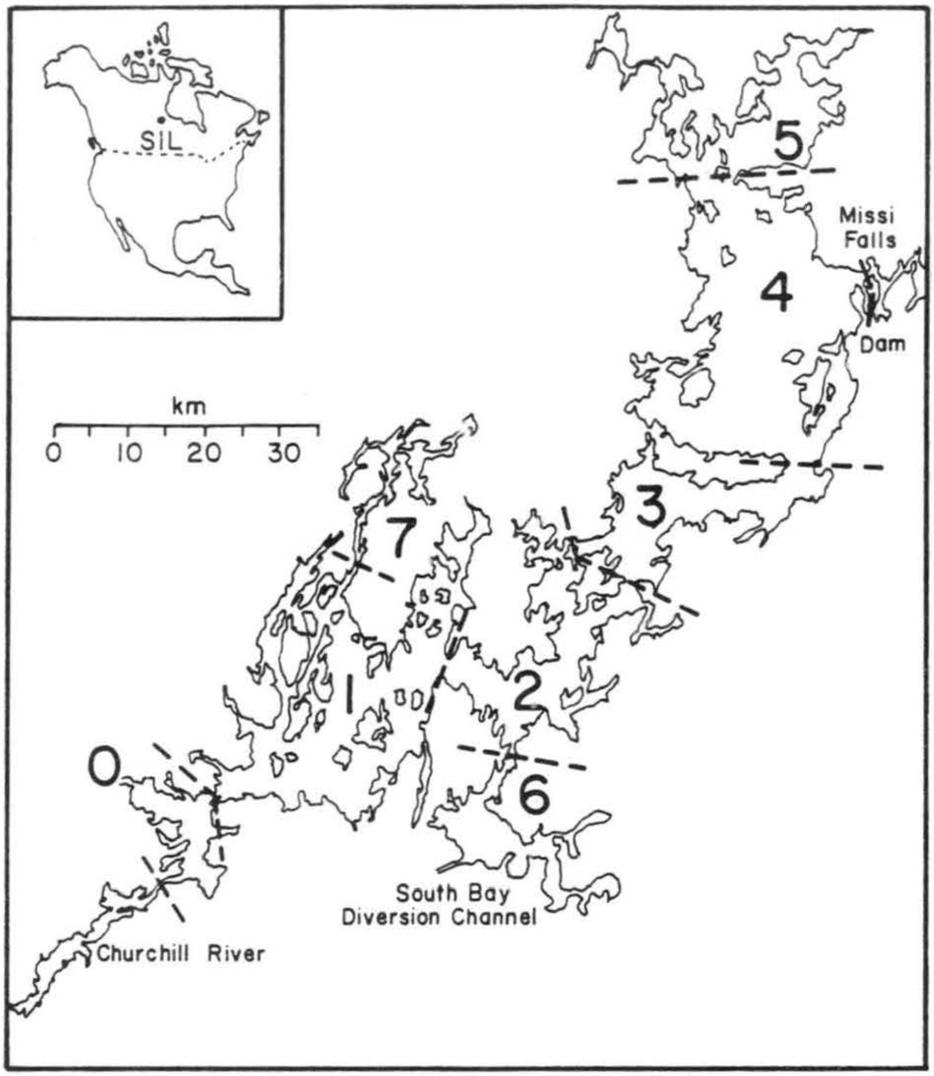

FIG. 2. Southern Indian Lake location map. Numerals are assigned to regions as defined in Newbury et al. (1984).

and organic deposits. Most of the sediment was deposited nearshore, and there was less effect on offshore sediment concentrations. In general, increased sediment concentrations offshore reduced Secchi disk transparency and light penetration. The mean water column light intensity, $\bar{I}$ (Table 1), was further reduced because of the greater mean depths of these well-mixed regions (Hecky 1984). After impoundment, SIL was, on average, a darker, less transparent lake than before.

Biological Changes

Primary productivity after impoundment either was increased in the relatively well-illuminated regions of the lake, e.g. region 5 (Table 1), or was unchanged in regions where $\bar{I}$ declined below $5.0\,mE \cdot m^{-2} \cdot d^{-1}$ (Hecky and Guildford 1984). In the latter regions, postimpoundment light limitation replaced preimpoundment phosphorus limitation with no significant change in integral production. Seasonal mean chlorophyll concentrations rose in all regions of the lake in response to less light and more nutrients from shorelines. Phosphorus deficiency was relieved in all regions of the lake except in small, shallow bays where $\bar{I}$ remained high enough to cause utilization of all available nutrients (Hecky and Guildford 1984; Planas and Hecky 1984). Such bays had significantly increased integral primary production.

The zooplankton and zoobenthic communities responded in opposite ways to impoundment. Zooplankton biomasses decreased by 30–40% in the major regions of the lake. Cladocerans and small cyclopoid copepod species accounted for most of the declines in biomass (Patalas and Salki 1984). Calanoid copepods were less affected, with some larger species actually being more abundant and more widespread after impoundment. *Mysis relicta* increased from being very rare to being common. Patalas and Salki (1984) attributed these changes in the zooplankton community to a combination of lower temperatures and reduced predation because of poorer transparency. The density of zoobenthos generally increased in SIL, and no significant changes in community composition occurred (Wiens and Rosenberg 1984). The magnitude of regional responses in zoobenthos (Table 1) could be explained in terms of nutrient and organic inputs from flooded shorelines, phytoplankton primary production, and concentrations of sus-

TABLE 1. Comparison of morphometric, hydrologic, limnological, and biological factors for four SIL regions before and after impoundment. Inundation ratio (Wiens and Rosenberg 1984) is the proportion of flooded land to postimpoundment water area. Mean depth and flushing rate are from Newbury et al. (1984). Temperature change is based on mean water mass temperature at maximum heat content (Hecky 1984). Seasonal mean suspended sediment concentrations were calculated by Wiens and Rosenberg (1984) from data of Hecky et al. (1979) for years 1974 and 1977. $\bar{I}$ is the mean water column light intensity during the day for 2 yr before and 2 yr after impoundment (Hecky and Guildford 1984). Secchi values are from Hecky (1984). Erosive inputs are the means for the first three postimpoundment years; erosive inputs were negligible before impoundment (Hecky and McCullough 1984). Primary production and chlorophyll data are from Hecky and Guildford (1984). Zooplankton biomass estimates compare the postimpoundment mean with 1972 (Patalas and Salki 1984), and zoobenthos density compares 1977 with 1972 (Wiens and Rosenberg 1984). Underlined pairs of data were considered not significantly different statistically by the authors. Suspended sediment, $\bar{I}$, Secchi disk, primary production, chlorophyll, and zoobenthos density values for region 6 are for the western subregion (Hecky 1984), which accounts for 62% of the area and 72% of the volume of region 6. All other parameters are calculated for the whole region.

	Region							
	1		4		5		6	
	Pre	Post	Pre	Post	Pre	Post	Pre	Post
Inundation ratio	0	0.09	0	0.16	0	0.31	0	0.08
Mean depth (m)	8.0	10.1	12.1	13.0	4.9	5.9	3.5	5.8
Flushing time (yr)	0.12	0.17	0.23	1.4	1.5	2.8	4.2	0.03
Temperature change (°C)	0	−0.8	0	−1.3	0	−1.4	0	−1.3
Suspended sediment ($mg \cdot L^{-1}$)	3.2	8.1	1.2	6.3	1.7	4.1	3.0	11.0
$\bar{I}$ ($mE \cdot m^{-2} \cdot min^{-1}$)	6.2	4.0	10.0	4.9	15.9	9.8	13.9	5.5
Secchi disk (m)	1.4	0.9	2.9	1.3	3.0	2.3	1.6	0.7
Erosive input ($g \cdot m^{-2} \cdot yr^{-1}$)	≈0	1390	0	3312	0	770	≈0	1700
Primary production ($mg \cdot m^{-2} \cdot d^{-1}$)	530	460	570	560	400	720	220	290
Chlorophyll ($mg \cdot m^{-3}$)	4.6	5.0	2.9	4.0	2.4	4.4	1.9	3.6
Zooplankton biomass ($mg \cdot m^{-3}$)	905	707	930	625	1855	957	1486	933
Zoobenthos density ($no. \cdot m^{-2}$)	6200	5500	3800	8300	2800	6100	1000	1500

pended sediments. In some areas, high suspended sediment concentrations apparently negated the increased input of organic substrate. Crawford and Rosenberg (1984) showed that one major source of organic substrate, black spruce (*Picea mariana*) needles, was quickly colonized and broken down mainly by chironomids. This study suggested that such flooded vegetation would be rapidly utilized. Within 3 yr after impoundment there was evidence that the zoobenthic densities were returning to preimpoundment levels in some regions (Wiens and Rosenberg 1984).

Fisheries

The catch per unit effort (CUE) of whitefish (*Coregonus clupeaformis*) on the traditional fishing grounds declined after impoundment (Bodaly et al. 1984b). This decline caused a redistribution of commercial fishing effort from region 4 to region 5 in order to exploit lower quality (darker color, higher *Triaenophorus crassus* cyst counts) stocks that were formerly avoided. The lower CUE was attributed to redistribution of whitefish stocks, which may have been due to reduced visibility either affecting feeding success, which in turn caused movement to more favorable conditions, or affecting schooling behavior (Bodaly et al. 1984b). Total whitefish catch was maintained for 5 yr after flooding by increased total fishing effort, but then, effort slowed and total catch declined. Patalas and Salki (1984) noted that the increases in *Mysis* and other large calanoid copepod abundances concurred with the apparent decline in whitefish abundance, suggesting reduced whitefish predation on food resources. Reduced predation by adult whitefish may also have contributed to the general increase in zoobenthos as well. In situ experimental incubations showed that high sedimentation of silts and clays negatively affected whitefish egg survival (Fudge and Bodaly 1984), creating concern for the long-term abundance of whitefish in SIL.

Impoundment produced a very strong year-class of northern pike (*Esox lucius*) in 1977, the 1st yr of high spring water level, but young pike were much less abundant in subsequent years (Bodaly and Lesack 1984). Adult pike showed no effect of impoundment on growth, mortality, or condition. The lack of response in adult pike populations was attributed to the low degree of flooding in the chosen study area on SIL (Bodaly and Lesack 1984). Mercury concentrations in muscle increased in all commercial fish species after flooding (Bodaly et al. 1984a). Northern pike and walleye (*Stizostedion vitreum vitreum*) exceeded the Canadian marketing limits for mercury concentration in flesh and in some cases exceeded the marketing limit for export. These increases in fish mercury concentration also threatened an important domestic food source.

Compensation

The Northern Flood Agreement, among the federal and provincial governments, Manitoba Hydro, and five Indian reserves affected by the Churchill–Nelson River hydro development, was signed in 1977 (Fig. 1), and it included a mechanism for arbitration of compensation claims. The South Indian Lake community was not signatory to this agreement and may not be covered by it. However, as the local economy was quite dependent on the fishery, a series of annual compensation packages were negotiated between Manitoba Hydro and the South Indian Lake Fishermen's Association over the period 1978–82. These compensation plans differed in size and intent from year to year and ranged in total value from approximately \$40 000 to \$600 000. They included, at various times, provisions for bonus payments paid on a per pound basis, replacement of lost or damaged nets and motors, fish grade sorters, assistance for fishing nearby lakes unaffected by flooding, and payments to individual fishermen based on historic production. In early 1983, a final settlement of $2.5 million was agreed upon, which would compensate for all present and future damages to the fishery. Wagner (1984) found that the postimpoundment commercial fishery would not be economically viable without the compensation payments that were being made for value losses attributed to the impoundment, and he concluded that fishermen were undercompensated because the extra effort required to recover a marketable catch was not recognized as an increased cost.

Results and Discussion

Reservoir Theory

The paradigm

The U-M study surveyed the reservoir literature to establish a general catalogue of responses by aquatic ecosystems to impoundment. The literature available at the time demonstrated that many new reservoir ecosystems had a similar developmental sequence. Although the majority of reports was based on Russian experience (summarized in column 1 of Table 2), the developmental sequence was reinforced by the smaller number of experiences from the tropics and North America, and similar characterizations can be found in numerous reviews (e.g. Frey 1967, Lowe-McConnell 1973; Baxter 1977). We will refer to the developmental sequence in column 1 of Table 2 as the "reservoir paradigm." A scientific paradigm, sensu Kuhn (1970), is a widely accepted model or pattern for a phenomenon, the correctness of which is not seriously questioned by practicing scientists beyond minor modifications in articulation. The reservoir paradigm makes the general prediction of an initial trophic upsurge during which all biotic communities have higher standing crops and productivities. Much of the experience on which the paradigm was based was biased towards valley reservoirs where new bodies of water were created from initially riverine conditions.

The U-M study

SIL was already a lake that would undergo marginal flooding, so the U-M study modified the existing paradigm (Table 2, column 2). It was recognized that shoreline erosion would be the major physical impact. Erosion was a concern in SIL because of long, open fetches available for wind-generated waves to form, which would dissipate on the widespread, erodible, fine-grained glacio-lacustrine deposits in the permafrost-affected backshore. Appreciation of the potential for erosion, in turn, modified expected littoral productivity in the vicinity of actively eroding shorelines and led to a prediction of decreased zoobenthic productivity in offshore areas because of increased sedimentation (Table 2, column 2). Positive trophic responses leading to increased food availability for fish were only expected in protected nearshore areas of low erosion, so only the littoral dwelling pike (of the commercially important fish species) were predicted to increase in productivity after impoundment. Spawning problems were predicted for the most important commercially fished species, whitefish and walleye, and these problems were projected to lead to falling production for these species after impoundment with a slow return to preimpoundment levels as new spawning habitat evolved. The prediction of only marginal increases in fish production after impoundment resulted from an integration of food chain effects and spawning effects and was a significant departure from the paradigm.

The LWCNR study

A year of field study and a more defined operating regime allowed refinement of some of the U-M study predictions, elimination of others, and generation of one significant new prediction concerning decline in CUE in the commercial fishery (Table 2, column 3). Predictions about thermal stratification and deoxygenation were more definitive. The prediction that shoreline erosion would be substantial was modified by stating that most of the eroded material would be deposited nearshore. Offshore turbidity increases were expected to reduce drastically light availability to phytoplankton and nullify the effect of nutrient increases on algal growth after impoundment. Consequently, it was predicted that offshore primary production over most of the lake would not increase after impoundment. The LWCNR suggested that offshore effects on zoobenthos would not be as negative as was anticipated in the U-M study; however, no specific prediction on benthic productivity in the immediate, postimpoundment period was offered because of uncertainty concerning the interaction among nutrients, sedimentation, and zoobenthos. Most of the predictions concerning fish ecology made by U-M were adopted by LWCNR. In the commercial fishery no increase in production was expected, and a decline in the CUE was predicted by LWCNR for the immediate postimpoundment period because of stock movements in response to the changed environmental regime. The LWCNR study had clearly abandoned the reservoir paradigm as not applicable to SIL (Table 2, cf. columns 1 and 3).

Comparison of Predictions and Observations

Although many of the predictions that were made and that can be presently assessed were qualitatively correct (Table 2, column 4), some predictions were wrong (Table 2, column 5). For example, pike spawning success became poor after being very successful in the 1st yr after impoundment. Also, growth and survival of pike were not detectably enhanced by impoundment. It had been feared that lake whitefish required shallow water over rocky bottoms for spawning and would not find such suitable substratum immediately after impoundment, but they continued to spawn on their original sites at greater depths.

A number of significant and totally unpredicted impacts occurred (Table 2, column 6). The decline in whitefish market quality contributed to the economic decline of the SIL commercial fishery (Wagner 1984). The contamination of fish by mercury threatened a major domestic food source and the marketability of the piscivorous walleye and pike. Other oversights by both LWCNR and U-M included significant changes in zooplankton and zoobenthic standing crops and a general cooling of the impounded lake.

The correct predictions of U-M and LWCNR were highly modified from the reservoir paradigm. The U-M study had decided that the general trophic upsurge expected from the reservoir paradigm would be limited to a few components of the food chain and/or to restricted portions of the reservoir and it anticipated only marginal increases in fish production. The LWCNR study predicted that there would be no increase in the production of important commercial stocks, and it forecasted a decline in CUE in the fishery. The U-M "office" study was nearly as effective in its predictions of aquatic impacts as the LWCNR study was after a year of field studies on SIL. The year of field studies allowed the LWCNR study to better define some

TABLE 2. Comparison of preimpoundment predictions with postimpoundment observations. Predictions were made with increasing familiarity with the system from left to right, reservoir paradigm, through Underwood-McLellan (U-M), to the Lake Winnipeg, Churchill and Nelson Rivers Study (LWCNR). Observations are grouped as to whether they were correctly predicted, incorrectly predicted, or unpredicted. Page numbers in parentheses for individual entries refer to the reference at the top of the column. Predictions and observations are identified with numbers and letters, e.g. 1a, to facilitate comparison down and across the table.

Predictions			Observations		
The reservoir paradigm (USSR, Rzoska 1966)	U-M (Underwood-McLellan and Associates Ltd. 1970b)	LWCNR (Hecky and Ayles 1974)	Correctly predicted	Incorrectly predicted	Unpredicted
		I. Physical factors			
(1a) Thermal stratification may appear	(1a) Thermal stratification may appear	(1a) No thermal stratification (p. 16)	(1)a No thermal stratification (Hecky 1984)		(1a) Mean lake temperatures and surface water temperatures decreased (Hecky 1984; Patalas and Salki 1984)
(1b) Deoxygenation may occur in hypolimnion	(1b) Deoxygenation in shallow bays with extensive flooding	(1b) Deoxygenation only in immediate vicinity of flooded soils (p. 19)	(1b) No significant deoxygenation in SIL (Bodaly et al. 1984c)		
	(1c) Intensive shoreline erosion and increased turbidity (p. 82, 85)	(1c) Extensive shoreline erosion will increase sedimentation rates, especially nearshore, and turbidity will increase (p. 19)	(1c) Shoreline erosion extensive (Newbury and McCullough 1984) Nearshore sedimentation rates high (Hecky and McCullough 1984) Turbidity increased (Hecky 1984)		
		II. Nutrient factors			
(2a) Increased nutrients due to leaching from flooded soils	(2a) Increased nutrients due to leaching from flooded soils (p. 84)	(2a) Increased nutrients due to active shoreline erosion and decay of vegetation (p. 18)	(2a) Phosphate concentrations increased (C. Anema, unpubl. data); phosphorus deficiency in algae eliminated (Hecky and Guildford 1984) Spruce needles are significant carbon input and are rapidly degraded by benthos (Crawford and Rosenberg 1984)		
		III. Algae			
(3a) Initially higher phytoplankton productivity	(3a) Higher offshore primary production for first 3–5 yr (p. 84)	(3a) No increase in offshore primary production over most of lake; probably lower primary production nearshore in exposed areas of high wind fetch (p. 19). In protected areas with high transparency, production will increase in short term	(3a) No significant increase in primary productivity in turbid regions of lake. Regions with high transparency showed increased production (Hecky and Guildford 1984)		
		IV. Zooplankton			
(4a) Decrease in number of species	No specific predictions	No short-term predictions			(4a) No decline in number of species
(4b) Higher biomasses, especially in Crustacea					(4b) Decline in numbers and biomass of zooplankton (Patalas and Salki 1984)

TABLE 2. (*Concluded*)

Predictions			Observations		
The reservoir paradigm (USSR, Rzoska 1966)	U-M (Underwood-McLellan and Associates Ltd. 1970b)	LWCNR (Hecky and Ayles 1974)	Correctly predicted	Incorrectly predicted	Unpredicted
		V. Zoobenthos			
(5a) Extensive development in nearshore areas, especially chironomids (5b) Usually increased biomass in profundal in first few years	(5a) Extensive development in protected flooded areas (5b) Decrease near eroding shorelines and offshore (p. 83, 85)	No short-term predictions	(5a) Extensive development of chironomid populations in flooded vegetation (D. M. Rosenberg and A. P. Wiens, unpubl. data)		(5b) Increase in profundal benthos immediately following impoundment and continuing to present (Wiens and Rosenberg 1984 and unpubl.)
		VI. Fish			
(6a) Higher growth rates initially because of abundant food (6b) Some spawning problems may occur	(6a) Forage fish do well in protected flooded areas (p. 83) and pike habitat increased (p. 83, 87) (6b) Reduced spawning success of whitefish and walleye (p. 87)	(6a) Increase in pike production (because of increased spawning habitat, growth, and survival) but no short-term increase in commercially important species, i.e. whitefish and walleye (6b) Spawning problems may occur for walleye and whitefish in short term (p. 19), as old spawning grounds would be drowned and subject to sedimentation		(6a) Poor pike spawning; no increase in survival or growth of pike (Bodaly and Lesack 1984) (6b) Whitefish spawning on old grounds, but there may be problems of egg survival (Fudge and Bodaly 1984)	
		VII. Commercial fishery			
(7a) Significantly increased fish yields	(7a) Marginal increases in fish production initially	(7a) No short-term increase in productivity (p. 21, 22) (7b) Decline in CUE in commercial fishery in short term because of stock movements (p. 21, 22)	(7a) No short-term increases in fish yield (7b) CUE of commercial fishery declined (Bodaly et al. 1984b)		(7c) Quality of whitefish catch declined (Bodaly et al. 1984b) (7d) Increased mercury concentrations in fish threatened marketing of predatory species and poses health hazard to domestic consumption (Bodaly et al. 1984a) (7e) Fishery requires compensation for survival (Wagner 1984)

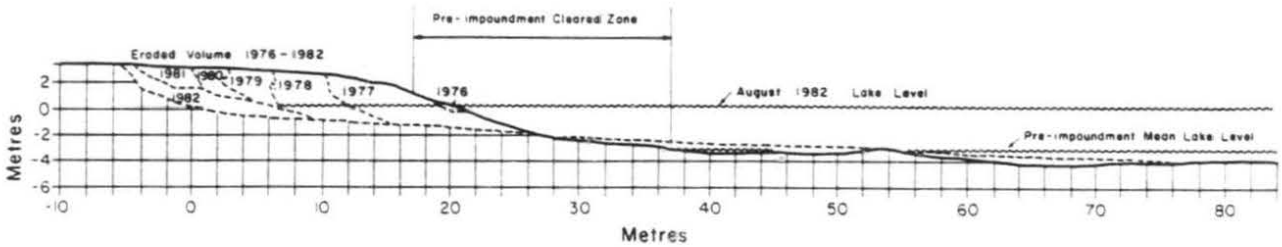

FIG. 3. Cross-section of an eroding shoreline on Southern Indian Lake that was cleared of timber before flooding. Material removed in succeeding years is indicated by broken line on bank cross-section. Material deposited nearshore is indicated by broken line under the new water level. Note that the shoreline zone cleared of timber was eroded away within 1 yr of flooding.

of the U-M predictions and to modify significantly a few others although much of this improvement resulted from having better defined project configuration. The most important contribution of the LWCNR study, a requirement of its terms of reference, was to define baseline conditions so that subsequent impacts could be evaluated.

The utility of both assessments, however, was marginal for two principal reasons. First, there were a number of highly significant impacts that were not predicted. Second, even correct predictions were usually qualitative, i.e. either they were not based on direct numerical estimates or they were quantified incorrectly. This second aspect is significant because decisions for major resource development are generally based on quantitative benefit/cost data. A qualitative statement alone, even if correct, often cannot directly enter such benefit/cost analyses. We will now review the attempts at quantification made in predevelopment studies in order to suggest how improvements might be made in the future.

Quantification of Predictions

The U-M study

The reservoir paradigm supplied to the U-M study a series of qualitative predictions about new reservoirs, but it did not offer a set of quantitative relationships for estimating changes in ecosystem parameters. However, quantification of resource impact was forced upon the U-M study by its terms of reference, which required the monetary assessment of costs and benefits of various diversion and impoundment options. The impacts identified by U-M were scaled against depth of flooding or extent of drawdown depending on which was recognized as causing the impact. For example, nutrient and organic loading from flooded terrain to the new reservoir was assumed to increase in proportion to the depth of flooding. In turn, primary production was assumed to be nutrient dependent and would thus increase with nutrient loading. This scalar approach to impact assessment was directly analogous to the linear programming model used by hydraulic engineers to optimize the choice of hydraulic structures where hydraulic head, flow, and usable or "live" storage determine power benefits. The creation of a numerical scale for the impact on biological systems allowed the generation of a similarly scaled, quantitative estimate of impact on the fishery resource based on the current market value of the resource. The assessment had to be stated in monetary terms to integrate it with the power benefits of various project configurations.

The U-M scalar approach to impact prediction allowed quantitative predictions to be made, but they were not derived from confirmed relationships in the reservoir literature. This approach created an idealized SIL lake-reservoir about which predictions could be made. Unfortunately, the idealized reservoir was an illusion, and it led to erroneous conclusions. For example, consider this statement offered without substantiation: "Fish populations in an artificial impoundment will be significantly affected if water levels are increased more than 10 feet and if drawdown exceeds five feet" (Underwood-McLellan and Associates Ltd. 1970c, p. 73). By implication, an impoundment created within this configuration would not have significant effects on fish populations. In fact, SIL was impounded and operated within a 10-ft (3.1 m) elevation change and a 3-ft (0.9 m) drawdown, but there have been significant effects on fish populations (Table 2). What went wrong?

On SIL, a significant change in the lake occurred as soon as the natural range of lake levels was exceeded. Shoreline erosion of the previously stable, forested backshore began introducing large quantities of organic debris, clay, and sand into the lake. Suspended sediment concentrations increased dramatically as did sedimentation, particularly in the lake's littoral zone off eroding shorelines. At exposed shorelines with high wave energies, erosion rapidly cut into banks and removed material well beyond the initial water contact (Fig. 3). At protected shoreline sites, the rates of shoreline recession were lower, but materials were still added in excess of the initial zone of flooding as permafrost-affected banks melted and settling of the backshore zone occurred.

Erosion proceeds at any given water level until a bedrock surface is contacted (Newbury and McCullough 1984); therefore, impoundment levels higher than 3 m would have produced only minor modifications to the longevity and extent of shoreline restabilization. The U-M study using a scalar approach underestimated the effect that shoreline erosion would have on the whole lake at low levels of flooding. Consequently, the study erred in accepting an apparently minor change in lake level as insignificant for fisheries because impacts on SIL did not increase in a linear manner with depth of flooding. Rather, the ecosystem endured a discrete change with the first water level increase over the natural range at which shorelines had become stable over several thousand years.

After the decision was made to proceed with the low-level flooding, the erroneous scalar analysis was manifested by an extensive timber clearing program for soon-to-be-flooded forested areas in various parts of the lake. The western half of South Bay (region 6, Fig. 2) was cleared to the 259-m (850 ft) ASL elevation (1 m above regulated water level) at a cost of several million dollars. The folly of this clearing program, either for aesthetic purposes or to retard debris generation, is obvious from Fig. 3. Over most of South Bay, the cleared zone was entirely eroded in the 1st yr of impoundment, and the present lakeshore in western South Bay is advancing inland as a steep vertical face draped with hanging terrestrial debris and ringed with fallen trees (Newbury and McCullough 1984). The only effective clearing done on SIL was on bedrock shorelines and on protected shorelines without permafrost.

The LWCNR study

There was no need in this study for quantitative prediction of resource impact as input to evaluating project alternatives because construction of the diversion was completed nearly simultaneously with the study (Fig. 1). The only quantitative prediction in the LWCNR study concerned the long-term (i.e. after shoreline stabilization) effects of river diversion on the productivity of the northern regions of SIL. Using nutrient loading theory available at the time (Vollenweider 1968), LWCNR (1975) forecasted a 30% decline in primary production and fish production in those regions because a large proportion of a crucial nutrient, phosphorus, supplied by the Churchill River would be diverted with the river. The scientists involved with the study recognized the lack of quantitative theory for relating flooding and erosion to water quality and biology, and it was this lack that prompted the continued study of SIL and research into processes linking flooded terrain with water quality and biological communities.

Deficiencies of the Paradigm

Success in prediction is the only valid criterion for choosing between scientific paradigms or theories. By this standard, the existing reservoir paradigm was unsatisfactory for SIL because many expectations from the paradigm were qualitatively incorrect or, if correct, they were not quantifiable. In the SIL reservoir, only the zoobenthos densities and nutrient release behaved, after flooding, as expected from the paradigm. The U-M study and LWCNR study made extensive modifications of the paradigm (Table 2). The fundamental modification of these studies was to recognize that the nature of the land to be flooded could determine the ecosystem response of the reservoir. In SIL, highly erodible permafrost-affected glacio-lacustrine and fine-grained tills were the predominant backshore material, and in regions with this material, high turbidity and high sedimentation rates occurred and modified the generally expected ecosystem response. But both studies and the paradigm overlooked a second important aspect, the dynamic heat balance of the lake reservoir that led to a general cooling of the lake after impoundment. Both of these aspects were unappreciated in the reservoir paradigm because the paradigm was based primarily on in-reservoir biological studies of relatively deep riverine impoundments. In such impoundments, changes in depth are generally large, leading to thermal stratification and a warming of the surface-mixed layer, and wind fetches are often limited so that erosion rates are relatively low even if erodible shoreline material is present. Reservoir studies that contributed to the paradigm emphasized in-reservoir biological responses to impoundment rather than transfer processes (heat input, erosion, leaching), so the paradigm did not include an appreciation of the effect of different kinds of thermal structure, vegetation, and soils on water quality and ecosystem productivity. Consequently, the experience and knowledge gained from existing reservoirs was not easily transferable to a new reservoir situation.

Successful use of the existing reservoir paradigm requires finding a well-studied, analogous reservoir that has a similar climatic regime, morphometry, terrain, extent of flooding, biological community, etc. In fact, choosing an analogue is difficult given the nonrandom distribution of critical factors. Proponents and opponents of various water development schemes can choose reservoirs that they feel match the proposed situation but that may have evolved quite differently after impoundment. A reservoir frequently mentioned by proponents as an analogue to SIL at the time of the preimpoundment assessments was Reindeer Lake in northern Saskatchewan. This naturally large lake was impounded in 1942. The latitude, increase in water level, surface area, operating regime, climate, bedrock geology, and preimpoundment fishery were all similar to those of SIL, but Reindeer Lake suffered only marginal effects on its fishery after impoundment (Atton 1975). The critical difference between SIL and Reindeer Lake was the distribution of erodible, fine-grained shorelines in their basins. In SIL, extensive bank erosion has altered the sedimentation regime and water quality of the lake. Similar deposits were extremely sparse at Reindeer Lake and erosion was minimal. Thus, even if agreement can be reached that only one characteristic of the proposed analogues is different, without a predictive model for the effect of that characteristic, antagonists can still disagree on what effect that one difference will have on the overall development of the new reservoir.

A New Paradigm

A new paradigm is required for future reservoir impact studies. The act of prediction assumes that the future, in some way, already exists in the present (Schumacher 1973) and that it only needs to be seen. For objective prediction making, the future must be observable by everyone and not just special "seers." For reservoirs, the observable facets before impoundment include reservoir surface area and geological characteristics of land to be flooded, proposed water depth, proposed operational regimes, meteorological conditions, river flows, biological communities, resource utilization, and resource valuation. Although quantitative prediction can only be generated for parameters that can be related to such observable quantities before impoundment (see Nielson 1967), the number of quantitative relationships established between preimpoundment parameters and postimpoundment water quality and biological responses remains small. The reservoir modeling attempts of Ostrofsky and Duthie (1978), deBroissia et al. (1981), and Grimard and Jones (1982) are examples of input–output models (Vollenweider 1975) that are specifically designed for new reservoirs and that relate water quality effects to measurable preimpoundment parameters. These models have focussed on reservoir primary productivity and its presumed dependence on phosphorus as they attempt to predict the internal loading of phosphorus from flooded terrain. These models require transfer coefficients, for the input of phosphorus per unit of flooded area, that are analogous to phosphorus export coefficients for watersheds (Dillon and Kirchner 1975). The validity of these reservoir models that predict internal loading of phosphorus remains largely undetermined. Grimard and Jones (1982) pointed out that there were insufficient data on phosphorus in any new reservoir to allow a definitive test of their model. Although these modeling efforts are still in their infancy, they represent the best avenue for improving our predictive capability concerning nutrients in new reservoirs.

The new paradigm requires transfer coefficients applicable to observable preimpoundment parameters to model the fluxes of mass and energy into a new reservoir. In SIL, internal loading of sediments from eroding shorelines had effects throughout the ecosystem and was a focus for our studies. The empirical relation between energy and erosion developed by Newbury and

McCullough (1984) allows eroded volumes to be predicted in new reservoirs based on preimpoundment shoreline mapping and meteorological records. The sedimentological fate of the eroded inorganic material, particularly its high retention near-shore, may also be predictable if the behavior of this material during transport, described by Hecky and McCullough (1984), can be shown to apply to other reservoirs. In concert, these studies demonstrate how suspended sediment concentrations might be predicted. In addition, Hecky (1984) found that light extinction in SIL was a linear function of suspended sediments, and Hecky and Guildford (1984) showed that algal populations in SIL became light-limited as the mean water column light intensity fell below $5\,mE \cdot m^{-2} \cdot min^{-1}$. The effects of these physical changes in the environment on the primary trophic level were qualitatively predicted by the LWCNR study, and they are now amenable to quantitative analysis.

Predictability of Higher Trophic Level Effects

The qualitative responses of the biological communities above the primary trophic level in SIL to reservoir formation were, in general, poorly predicted by the reservoir paradigm, U-M, and LWCNR. In fact, the number of completely unpredicted significant responses tended to increase with trophic level. The reservoir paradigm has the general expectation of an increase in production and biomass at all trophic levels; but in SIL, whole communities as well as constituent species responded differently to the new physical, chemical, and biological regime. For example, algal productivity was largely unchanged (Hecky and Guildford 1984), profundal zoobenthos abundance increased (Wiens and Rosenberg 1984), and zooplankton abundance decreased (Patalas and Salki 1984) after impoundment. Patalas and Salki (1984) offered several examples of densities of individual zooplankton species increasing or declining with impoundment and diversion. Perhaps the most difficult response of all to predict was that of the fish community, which can be sensitive, not only to changes in abundance of prey species and thereby, ecosystem energy flow, but also to direct physical effects on behavior, feeding success, and spawning habitat. The major biological responses in SIL unpredicted by both U-M and LWCNR studies were (1) the increase in zoobenthos densities, (2) the decrease in zooplankton biomass, (3) the increase in mercury concentrations in fish flesh, and (4) the decline in market quality of the whitefish catch. Does present ecological theory imply that these biological responses are likely to remain "unpredictable" (sensu Rigler 1982), or are there testable hypotheses to explain their occurrence in SIL?

The responses in the zoobenthos and zooplankton communities involved primarily increased or decreased abundances of the species extant before impoundment, i.e. there were no major shifts in the communities due to extinction or immigration. Rigler (1982) argued persuasively that long-term responses of individual species may well be "unpredictable" at present or forever because under current ecological and evolutionary theory species are expected to change through time, but he maintained that nonevolving state variables such as biomass of zooplankton or zoobenthos are subject to at least empirical prediction. The observed increase in profundal zoobenthos in SIL, although typical of many new reservoirs (Table 2, column 1), was not expected by the U-M or LWCNR studies. The U-M study had forecasted a decline in zoobenthic biomass because of increased sedimentation rates, while the LWCNR study had foregone a specific prediction because of uncertainty over the significance of increased sedimentation to the zoobenthos. Wiens and Rosenberg (1984) concluded that the pattern of responses in SIL was dependent on the balance between organic loading and inorganic sedimentation. These factors and the zoobenthos response are susceptible to experimental manipulation in mesocosms (Grice and Reeve 1982), as well as to confirmation in new reservoirs, and therefore their hypothesis is testable. The observed decline in zooplankton standing crops was not predicted but likely could have been, in principle, if the lowered mean water temperatures as well as postimpoundment turbidity levels had been quantitatively predicted (Patalas and Salki 1984). These hypotheses concerning the zooplankton decline are also testable. Improved predictability for zoobenthos and zooplankton abundance in new reservoirs requires (1) accurate forecasts of energy budgets and internal loading and concentrations of organic material and sediment and (2) quantification of the relationships between these environmental factors and these communities.

In the clear light of hindsight, it is possible to state that the problems that the fishery encountered in quality (Bodaly et al. 1984b) and in mercury contamination (Bodaly et al. 1984a) should have been considered by the preimpoundment assessment; but they were not considered because they were not part of the existing paradigm (Table 2). The rapid change in whitefish quality in SIL is explained by a shift in exploitation to lower quality stocks already in the lake at impoundment (Bodaly et al. 1984b). The presence of lower quality stocks in SIL was historically known, and certain regions of the lake were avoided to ensure that the catch was of high quality. The LWCNR study predicted stock movements and lower CUE, so it could have identified a potential quality problem if it had considered the historic fishing patterns on the lake. Such a prediction would not have been quantitative, but it would have been a useful warning. Dispersal of lake whitefish may explain the reduced CUE and quality change after flooding, but causes for the dispersal are still unknown. Reduced underwater light intensity and visibility were probably the most significant environmental changes in SIL (Hecky 1984), yet data on critical light levels for feeding, schooling, and other aspects of fish behavior are poorly known or unknown for the whitefish (Bodaly et al. 1984b). Better predictability for higher trophic levels, particularly the fish community, will require much more knowledge of species biology than is presently available.

The federal government inspection program for mercury in commercial fish shipments did not begin until 1970, but by the time of the LWCNR study in 1972 there was a growing appreciation that fisheries not associated with industrial pollution occasionally did have high mercury concentrations. In view of the severe effect mercury contamination can have on domestic consumption and commercial marketability (Bodaly et al. 1984a), concern about the effect of impoundment on the natural mercury cycle could have been identified. However, no more than a warning could have been given in the LWCNR (1975) study, as the first hypothesis associating fish mercury contamination and impoundments did not appear until later (Abernathy and Cumbie 1977). In fact, SIL was the first reservoir where an increase in mercury concentrations in fish after impoundment was *observed*, not just inferred. We are currently testing the hypothesis (Bodaly et al. 1984a) that flooding caused the increase in mercury concentrations in fish at SIL in mesocosm experiments using a radioisotope of inorganic mercury to follow its transformation to methylmercury and accumulation in fish.

Elevated mercury levels in predatory species and reduced whitefish marketability are quality issues for the commercial fishery, and as such, they are somewhat distinct from productivity issues. Both U-M and LWCNR studies considered productivity issues in great detail, but they underemphasized quality concerns. In terms of economic impact in the short term, the quality issues proved to be more damaging (Wagner 1984) than declines in productivity, although concerns for productivity may be well-founded in the long term. Both of these issues have led to monetary compensation programs for fisheries losses or claims for compensation, the need for which was not predicted by preimpoundment assessments. Institution of compensation has been retrospective and contentious, and lack of a comprehensive compensation scheme so far has penalized the SIL fishermen more than the developer (Wagner 1984). It would have been preferable to recognize before the event that compensation for fisheries losses would be necessary, and the principles of compensation agreed upon before the reservoir was developed. Future assessments must recognize the possibility of compensation by continuing preimpoundment baseline studies that emphasize resource production, quality, and utilization into the postdevelopment period to document change in the resource and its utilization.

Conclusion

Pre- and post-impoundment studies will allow testing of predictions and make the environmental impact assessment procedure scientific and self-improving (Rosenberg et al. 1981). Unfortunately, we concur with Beanlands and Duinker (1983, p. 23) that "Until now, environmental assessment has largely been a pre-development activity." Perhaps the major lesson from SIL is that this current approach to assessment is incomplete and unacceptable. The number of unexpected and poorly quantified impacts at SIL indicates that significant improvement remains to be made in the impact assessment of new reservoirs and river diversions. Prediction making must not become an end in itself because "... predictions are easily made; it is accuracy in a prediction which is difficult" (Neilson 1967, p. 166). Predevelopment predictions alone are not adequate to protect the habitat or the resource users. Such predictions should be recognized as planning aids that require testing in the postdevelopment period to establish their veracity and complete the environmental assessment process.

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SHORELINE EROSION AND RESTABILIZATION IN A PERMAFROST-AFFECTED IMPOUNDMENT

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In 1976, an 850 m^3/s river diversion was constructed through 300 km of permafrost-affected landscape in northern Manitoba. The diversion was accomplished by raising the level of a 1,977 km^2 riverine lake on the Churchill River (Southern Indian Lake) until the water spilled across a terrestrial drainage divide into a series of small valleys tributary to the Nelson River. Over 400 km^2 of permafrost-affected backshore area surrounding the lake were flooded. The mean annual temperature in the Southern Indian Lake region is -5°C. Three repeated phases of shoreline erosion in permafrost materials were observed; melting and undercutting of the backshore zone, massive faulting of the overhanging shoreline, and removal of the melting and slumping debris. At erosion monitoring sites in fine-grained frozen silts and clays, representative of over three-quarters of the postimpoundment shoreline, rates of retreat of up to 12 m/yr were measured. The index of erosion based on the wave energy impinging on the shoreline was 0.00035 m^2/tonne. After 5 years of erosion, restabilization of the shoreline has occurred only where bedrock has been encountered on the retreating backshore. Clearing of the forested backshore prior to flooding did not affect the erosion rates. The rapidly eroding shorelines have increased the suspended sediment concentration in Southern Indian Lake water and have triggered degradation of the commercial fishery.

INTRODUCTION

Southern Indian Lake lies in shallow irregular Precambrian bedrock depressions on the Churchill River in northern Manitoba (latitude 57°N, longitude 99°W) (Figure 1). The climate of the region is continental, with long cold winters and short, cool summers. Average mean monthly temperatures vary from -26.5°C in January to +16°C in July.

FIGURE 1 The 2,391 km^2 Southern Indian Lake reservoir is located on the Precambrian Shield in the discontinuous permafrost zone of central Canada. The lake lies on the northern boundary of the glacio-lacustrine deposits of Lake Agassiz.

The mean annual temperature is -5°C. One third of the annual precipitation of 430 mm occurs as snowfall during the average 200-day mid-October to late May snow cover period. The average accumulated depth of snow is 60 cm. Black spruce (Picea mariana (Mill.)B.S.P.), jackpine (Pinus banksiana Lamb.), and tamarack (Larix laricina (DuRoi)K. Koch) are the principal tree species of the boreal forest that covers the upland surrounding the lake. A layer of decaying mosses and lichens varying in thickness from a few cm to 5 m has accumulated since the final glacial retreat from the region 7,000 to 9,000 BP. During the deglaciation period, eskers, kames, and other proglacial landforms were deposited on the bedrock surface in the northern third of the region surrounding the lake. The deposits form a rolling upland with a local relief seldom exceeding 20 m. The uplands in the southern two-thirds of the basin lay within the area covered by glacial Lake Agassiz, a large proglacial lake that extended southward to the northern United States. Deposits of laminated silty clays up to 20 m thick occur throughout the region. In the southern region, the upland relief is greater and more abrupt, with exposed knolls and ridges of bedrock separated by poorly drained wetlands.

Permafrost is widespread in all terrain types surrounding the lake with the exception of the proglacial deposits in the northern region. The depth of the active layer varies from 0.5 to 2 m, depending upon local topography and the thickness of peat deposits. The temperature of the permafrost ranges from -0.2° to -0.8°C. Landforms associated with permafrost such as palsas, collapse scars, and peat plateaus occur frequently in the southern glacio-lacustrine region. Although the permafrost exceeds 10 m in depth in the upland areas (Brown 1973), it does not exist under the lake or under the narrow valleys of major tributaries.

In 1966, a program of hydroelectric development began in northern Manitoba to supply electrical energy to southern Manitoba and the central United States. A 927 km direct-current transmission line was constructed from southern Manitoba to hydroelectric dams at Kettle Rapids (1,272 MW capacity) and Long Spruce Rapids (980 MW capacity) on the lower Nelson River, 200 km southeast of Southern Indian Lake. Rather than extending the transmission line to potential dam sites on the Churchill River, a license was granted to Manitoba Hydro to divert 850 m^3/s (about 85%) of the Churchill River waters southwards across a drainage divide, through a long series of small channels and lake basins in the Rat and Burntwood River valleys tributary to the Nelson River above the power dams. In June 1976, a dam was completed across the natural outlet of Southern Indian Lake at Missi Falls, and the mean lake level was raised 3 m to divert the flow across the drainage divide at the southern end of the basin (Figure 2). The area of the lake was increased from 1,977 to 2,391 km^2. Flooding extended beyond the sub-lake thawed zone into the permafrost-affected upland.

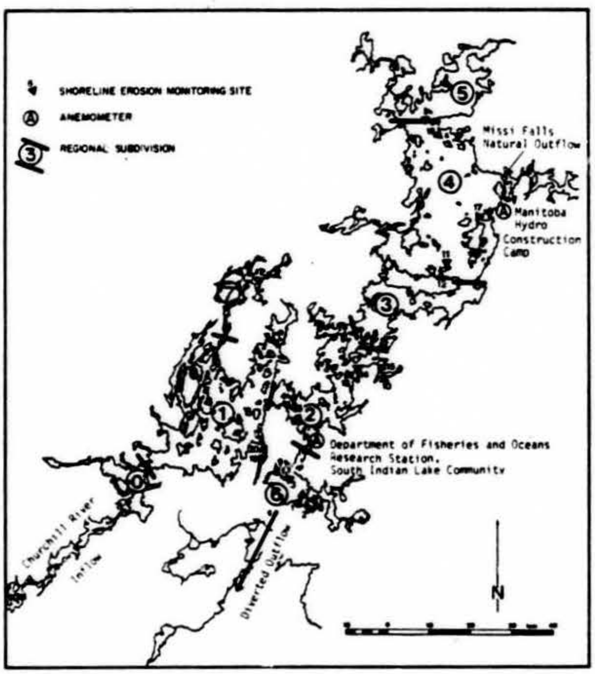

FIGURE 2 Southern Indian Lake is a series of bedrock-controlled, riverine basins on the Churchill River. The basins have been numbered to facilitate limnological studies. Erosion monitoring stations in granular deposits and fine-grained permafrost materials are located at 20 sites of varying exposure throughout the lake.

Preimpoundment studies of the effects of flooding on shoreline stability predicted qualitatively that erosion and solifluction of the shoreline materials would occur (Underwood-McLellan 1970, Lake Winnipeg Churchill Nelson Rivers Study Board 1975). Quantitative estimates of the rates and extent of shoreline erosion were not made, as analogous conditions had not occurred or had not been reported for such a large impoundments in permafrost.

METHODS

The study reported in this paper began in 1975, 1 year prior to the impoundment. Erosion monitoring sites were selected and surveyed at 20 locations having different exposures to wave conditions. The rates of erosion in the initial year of impoundment and the relative resistance to erosion of the permafrost materials have been previously reported (Newbury et al. 1978). The resistance to erosion was based on an index of erosion reported by Kachugin (1966) as a "washout coefficient, K_e," which expressed the volume of backshore material eroded per unit of wave energy dissipated on the shoreline. Kachugin's units of the washout coefficient of m^2/tonne are derived from the quotient of cubic meters of eroded materials per meter of shoreline length divided by the perpendicular component of the wave energy acting on the backshore expressed as tonne-meters per meter of shoreline length $(m^3/m)/(tonne\text{-}m/m)$. Eroded volumes have been surveyed annually since impoundment at erosion monitoring sites. Wave energies have been hindcast from wind speeds and directions recorded at 2 sites adjacent to the lake (Figure 2) using the modified Sverdrup-Munk procedure (U.S. Army Coastal Engineering Research Center 1966).

The lake was divided into eight sub-basins (Figure 2), for which the contribution of shoreline materials to the lake after impoundment was estimated. The washout coefficients determined from the monitoring sites were combined with the hindcast wave energies acting on 331 reaches of shoreline to determine the total erosion in each basin. The actively eroding shoreline reaches were mapped by aerial and boat reconnaissance of the whole lake in 1976 and 1978.

RESULTS AND DISCUSSION

Erosion Processes

Representative textures and ice contents of shoreline deposits at the monitoring sites are given in Table 1. An example of surveyed profiles showing annual erosion and nearshore deposition of lacustrine clay at a relatively high wave energy site is shown in Figure 3. The erosion of frozen fine-grained materials on shorelines surrounding the larger basins of the lake was observed to proceed in a repeated sequence of melting, slumping and removal phases. In the initial phase, melting occurs below and slightly above the water surface. In the second phase, the partially thawed materials flow out to form a silty-clay beach strewn with scattered frozen blocks. In some cases, caverns or melt niches are formed that are up to 1 m in height and extend up to 3 m into and under the frozen backshore materials (Figure 4). In this situation, the overlying cantilevered block splits away from the main land mass and falls onto the foreshore (Figure 5). In the final phase, wave erosion re-

TABLE 1 Texture and Ice Content of Mineral Materials at Shoreline Monitoring Sites. (Dashes indicate samples were not analysed.)

Site	Textural analysis of parent materials (%) Sand	Silt	Clay	Ice content (% of dry weight)
1	1	15	84	43
2	0	16	84	-
3	1	34	65	-
4	0	15	85	64
5	1	26	73	-
6	1	34	65	64
7	0	49	51	-
8	19	16	65	-
9	1	17	82	-
10	35	46	19	-
11	10	45	45	47
12	1	19	80	-
13	8	39	53	56
14[a]	2	36	62	92
15	98	0	2	-
16[a]	9	33	58	-
17[a]	4	37	59	43
18[b]	-	-	-	-
19	1	34	65	-
20	-	-	-	-

[a]Samples analyzed were of backshore lacustrine deposits. To date, erosion at these sites has been predominantly of former sandy beach materials.

[b]Predominantly fine to coarse sand with some silty beds.

moves the fallen debris and the warm lake water is again brought in contact with frozen backshore materials. In the silty-clay glacio-lacustrine materials, the form of the slumping and eroding shorelines does not change substantially as the backshore retreats inland. If bedrock or coarse granular materials are encountered at the eroding face, the inland movement at the water level ceases but erosion of the backshore continues until a wave-washed bedrock zone or a stable beach is established.

At non-permafrost sites in coarse granular deposits (Sites 15 and 18), erosion and deposition processes agreed with those reported by Bruun (1962) for similar materials.

Erosion Rates at Monitoring Sites

The total volume of annual erosion and the washout coefficient, K_e, at each of the 20 monitoring sites surrounding the lake are summarized for 4 years of impoundment in Table 2. Where bedrock was not encountered, the mean washout coefficient for the permafrost materials was generally one-half of that reported by Kachugin (1966) for similar materials in the unfrozen state. Although this suggests that permafrost conditions may retard erosion, no data are supplied with the Soviet Union observations, and the magnitude of the washout coefficient may have been based on a different method of determining an index of the wave energy.

There was a wider range of K_e values observed during the first year of impoundment than in the following 3 years. At sheltered sites exposed to low wave energies, several open water seasons were required to destroy the protective moss and root cover at the water's edge. At exposed sites, large volumes of peat were quickly removed from the flooded foreshore, producing high K_e values in the first year. Because of the variability of early years, general K_e values were calculated using 1978-1980 data only. Also excluded from the general K_e determinations were values at sites after bedrock had been encountered at the eroding face (Sites 6, 8, 10, and 11). Based on the 16 shoreline sites which extend over the range of materials and fetches encountered on Southern Indian Lake, the general K_e value found by linear regression was 0.00035 m^2/tonne ($r^2 = 0.71$, $n = 42$, Figure 6).

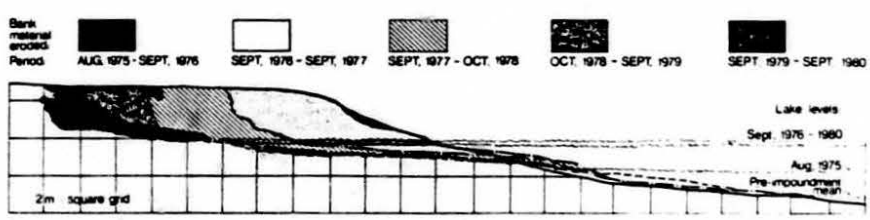

FIGURE 3 A typical consecutive annual sequence of eroding shoreline profiles at Site 1 (Figure 2). Slumped material from the initial 1975-76 period was removed in the following year.

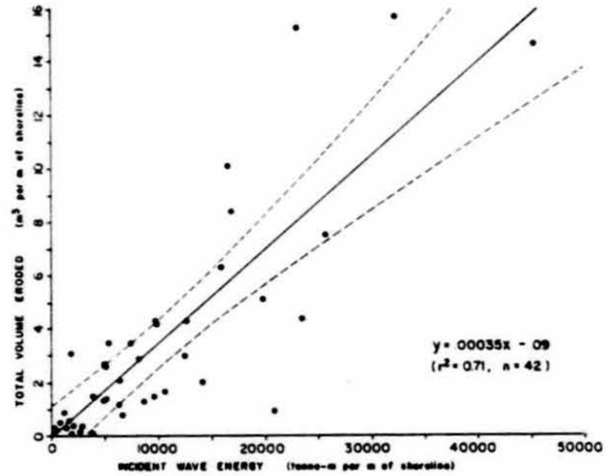

FIGURE 6 The linear relationship between eroded materials and wave energy for the 1978-1980 period on Southern Indian Lake is similar to that proposed by Kachugin (1966) for reservoirs in the Soviet Union. The scattered data points and wide confidence limits (95%) at high erosion and high energy sites suggest that the relationship may be curvilinear.

TABLE 3 Estimated Total Dry Weight[a] of Mineral Materials Eroded From the Shorelines of the Major Basins of Southern Indian Lake for the Period 1976-1978 (10^6 kg)[b]

Region	1976	1977	1978
0	122	177	166
1	528	672	615
2	238	311	290
3	478	668	608
4	1594	2099	1916
5	207	275	229
6	190	273	247
Whole Lake	3357	4475	4071

[a] The volume of dry mineral material eroded was calculated using an average water content of the perennially frozen silty-clays of 58% dry weight and assuming a bulk dry density of 2600 kg/m^3.

[b] After Hecky and McCullough (1983).

Sediment Contributed by Shoreline Erosion

The total dry weight of mineral materials eroded in the years 1976, 1977, and 1978 for each basin excluding the limited exposure shorelines of region 7 of the lake, is summarized in Table 3. Estimates were not extended beyond 1978, as that was the last year in which a reconnaissance survey was undertaken to determine the portions of the total shoreline in overburden and bedrock materials. Postimpoundment bank erosion dramatically increased the turbidity of the lake (Hecky and McCullough 1983). The long-term preimpoundment sediment input to the whole lake, estimated from Churchill River inflows, is 200 x 10^6 kg/yr. The sediment input from eroding shorelines following impoundment exceeds 4,000 x 10^6 kg/yr.

Time Required for Shoreline Restabilization

In the first 5 years of impoundment on Southern Indian Lake, restabilization in permafrost-affected fine-grained materials has occurred only on shorelines where bedrock underlying the backshore zone was exposed at the water's edge. Where bedrock was not encountered, there has been no change in the melting, slumping and eroding sequence of shoreline migration. The annual erosion indices at monitoring sites in fine-grained materials have shown no diminishing trend following the first year of impoundment (Table 2). The clearing of shorelines up to the impoundment level did not affect erosion rates.

Because sub-surface exploration of the bedrock topography surrounding the lake is prohibitively expensive, the time required for shoreline restabilization can be estimated only from the frequency of occurrence of bedrock at the monitoring sites scattered throughout the lake. Eighteen of the monitoring sites occur in fine-grained materials which are representative of over three quarters (2,841 km) of the postimpoundment shoreline. In the initial 4 years of impoundment, bedrock was encountered in the retreating backshore of the 4 most exposed monitoring sites. Assuming that the bedrock distribution is similar at less exposed sites, the rate of bedrock encounters should decrease, because more time will be required to remove the overburden. If the rate decays geometrically, 4/18 of the remaining eroding shoreline will strike bedrock every 4 years until the preimpoundment condition is restored.

Prior to impoundment, 76% of the shoreline was bedrock controlled. Following impoundment, bedrock was exposed on only 14% of the shoreline. Because the wave energy distribution on the lake and the bedrock topography were not changed by the impoundment, the same proportion of shoreline will ultimately be bedrock controlled. At the assumed geometric recovery rate of the sample shorelines, it would take 35 years to restore 90% of the fine-grained permafrost shorelines to their preimpoundment condition. Although this is an approximate estimate, it is likely that the instability of the Southern Indian Lake shoreline will not change for several decades. The discharge of bank sediment into the lake will continue to disrupt the fishery resources upon which the local residents are dependent (Bodaly et al. 1983a,b).

ACKNOWLEDGMENT

For over 20 years, the late R. J. E. Brown of the National Research Council of Canada undertook permafrost research in Manitoba and throughout northern Canada. In 1978, following the Third International Conference on Permafrost, he conducted a memorable tour of the Freshwater Institute project on Southern Indian Lake, which focused international attention on hazards of extensive flooding in permafrost terrain. The research staff of the Southern Indian Lake project wish to acknowledge his support, encouragement, and significant contribution to the understanding of permafrost phenomena.

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ABSTRACT

In the past, wildlife was largely ignored by utilities when selecting routes for energy transport facilities. This attitude has changed drastically during the last 20 years and the outcome has been to focus on species with high socioeconomic values in corridor selection studies. The objective of this overview paper is to discuss selected data-related aspects of the wildlife issue in routing energy corridors. The first section of the paper will discuss selected impacts created on wildlife by energy corridors and the role of corridors in shaping animal communities. The second part of the paper will address the wildlife data issue through the following topics: critical habitat, key species, loss of wildlife by small increment and experimental vs descriptive approaches. The third section of the paper will focus on the difficulties of gathering proper data to make predictions concerning wildlife and the role of management in route selection and right-of-way maintenance. Finally, conclusions will be formulated concerning the importance of wildlife issues in relation to other biophysical constraints in selecting routes for energy corridors.

INTRODUCTION

Energy and transportation corridors have affected wildlife since the early settlement of North-America and yet, historically wildlife issues have not played an important role in the planning of these facilities. Matthiessen (1964) gives a vivid account of the role that the construction of the first main transcontinental rail lines played in the demise of the buffalo (<u>Bison bison</u>). It is only during the last 20 years that increased attention has been paid to wildlife in corridor and route selection studies and the vast majority of these studies focused on species with high socioeconomic values such as ungulates, furbearers, waterfowl and endangered species.

The integration of wildlife issues in the planning process was difficult for two main reasons. The first was that the scientific literature was not conclusive on the benefits and disadvantages of rights-of-way (ROWs) on wildlife species or animal communities. The second was that wildlife was perceived very differently by various people, public agencies or disciplines and thus wildlife was often arbitrarily given a very different level of importance from one route selection study to another.

The objective of this overview paper is to discuss selected data-related aspects of the wildlife issue in the selection of energy corridors and ultimately the routing of the ROWs themselves. The discussion will address the following issues: 1) repercussions of ROWs on wildlife and animal communities, concepts, methodologies and approaches used to conduct the wildlife analysis in route selection, 2) the role of management, and finally 3) the importance of wildlife issues in the selection of corridors and routes for energy transportation.

REPERCUSSIONS AND CONTRAINTS

Rights-of-way and their associated structures can affect wild animals directly through collisions or indirectly through habitat modification. A priori one would expect that the greatest impacts on wildlife would result from large scale habitat modifications especially those which involve the clearing of ROWs through the forested environment. From a data point of view it is easier to collect and manipulate habitat data than it is animal data therefore it has been more common to analyze the wildlife issue from the habitat standpoint. Direct repercussions on animals on the other hand are more difficult to forecast but they have been shown to be numerous in retrospective studies and thus greater efforts are needed toward collecting the proper data to make valid predictions. The following are a selection of demonstrated impacts which show the difficulty of choosing the suitable data base for the impact assessment of ROWs on wildlife.

Animal aggregations

Dense aggregrations of animals or migrating populations can be extremely vulnerable to serious losses due to traffic or structures. Klein (1971) reported high mortality of reindeer (*Rangifer tarandus*) on roads and railroads, especially in winter. Allen and McCullough (1976) and Puglisi *et al* (1974) respectively reported heavy white-tailed deer (*Odocoileus virginianus*) mortality on Michigan and Pennsylvania highways, while Grenier (1974) estimated the highway moose (*Alces alces*) kill at 15-20 per cent of the adjacent population in a Quebec park. Davis (1940) reported on the highway-related mortality of medium-sized mammals in Texas. Massive roadkills of amphibians on wet warm spring or early fall evenings are difficult to document in the literature (van Gelder 1973; Moore 1954; Carpenter and Delzell 1951), but occasionally anecdotal information can be revealing. For instance Bider (unpub. data) has observed a kill of 500 leopard frogs (*Rana pipiens*) during a 3-hour migration on 1 km of farm road adjacent to a river hibernaculum in southern Quebec. In Switzerland, some mountain roads are temporarily closed during amphibian migration. Under certain conditions (e.g. partial darkness, fog), powerline structures such as towers, conductors and ground wires can inflict appreciable losses to bird populations (Anderson 1978; Blokpoel and Hatch 1976; Stout and Cornwell 1976; Boeker and Nickerson 1975; Scott *et al* 1972; Siegfried 1972; Cornwell and Hochbaum 1971; Ogilvie 1967; Stoddard and Norris 1967).

Rights-of-way structures are sometimes used to advantage by wildlife. Bridges, towers and poles are often used by wildlife for nesting sites (Bridges and McConnon 1981; Gilmer and Wiene 1977; Stahlecker and Griese 1979; Prevost *et al* 1978). Some highway interchange overpasses have been shown to harbour densities of woodchucks (*Marmota monax*) which were several fold higher than those found in adjacent agricultural areas (Doucet *et al* 1974). Railroad and highway ROWs can be conducive to waterfowl nesting (Voorhees and Cassel 1980; Oetting and Cassel 1971; Page and Cassell 1971).

Animal communities

Changes in the structure of animal communities following the development or maintenance of ROWs have been discussed in some studies. Doucet and Bider (1982) showed that most forest species except amphibians reduced their activity in a newly developed ROW and Bramwell and Bider (1981) reported the same phenomenon following a defoliation experiment in a ROW. As the early stages of vegetation develop in powerline ROWs, pioneer small mammal communities develop (Adams and Geis 1981; Schreiber *et al* 1976). As the brush community develops, bird species richness increases in ROWs (Bramble and Byrnes 1984; Chasko and Gates 1982; Meyers and Provost 1981).

The increased productivity and higher availability of preys in ROWs could result in greater predation intensity. Ladino (1980) reported higher activity for mammalian and reptilian predators in powerline ROWs. Several studies have reported greater animal activity at the forest edge (Wegner and Merrian 1979; Doucet 1975; Bider 1968). ROWs constitute long double ecotones and Gates and Lyzel (1978) have shown that this concentrates nests, producing an ecological trap where eggs and youngs are extremely vulnerable to predators.

Complete isolation of populations by the implantation of a ROW has never been demonstrated. Doucet *et al* (1981), Lamothe and Dupuy (1982) and Willey and Marion (1980) have all reported deer crossing powerline ROWs in winter. Doucet and Brown (1983), Adams and Geis (1981) Schreiber and Graves (1977) and Schrieber *et al* (1976) observed small mammals crossing ROWs in winter and summer. However, Joyal *et al* (1983) and Doucet *et al* (1981) have respectively shown that under given conditions, moose and deer crossings were reduced in ROWs during winter. Oxley *et al* (1974) have shown that white-footed mice (*Peromyscus leucopus*) and chipmunks (*Tamias striatus*) failed to cross a 90 m wide roadway. Finally, Doucet and Brown (1983) found marked differences in hare (*Lepus americanus*) activity, during a population peak, in the adjacent woods on each side of a 30 m wide ROW.

Dispersal

Dispersal of plants and animals have been facilitated by ROWs. Huey (1941) was one of the first to report such a range extension for pocket gophers (*Thomomys*) in Arizona. Getz *et al* (1978) showed that field voles (*Microtus pennsylvanicus*) used roadways as dispersal routes. The presence of field voles in cleared ROWs in forested areas and the presence of grassland bird species (Chasko and Gates, 1983) indicate that animals have the potential to disperse in ROWs. This potential for dispersal brings up two important points. One, the biogeographical concept of saltatorial dispersal from one habitat patch to another could well be a phenomenon taking place in a cleared ROW. The second point was discussed by Schrieber *et al* (1976) and is related to the spread of diseases by animals such as rodents expanding their range through dispersal in established transport and energy ROWs.

Indirect impacts

The last group of impacts are those that are long termed or resulting indirectly from the implantation of a ROW. The Newfoundland railroad was completed at the turn of the century and it ran through the migration route of the main caribou herd. Hunters quickly adjusted to hunt the caribou as they crossed the ROW and by 1925 this scheme had largely contributed to the demise of the 40 000 caribou herd (Bergerud 1983). Deer and moose declines in Quebec in the 60's have been correlated to excessive hunting in areas where access was facilitated by the development of new highways (Bider and Pimlott 1973). The forecasting of long-term and/or indirectly induced changes in wildlife populations is an issue which deserves serious considerations in route and corridor planning.

CONCEPTS, CRITERIA AND APPROACHES

Critical habitat

Wildlife species need a habitat mosaic which enables them to feed, breed, raise young and rest. It is recognized however that some habitat components are more important than others in time and space. This has facilitated the adoption of the concept of critical habitat in route selection studies. Critical habitat remains an ill-defined concept; it has been variously interpreted as habitat which: 1) harbours a high diversity of life forms (e.g. marshlands), 2) fulfills a specific seasonal need for a given species (e.g. winter yards for white-tailed deer), 3) provides a fragile and/or limited refuge to a rare or endangered species, and 4) fulfills some intermediary function in the biology of a given species (e.g. migration routes of caribou).

Despite some shortcomings, the concept of critical habitat remains valid but it needs clarification and refinement. The central problem is the fact that it is difficult to recognize critical habitat without proper information. Habitat evaluation and mapped results have been lagging at scales appropriate for planners. The maps produced by ARDA for example were often inadequate planning tools because of poor resolution, lack of coverage or incomplete wildlife information. This was demonstrated during the recent routing of high voltage transmission lines in Quebec where several existing deer yards failed to appear on the ARDA maps for ungulate potential. Yet, in some areas, the ARDA analysis remains the best overall mapped information available. The next step is to seek information from the regional level; a slow process under the best circumstances, especially if it requires additional field surveys. There are efforts underway which should improve this situation. For instance the Canadian Committee on Ecological Land Classification has a wildlife working group which is developing appropriate methodologies and format for habitat mapping (Taylor 1979). Hounsell and Risley (1982) have developed a habitat classification system to predict the effects of powerline ROWs on wildlife. In Quebec, the Ministry of Leisure, Fish and Game has been concerned about wildlife habitat and it is striving to give legal status and protection to defined and identified critical habitats (Sarrazin 1983). Results of these efforts should improve the planning process in relation to wildlife habitats because planners will have access to organized information and guidelines (e.g. maps, laws) concerning valued wildlife habitat. However it appears that planners will be left with certain decisions concerning priority critical habitats in conflicting situations.

Key species

In the majority of planning studies, the wildlife analysis is often limited to a few so-called socioeconomically important or key species. This approach raises two important concerns. The first one is that key species means different things to different people. For instance it could be an endangered species or a species that plays a dominant role in the evolution of the structure of the animal community. There is no reason why wolves, beavers, robins or bullfrogs cannot qualify under given circumstances. But the question remains as to who and what criteria should decide which are the key species in a given route selection study. The second concern is that the soundness of determining a priori that some species are more important than others in the ecosystem is a highly questionable practice, whatever the scale of values used. Elton (1927) considered arctic copepods as key industry animals and Pianka (1983) and Paine (1966) defined keystone predators. Although these studies suggested that some species are dominant in shaping the structure of animal communities, in general few ecological studies have supported the concept that some organisms are more important than others in ecosystems. Indeed the holistic approach embraced by most ecologists makes such distinctions of importance highly dubious.

Losses by small increment

By definitions, ROWs are narrow strips and this has promoted the belief that site-specific impacts were of little consequence (with a few exceptions: e.g. pipeline spill risks and caribou migration disruption in the north). Thus the planning process often considers that only a small fraction of a given habitat (e.g. marsh, roost, deer yard) is lost when bisected by a ROW and that most animals can relocate outside the disturbed zone. Although this analysis can withstand regulative and public scrutiny in successive projects, small incremental losses could in the long run jeopardize the resource as a whole through direct or indirect repercussions such as those discussed earlier.

The same concept can be applied to wilderness. Wilderness has several definitions; it can represent vast areas of untouched land or innaccessible areas sometimes in a park or reserve and some wilderness areas are protected by legal status. We often attach the attribute pristine to wilderness and an acceptable definition of pristine is that which is unspoiled by modern tendencies. If we subscribe to the concept that some wilderness should remain free of large scale interventions, it follows that the routing of a ROW through such an area under the assumption that only a small fraction of the unit is lost should be opposed for two reasons. First, it destroys the very concept of a pristine area, if not wilderness itself. The second reason is related to losses by small increment. If a ROW can be routed through a wilderness area under the assumption that only a small fraction of habitat is touched, the same assumption can be carried out to successive route proposals in the same or different wilderness area and thus all wilderness areas could be encroached and jeopardized.

Experimental vs descriptive approach

It is unrealistic to attempt to develop a complete understanding of all ecosystems or animal communities in order to route an energy transport linear facility through the rural or forested environment. However, in order to consider the wildlife issue, it is imperative to have at least a preliminary understanding of the ecological relationships which various wildlife species maintain with each other and with their habitat. To predict the repercussions of ROWs on wildlife a choice usually has to be made between a descriptive approach and an experimental approach. The descriptive approach is often speculative and fails to yield the data required to determine the repercussions concerning reproduction, feeding, behaviour, predation, dispersal and ultimate fate of wildlife populations under consideration. On the other hand, the use of experimentation and scientific methods has been slow and at times inconclusive in producing information helpful to planners. Let us examine this weakness in relation to white-tailed deer, a "hot species" in many ROW planning studies in the northeast. The wildlife literature (Halls 1978; Dasmann 1971 and Hosley 1956) suggests the creation of forest clearings to produce pioneer vegetation and provide winter browse for deer. Bramble and Byrnes (1974, 1982) observed increases in deer browse in powerline ROWs in Pennsylvania. These results, although useful in route selection in deer range at large, become of dubious value in northern deer yards where cover is so critical. A five year study (Doucet et al 1981) in such a yard showed that deer were less active in a 30 m wide powerline ROW than in the adjacent forest in the winter and the authors suggested that deer yards should be avoided by energy transportation ROWs. However, a recent study (Doucet and Brown 1983) conducted in the same yard showed that deer spent considerable time browsing in the ROW during winter. Thus after 10 years of research the results are still inconclusive concerning the trade-off between the loss of cover and the gain in food production in relation to ROWs in northern deer yards. The magnitude of this trade-off is also likely to change for each yard depending upon winter severity and annual population levels within a yard. This one-species scenario shows that there are cases where conclusions based on research data are of limited assistance to the corridor and route selection process.

Several authors (Beanlands and Duinker 1983; Romesburg 1981; Greer 1979; and Ward 1978) have deplored the limited use of scientific methods in environmental studies. These authors, however, pointed out the difficulties of conducting control-treatment studies during the preliminary impact assessments of a proposed project. Beanlands and Duinker (1983) have suggested the use of an ecological perspective to the biological components of impact studies. This approach would need to use some unifying ecological processes such as eutrophication, or nutrient cycling as a negotiable currency. In general, studies on whole animal communities are difficult to conduct because of multi-technique sampling problems. Studies by Bramwell and Bider (1981), Ladino (1980) and Doucet and Bider (1982) using sand transects as a technique and animal activity as a currency have shown the short-term effects of cleared powerline ROWs on terrestrial animal communities but the long term effects remain largely unknown.

We firmly believe that experimental research on representative problems would evaluate the legitimacy of several concerns related to wildlife and ROWs and the outcome would increase the efficiency of the planning process. Although the impractibility of conducting many large scale treatment-control studies is recognized, it seems that a potential solution to this difficulty is to implement a few representative long-term studies and to establish monitoring for a number of typical projects (Beanlands and Duinker 1983). This approach would eventually generate a data base from which to make predictions and suggest mitigations.

Another approach is to create experimental reaches of energy and transportation ROWs for the specific purpose of investigating repercussions on wildlife. These special sections could be submitted to treatment-control experiments and could contribute to ecological science and improve the accuracy of predictions for corridor planning.

Results of long-term studies may turn out to be the necessary information for efficient planning in relation to wildlife and ROWs. How long should those studies be is a difficult question to answer but in order to obtain adequate data to make predictions possible concerning wildlife populations and related processes, study specifications should consider for instance the cyclical nature of several wildlife populations and ROW maintenance cycles.

DATA REQUIREMENTS

There are several types of data required in order to integrate wildlife issues in the route selection process. Two specific types of data are: 1) those related to impacts or constraints associated to routing and ROW implantation, and 2) those data required to determine the effects of ROW maintenance activities on wildlife (e.g. timing of work, types of machinery, labour force, phytocides, fire, etc.). The approach has been that baseline data will provide sufficient ecological understanding to permit the formulation of predictions. Although this sounds logical, the problem lies in the fact that the expression baseline data is much too vague in time, space and scope. The interpretation of this concept by environmental specialists in a permit procurement system has been to collect a minimum of data to satisfy the guidelines. Many of these data collections were of limited use in successive route selection studies mainly because very few of these studies included systematic long-term monitoring (Beanlands and Duinker 1983). In addition, very few of these analyses concerned themselves with indirect impacts. Consequently there is a paucity of good representative studies which could serve as backbone in new route selection studies. The post-construction monitoring often presents a non-expansive and practical way of producing before and after type data (Beanlands and Duinker 1983). Certainly this approach should provide pertinent wildlife data and conclusions applicable to future routings of linear energy facilities.

As stated earlier, an improvement in approach would be attained through rigorous experimental studies on specific wildlife problems related to ROW implantation or maintenance. Such studies, using scientific methods, would consider topics like habitat management, edge effect, plant and animal communities, animal activity, predation, competition, dispersal and safety hazards, in order to develop the necessary data to make predictions concerning impacts of ROWs on wildlife. Efforts in that direction have produced useful preliminary results concerning animal activity (Chasko and Gates 1982; Bramwell and Bider 1981; Doucet et al 1981, Ladino 1980), but research must be continued to determine the true impact mosaic of ROWs on wildlife. Finally, at some point in time, research will have to address the complex problem of indirect impacts.

MANAGEMENT

Habitat management

Most wildlife management ventures to date in energy transportation ROWs were directed at habitat modifications. It is quite amazing how management efforts get the go ahead despite flagrant lack of data concerning the wildlife issue. The routing of ROWs creates a spatio-temporal trade-off where some species benefit while others are stressed and unless we understand the magnitude of this trade-off, it remains extremely difficult to make enlighted management decisions concerning wildlife. There is considerable general and somewhat technical information on the management of ROWs for fish and wildlife (e.g. Leedy and Adams 1982; Galvin et al 1979, and Meyers and Provost 1981). Although these reports and several others are more concerned with habitat "grooming" than habitat management they indicate that at times, and through positive management decisions, a certain compatibility can be achieved between wildlife requisites and the routing of ROWs. Certainly, to date, there is available knowledge on some specific wildlife concerns which can be integrated in the planning process in order to address some specific wildlife issues and reduce the impact and sometimes possibly improve the fate of some wildlife species. In the northeast for example, few species arouse public concerns as much as white-tailed deer. These ungulates congregate in traditional winter yards which represent critical habitat for the survival of the deer populations. Under these circumstances, one would consider yards to be major constraints to corridor routing. However a deer yard can be broken into two major components which are cover and food: and while the clearing of coniferous cover to route an energy facility should not be considered, deciduous stands on the other hand can present a viable alternative under specific circumstances. Textbooks on yard management (e.g. Dasmann 1971) suggest to rejuvenate climax deciduous stands to produce deer browse, thus the routing of energy transportation ROWs in such mature deciduous stands within a deer yard appears feasible. Once this concept is accepted it becomes possible to formulate objectively and integrate the details of a management plan into a project (e.g. clearing-by-small-blocks rotation). A successful case has been reported (Lamothe and Dupuy 1982) in Quebec, where a twin 735 kV line was routed through an active deer yard. By locating the elevated towers in clearings practiced in deciduous and mixed stands and raising the conductors to spare coniferous stands, the loss of cover was minimized and deer were provided with quantities of browse from the slash of the original clearing which was carried out in the winter and browse produced by the new growth in successive years. Overall, it is possible that the project may have benefited deer. Unfortunately the long term monitoring of this project was not geared to determine the cover-browse trade-off and it remains difficult to determine the effect of the maintenance schedule on browse production and availability.

It is most important to emphasize that we can only proceed on a case by case basis and that all deer yards are not systematically suited for the routing of an energy transportation ROW. In addition, the routing of a ROW in a deer yard eliminates the wilderness characteristics which are sometimes attached to such habitats. Finally on a comparative scale, deer are certainly much better off to have a linear energy transport facility encroach their yards than a housing development. This kind of choice is not usually left to wildlifers and/or planners alone.

Management: a panacea

Habitat management should not be considered the cure-all for the various problems that ROWs present for wildlife populations. The management of ROWs for wildlife is a complex issue (Meyers and Provost 1981). The be effective, habitat management must be carried according to a set of objectives, otherwise the effects could amount to well- intentioned habitat "grooming". From a wildlife point of view, linear energy transportation facilities present three different sets of problems which are those associated with: 1) planning, 2) construction and, 3) operation and maintenance. The planning phase not only determines the route but it is also responsible for the formulation of guidelines and terms of references for the construction and operation phases. Thus communication is essential between these phases if management is to be successful. Since planning, construction and operation are carried out by different agencies or divisions within an agency, communication breakdowns start during the construction phase and often become complete during the operation and maintenance phase.

Certainly decisions made concerning the routing and construction aspects can be called management decision but the decisions concerning the operation and maintenance of a ROW, in practice, are generally left to the proponent. The planning phase should consider the maintenance aspects of a ROW very seriously in relation to wildlife during a route selection study. This approach requires three important requisites. One, the terms of references for ROW habitat maintenance must remain extremely simple. Secondly, follow-up or monitoring programs must be out in place; such programs could be integrated in the overall ROW inspection program. Finally communications must be assured between the master plan responsibility levels and the field maintenance levels. Breakdowns at this latter stage are as easy to find as bulldozer operators.

ROLE OF WILDLIFE DATA IN CORRIDOR SELECTION

The route selection process must consider wildlife within an array of other biophysical constraints along with social values, costs and technical constraints. The importance of wildlife in such multidisciplinary approaches has been characterized by a roller coaster approach where wildlife issues played a very different role in various studies. Perhaps one reason for this is the willingness of the public to abandon their rights or interest in wildlife (Schoenfeld and Hendee 1978). When there is a conflict with other issues (e.g. forestry, agriculture) in a route selection study, often one can expect wildlife issues to play a secondary role under a lack of sustained public interest. On the other hand, several public groups are often most eager to add weight to other issues such as agriculture, forestry and recreation. Perhaps one reason for these issues gaining momentum during a study is the fact that they lend themselves to dependable predictions and forecast. For example, it is easier to determine that 200 ha versus 600 ha of agricultural land will be lost depending on the outcome of a route selection study where two alternatives are considered. Wildlife issues are usually not as clearly presented and perhaps the lack of adequate data, at times, can contribute to the ultimate demise of wildlife concerns in the corridor and route selection process. There are very few well informed voices which speak for wildlife in route selection studies and unless a species or habitat is legally protected, the level of constraint of the wildlife issue is greatly reduced. If the importance of wildife issues is to be established from social values and national heritage points of view, this can only be achieved through better knowledge of the impacts of ROWs on wildlife communities.

CONCLUSION

Wildlife populations can be affected in many ways by linear energy transport facilities. These impacts can be short- or long-termed, direct or indirect, trivial or of great significance. An essential component of the planning and impact assessment processes is to make predictions. Unless you have a definite idea of: 1) which populations are present and at what time of the year they are most vulnerable to habitat modification, and 2) which ecological processes (e.g. predation) will be affected by a ROW, it becomes extremely difficult to make accurate predictions concerning the fate of these populations, their behaviour, dispersal and community structure, especially in a long-term perspective. The role of energy corridors in shaping animal communities represents a recent interest in the scientific community. It is obvious that some species will benefit while others will be disadvantaged and the trade-off presented by energy ROWs remains difficult to predict from the evidence available. Scientific research in this field is badly needed to provide the proper data to make accurate predictions concerning wildlife and route selection.

Research concerning habitat management in established ROWs could reduce constraints from the route selection process and at the same time improve the fate of wildlife populations occupying habitats which become bisected by energy transport linear facilities. The failure to adopt this course of action will result in a state of stagnation where each route selection will produce various quantities of descriptive wildlife data which will be of limited use to the route selection process and to management efforts.

The role of wildlife issues in corridor selection will be enhanced and simplified through more intensive research in that specific field of wildlife biology. Better overall evaluation and mapping of wildlife habitat (e.g. through improved resolution in remote sensing) will enable planners to recognize problems early in the corridor and route selection process. The maturing of the above considerations can only take place if planners and wildlife specialists maintain an open dialogue to ensure a realistic and favorable inclusion of wildlife issues in the route selection process. In the long run, this will make the route selection process easier, more conflict free and more efficient.

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ICE CONTROL MEASURES

ON THE

ST. LAWRENCE RIVER

Presented at:

EASTERN SNOW CONFERENCE

OSWEGO, N.Y., FEBRUARY 1972

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ICE CONTROL MEASURES ON THE ST. LAWRENCE RIVER

Introduction

Within the 160 mile reach of the lower St. Lawrence River between Montreal and Quebec City (see Fig. 1) flooding of low lying areas and damage to shore property used to always be hazards associated with winter. The primary reason for such flooding lay in the fact that the ice still had a firm grip on the river at the time of the spring freshet. Flooding also occurred during the early and mid-winter months due to ice jams caused by shoving and telescoping of the ice cover before final consolidation. Marks on buildings along the Montreal waterfront attest to the heights to which the most disastrous of these floods reached; the worst being that of 1886 when Notre Dame Street virtually became a river. The municipalities around and downstream shared with Montreal the distress associated with the annual spring run-off and ice break-up in the St. Lawrence. It is recorded, for example, that 50 people lost their lives during the spring flood of 1865, and that in the Sorel-Berthierville region miles of low-lying areas were innundated.

From earliest times, a particularly vexatious problem has been the formation of an ice bridge at Cap Rouge, some five miles west of Quebec City. Here, at the site of the impressive Quebec Bridge, the St. Lawrence is about 175 feet deep, but its width is reduced to less than half a mile. Under conditions of extremely cold weather and a heavy run of drift and sheet ice, this narrow passage was often spanned by a bridge of ice during slack water at high tide. If this bridge was not dislodged by the following ebb tide, a solid barrier would form and generally remain in place until spring. Not only did this lead to considerable flooding upstream, but when the bridge finally let go, the effects in Quebec Harbour were often disastrous. The break-up of the Cap Rouge ice bridge on May 9, 1874 is reported to have resulted in waterfront damage, including sunken ships, to the extent of one million dollars.

The first organized attempt at ice control was made in 1906 when two federal government icebreakers made their presence felt on the St. Lawrence. In the early spring of that year the brand new Lady Grey* and Montcalm, working together, successfully broke up the ice bridge at Cap Rouge before flood dangers developed. Although this was a modest beginning, it was literally a "breakthrough". The Lady Grey is shown at work in Fig. 2.

It soon became evident that the formation of an ice bridge could be prevented by continual icebreaker patrol throughout the winter, and with this barrier removed, it was a relatively easy matter to maintain an open channel as far upstream as Trois-Rivières. With the addition to the ice-breaker fleet of the Saurel in 1929 and the N.B. McLean a year later, it was generally possible to open a channel to Montreal in time to provide an escape route for the heavy run of ice and freshet waters in the spring. There still remained, however, the problem of early and mid-winter floods. It was not until the acquisition of more powerful and modern icebreakers in the early 1950's and the inauguration of continuous ice-breaking operations through the winter that the situation was controlled and the incidence of flooding drastically reduced. Due to this flood prevention program, improved icebreaking techniques, and recently installed control works, the disasters of former years are now all but forgotten. Although we must accept the fact that we are as yet unable to prevent the formation of ice jams, they are now removed by the icebreakers long before there is danger of flooding. As a matter of interest, the maximum level at Montreal last winter occurred on February 5, when an ice jam at Montreal East raised the water level to about 13 feet above normal for a short period, as compared with levels 25 feet above normal which frequently occurred in the past.

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* The Lady Grey sank dramatically after a collision while assisting the Quebec ferry in 1955.

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Problem Areas

As already mentioned, a major problem area is the Quebec Narrows section at Cap Rouge. The tidal range at Quebec is in the order of twenty feet and effective icebreaking operations can only be carried out on the ebb tide which flushes the broken ice downstream. These operations may have to be curtailed or even suspended under conditions of poor visibility. During critical periods of winter, at least one icebreaker is on standby at Quebec to prevent the formation of jams. The winter of 1967-68 is illustrative of the very serious conditions which can arise in this section. During the extremely cold weather in the first two weeks of January 1968, the ice bridged over in the Quebec Narrows and with continuing extreme temperatures a very severe ice jam quickly formed upriver as far as Trois-Rivières. It took the combined efforts of nine Coast Guard icebreakers (including the three "heavies") about three weeks to finally clear the channel to Montreal. The conditions which faced the icebreakers are well illustrated in Fig. 3.

Between Quebec and Trois-Rivières tidal action and icebreaker patrol generally keep the channel open throughout the winter. From Trois-Rivières to Montreal, however, the situation is much more complex, and in this reach there are two key areas for controlling the ice problem - the Lake St. Peter-Lanoraie Section and the LaPrairie Basin.

Ice Cover Formation - Trois-Rivières to Montreal

The ice cover in the St. Lawrence River from Trois-Rivières to the foot of the Lachine Rapids develops in much the same way every year. With the advent of cold weather, water temperature gradually falls to the freezing point, drift ice begins to appear over the surface, and sheet ice forms in the bays and in areas of slack water. This newly formed ice tends to break off under the action of wind and waves and together with a mixture of "frazil" and slush, drifts downstream into Lake St. Peter where the average current velocity is less than 1.5 feet per second, and at times of strong northeasterly winds there may be no surface movement at all. This condition allows border ice to form and bridge across the narrow outlet of the lake. The mass of drifting ice accumulates against the bridge and a cover soon forms. This usually happens toward the end of December. Thereafter, the ice cover packs upstream to Montreal at a rate dependent on the meteorological conditions and in a manner governed by the hydraulic characteristics of the river.

The mechanics of ice cover growth by packing are not yet fully understood. In the past, the process was based on simple current velocity criteria, with the limiting velocity for advance of the cover by packing being taken as 2.25 feet per second. It is now generally accepted that progression of an ice cover is related to the Froude number, the value of which determines whether ice floes will pack against the leading edge of the advancing cover, or be drawn under it. The actual value of this number for a particular section is probably dependent on various factors, including the hydraulic conditions and the characteristics of the ice at the section. From observations made by Kivisild[1], it appears that the average critical Froude number is about:

$$F = V/(gD)^{\frac{1}{2}} = 0.08$$

where V = velocity of the current at the section
D = mean depth of water at the section
g = acceleration due to gravity

According to Kivisild, when the Froude number is less than 0.08, ice accumulates against the edge of the cover and the pack grows upstream, becoming more massive as the Froude number approaches the critical value. At higher Froude numbers ice is drawn under the cover and is deposited on the underside to form a "hanging dam" at sections where the shear stress against the cover is below a certain value. This process

...5

creates a backwater effect upstream and makes hydraulic conditions favourable for ice to accumulate against the cover, thus allowing the pack to continue its growth upstream. This is the manner in which the ice cover advances from Lake St. Peter to Montreal. A typical ice cover formation is illustrated in Figs. 4 and 5. If the weather remains very cold the ice cover will be relatively strong. A period of mild weather, however, can weaken the cover sufficiently that it cannot withstand the thrust on it. At this point the cover may buckle or telescope and compress to form a heavy ice jam, with subsequent sharp increases in upstream water levels. Serious jamming frequently occurs in the narrows at the head of Lake St. Peter, in the Lanoraie section and in Montreal Harbour.

A key factor governing the ice problem at Montreal and downstream is the continuous generation of ice throughout the winter in the seven mile open water reach of the Lachine Rapids and lower end of Lake St. Louis. The vast quantities of ice generated in this section continue to flow downstream until the increase in water levels in upper Montreal Harbour caused by the advancing ice cover is sufficient to reduce current velocities and permit an ice cover to form in LaPrairie Basin up to the foot of the rapids. Thereafter, with continuing cold weather, the cover in the basin consolidates and the bulk of the ice generated in the rapids area is stored under it. This marks the end of a critical stage in the winter's operations and normally there is little further trouble. A typical ice cover in Montreal Harbour during the fifties is shown in Fig. 6.

In Lake St. Peter a further complication arises after the channel has been opened by the icebreakers in early winter. The lake is some 8 miles wide and 20 miles long with an average depth of about 10 feet. The Ship Channel, which passes through the middle of the lake, is 800 feet wide and dredged to a depth of 35 feet. The problem here is that from time to time large pieces of the cover break off through the action of wind and the waves of passing ships. These large masses move into the shipping lane and effectively block it.

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One of the most troublesome areas is the northeast section of the lake, and an example of what can happen is illustrated in Fig. 7.

Icebreaking and Winter Navigation

Until a few years ago icebreaking between Montreal and Quebec City was traditionally carried out only for flood control, and the maintenance of an ice-free track for ferry crossings at Quebec, Trois-Rivières and Sorel. Officially no direct assistance was given to ships navigating in this reach of the river except in emergencies. During the last decade, however, there has been a growing trend of ships taking advantage of these icebreaking operations to reach Montreal in the winter months. The closed season, i.e. the time between last departures and first arrivals at Montreal has been progressively shortened from about five months at the beginning of the century until today the port is virtually open to year-round navigation. Most of the ships sailing to Montreal in winter have specially reinforced hulls to combat ice conditions in the Gulf and the St. Lawrence River. Fig. 8 shows the trend and growth in the number of vessels sailing to Montreal during the winter months, January through March.

Although the major role of the icebreakers in this reach of the St. Lawrence is still flood control, they have now taken on the additional responsibility of maintaining an open channel in support of the developing winter traffic through the area. The annual cost of icebreaking between Quebec and Montreal is at present about $750,000 with the following ships of the Coast Guard fleet usually assigned to these operations:

N.M. Rogers	-	13,000 Horsepower
N.B. McLean	-	6,500 Horsepower
Montcalm	-	4,000 Horsepower
Ernest Lapointe	-	2,000 Horsepower

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An important factor in icebreaking operations is the width of channel opened. The aim is to open a lane wide enough to permit safe navigation and adequate ice evacuation. This policy leads to less ice being produced in the system, and the slightly higher velocities in the restricted channel favour better ice evacuation. Fig. 9 shows an historic representation of the ice cover formation and icebreaking operations between Quebec and Montreal for the winter of 1967-68. The influence of these operations on water levels in Montreal Harbour is further illustrated in Fig. 10.

At times bad weather can reduce visibility and seriously hinder the work of maintaining an open channel. The Ministry of Transport, recognizing the need to get the most effective use of its icebreaking fleet, is actively evaluating and experimenting with a number of accurate vessel location systems which would allow the icebreakers to work under conditions of extremely poor visibility. One Radar Positioning System now under extensive testing on the Norman McLeod Rogers has indicated that an accuracy of about 1 meter can be obtained in vessel location. When this equipment has been fully proved and becomes operational, downtime due to poor visibility should be practically zero.

Ice Control Structures

To enhance its ice control program to meet the dual challenge of reducing the danger of flooding and of assisting winter navigation, the Ministry has also installed a number of additional features in this reach of the river.

The construction of EXPO '67 involved the narrowing of St. Lawrence River channels in the Montreal area, resulting in a potential danger of more severe jams from a run of ice out of LaPrairie Basin, and hence possible flooding of not only EXPO itself, but of the low lying areas along the Montreal waterfront. To minimize this danger, a permanent ice control structure was constructed across the river at the lower end of the basin. The location and details of the structure are shown in Figs. 11, 12 and 13. Built in 1964-65 at a cost of $18,000,000, the LaPrairie Basin ice control structure was taken over by Transport in October of 1966 and came into full operation during the subsequent winter.

The structure, which is essentially an elaborate system of floating stoplogs set between piers, was designed to help form a stable ice cover on the basin earlier than would normally be the case. Once formed, this cover provides a large storage area for the ice continually generated in the Lachine Rapids section upstream. Thus, the large volume of ice which normally flows out of the basin in early winter is arrested and prevented from causing severe ice jams and consequent flooding in the Montreal area.

Successful operation of the control structure is dependent upon an increase in water level at the structure due to backwater effects of the ice cover advancing upstream into Montreal Harbour. Development of recommended operating procedures was based on extensive model studies. The two main factors investigated were the water levels at which the stoplogs should be placed to promote an ice cover, and the capacity of LaPrairie Basin to store ice under the cover thus formed. It was established from the model tests that if operated under certain stage discharge relationships created by ice jams downstream, the structure would initiate an ice cover on the basin. The operation is illustrated schematically in Fig. 14.

One method of estimating the volume of ice formed from a given area of water exposed to the cooling action of air is by establishing the rate of heat loss as the water is cooled to the freezing point and applying the rate found to later exposures. MacLachlin[2] determined this rate to be about 95 British Thermal Units transferred per day per square foot per degree difference between air and water temperatures, and went on to establish the equation:

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$$V = \frac{95.T.A}{144 \times 57.4} \qquad \text{(Simplified to } V = \frac{T.A}{87}\text{)}$$

where V = Vol. of ice formed per day in cu. ft.
T = Av. Diff. between air and water temp. in $^{\circ}F$.
A = Area of open water in ft.^{2}

This relationship was used to establish for various mean temperatures the cumulative production of ice in the open reach of the Lachine Rapids and Lake St. Louis, the area of which was taken to be 270×10^6 square feet. The storage capacity of the basin related to water elevation and rate of ice production in the Lachine Rapids, as obtained from model tests, is shown in the composite diagram in Fig. 15.

Because of the success icebreakers have had in recent years in maintaining an open channel, the high water conditions necessary for operation of the structure as described no longer occur, and usually the ice covers only about two-thirds of the width of the basin. Experimental work is now in hand to improve the efficiency of the ice retention capacity of the structure by converting some stoplogs to booms and floating them downstream. Tests are also being made with a boom made out of large diameter nylon rope.

Another feature of the Ministry's ice control program is the installation of a system of floating ice booms. Established initially on an experimental basis, the ice booms are now considered to be an integral part of the control works of the Ship Channel. The booms, which are constructed in 500 feet sections, consist of B.C. fir timbers 14 inches by 22 inches in section and 30 feet long, linked together with 2 inch diameter galvanized steel cable. A typical arrangement is illustrated in Fig. 16.

The booms in the Ship Channel have been designed and located to:

(a) Form a stable ice cover outside the shipping lanes as early as possible, thereby closing large areas of ice producing open water.

(b) Control the movement of this ice cover during its formation and retain it throughout the winter.

(c) Minimize erosion of the ice field caused by waves from passing ships and thus reduce the number of floes breaking off into the channel, presenting a hazard to shipping and creating serious ice jams.

The first booms were installed in Lake St. Peter five winters ago in the location shown on Fig. 17. This installation consists of four booms, each 2,000 feet in length. In the light of experience gained during the first two winters certain design modifications have been carried out to strengthen the booms and improve their performance. This installation has proved to be extremely efficient in preventing the ice cover from breaking up and has considerably eased the workload of the icebreakers in this area.

In conjunction with these booms artificial islands were created in Lake St. Peter to assist in control of ice. These islands (see Fig. 17), constructed of glacial till from dredging operations and topped with rock, measure 40 feet by 40 feet at the top (8 feet above low water datum), and were completed in 1968.

Ice booms have also been installed at two other locations, viz at Lavaltrie, near the lower end of the Vercheres Islands and at Ile St. Ours, a little further downstream (see Fig. 17). This is the second winter of operation for the Lavaltrie boom which has been very successful in controlling the ice in the channel north of the Vercheres Islands. The boom at St. Ours was installed this winter and its performance is being carefully observed.

Ice Studies

Complementary to, and in conjunction with the ice control measures described, an extensive program of research and development is continuing into all aspects of ice problems

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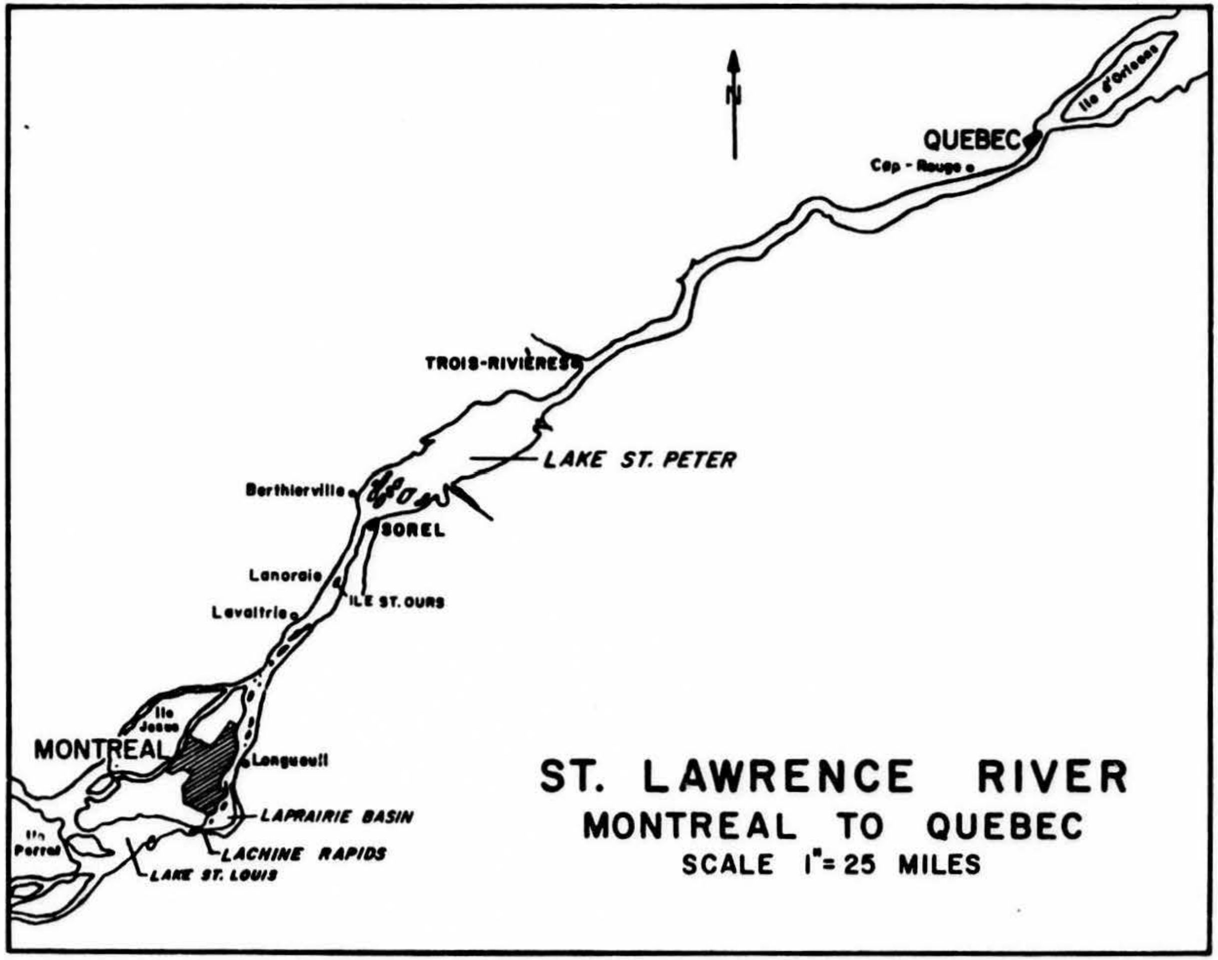
related to the St. Lawrence River. The work includes aerial photography, collection of data, field measurements, laboratory analyses, hydraulic model studies and icebreaker design.

References

1. Kivisild, H.R. - "Hanging Ice Dams" - IAHR 8th Congress, Montreal, 1959.

2. Report of Joint Board of Engineers - 1926 "St. Lawrence Waterway Project" APPENDIX E - "Ice Formation on the St. Lawrence and Other Rivers".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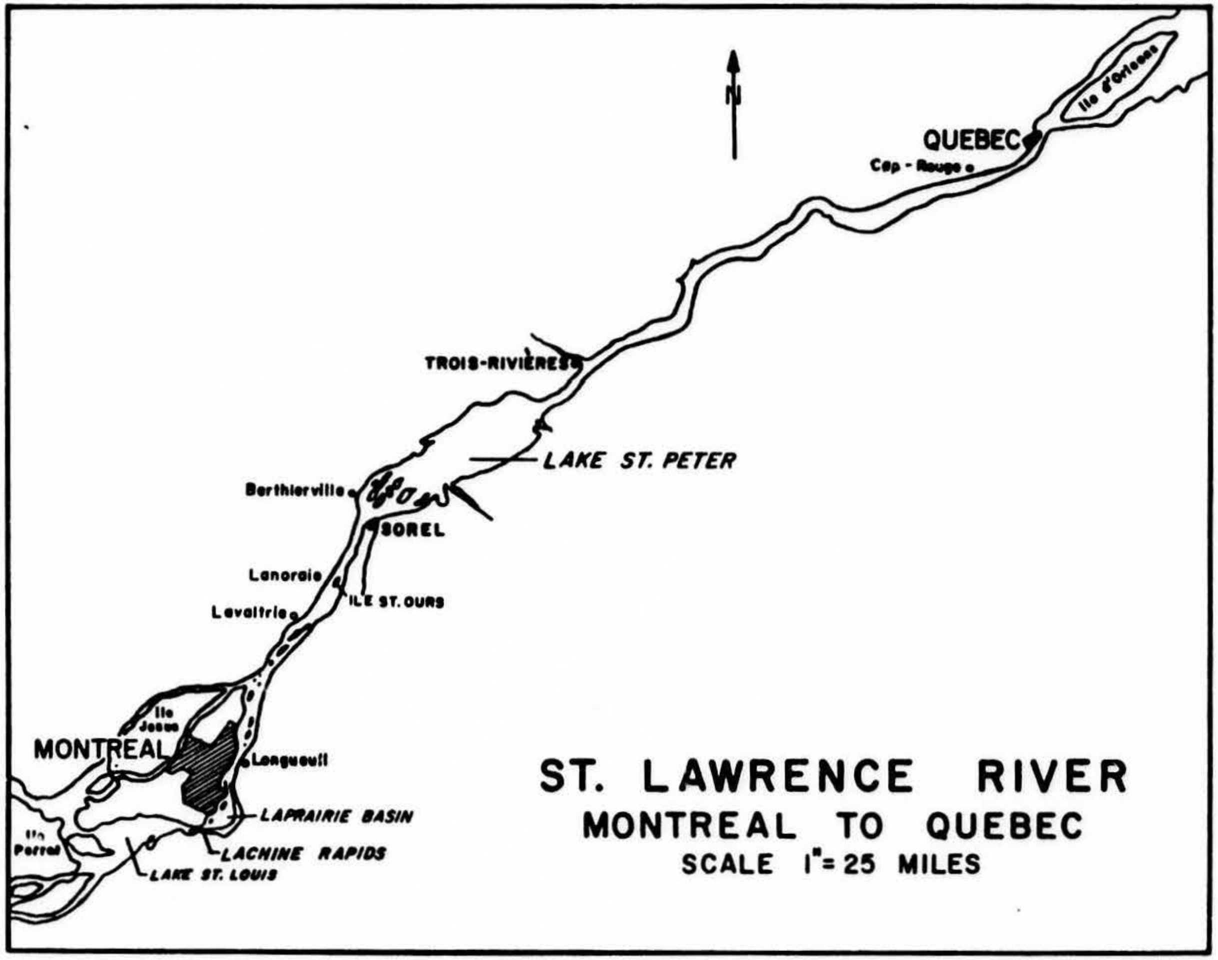
Cap - Rouge
TROIS-RIVIÈRES
LAKE ST. PETER
Berthierville
SOREL
Lanoraie
Lavaltrie
ILE ST. OURS
Ile Jesus
MONTREAL
Longueuil
LAPRAIRIE BASIN
Ile Perrot
LACHINE RAPIDS
LAKE ST. LOUIS
ST. LAWRENCE RIVER
MONTREAL TO QUEBEC
SCALE 1" = 25 MILES

FIG. No. 1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MONTREAL
Longueuil
LAPRAIRIE BASIN
Ile Perrot
LACHINE RAPIDS
LAKE ST. LOUIS
ST. LAWRENCE RIVER
MONTREAL TO QUEBEC
SCALE 1" = 25 MILES

FIG. No. 1

FIG. No. 2

THE "LADY GREY" AT WORK ON THE ST. LAWRENCE RIVER

FIG. No. 3

ICEBREAKERS BATTLING HEAVY ICE JAMS IN QUEBEC NARROWS - MID JAN. 1968

MONTREAL TO REPENTIGNY

REPENTIGNY TO ILE ST. OURS

FIG. No. 4

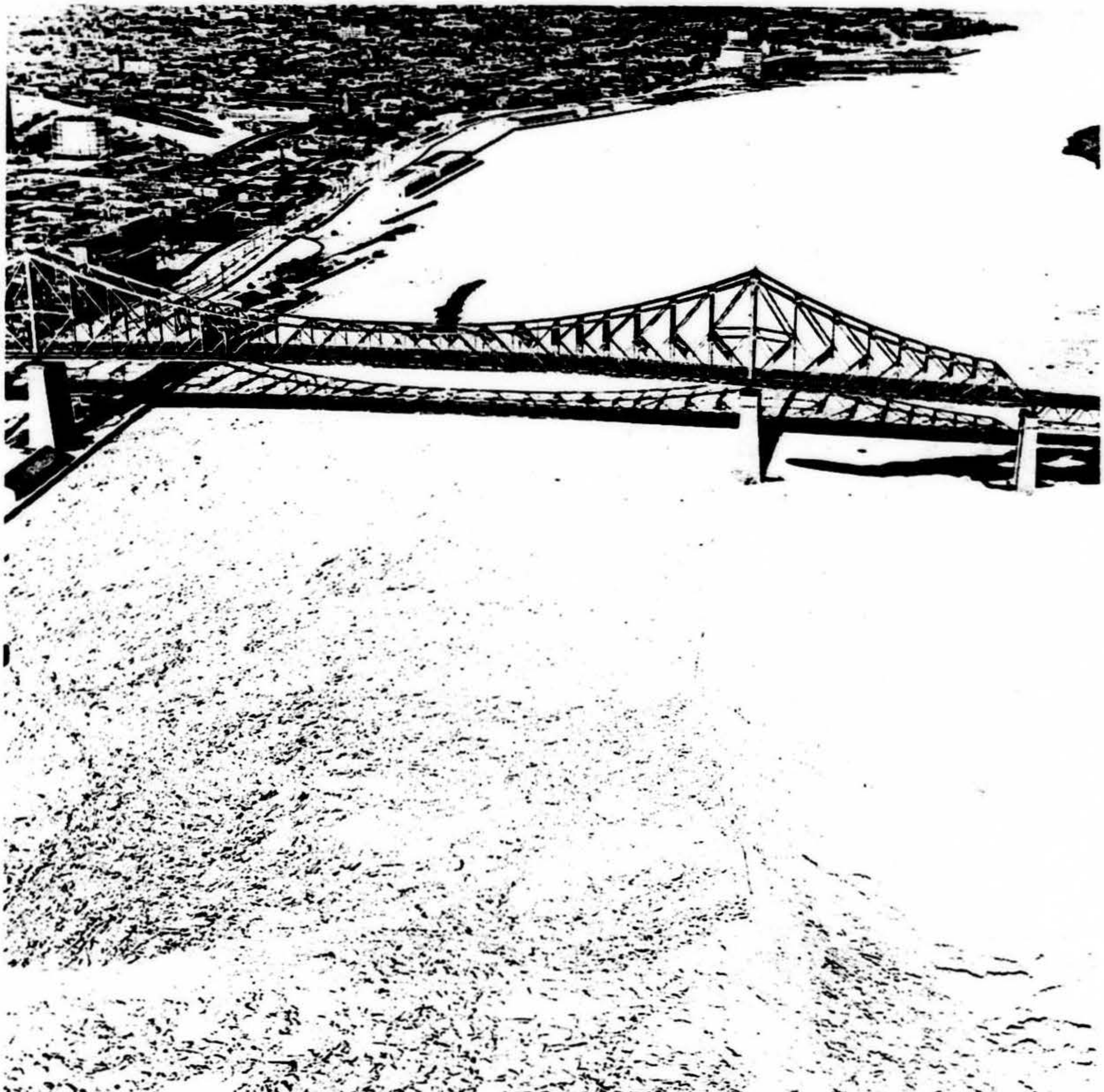
TYPICAL ICE COVER

ILE ST. OURS TO ILE MADAME

ILE MADAME TO PORT ST. FRANCOIS

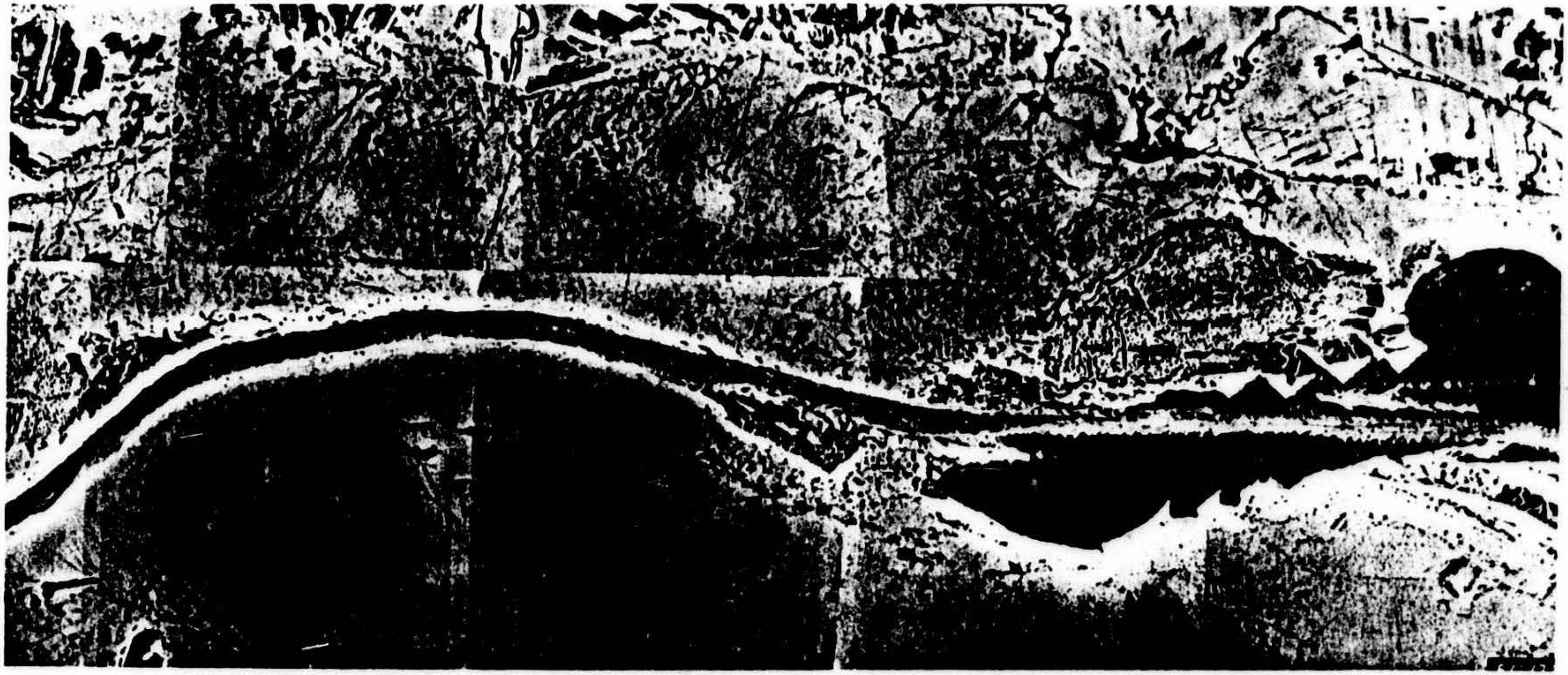
FIG. No. 5

TYPICAL ICE COVER



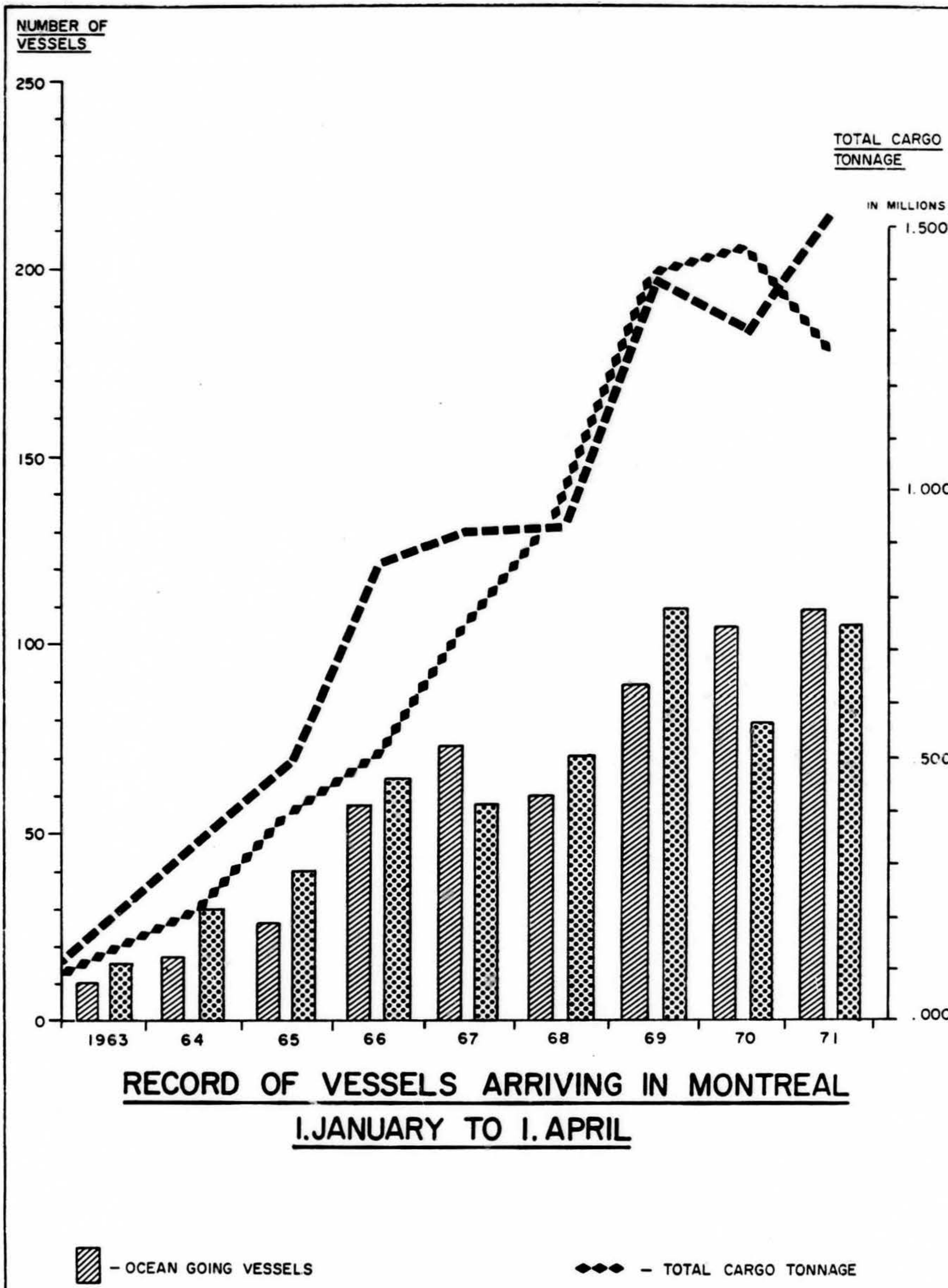
PACKED ICE COVER MONTREAL HARBOUR WINTER 1956

FIG. No. 6



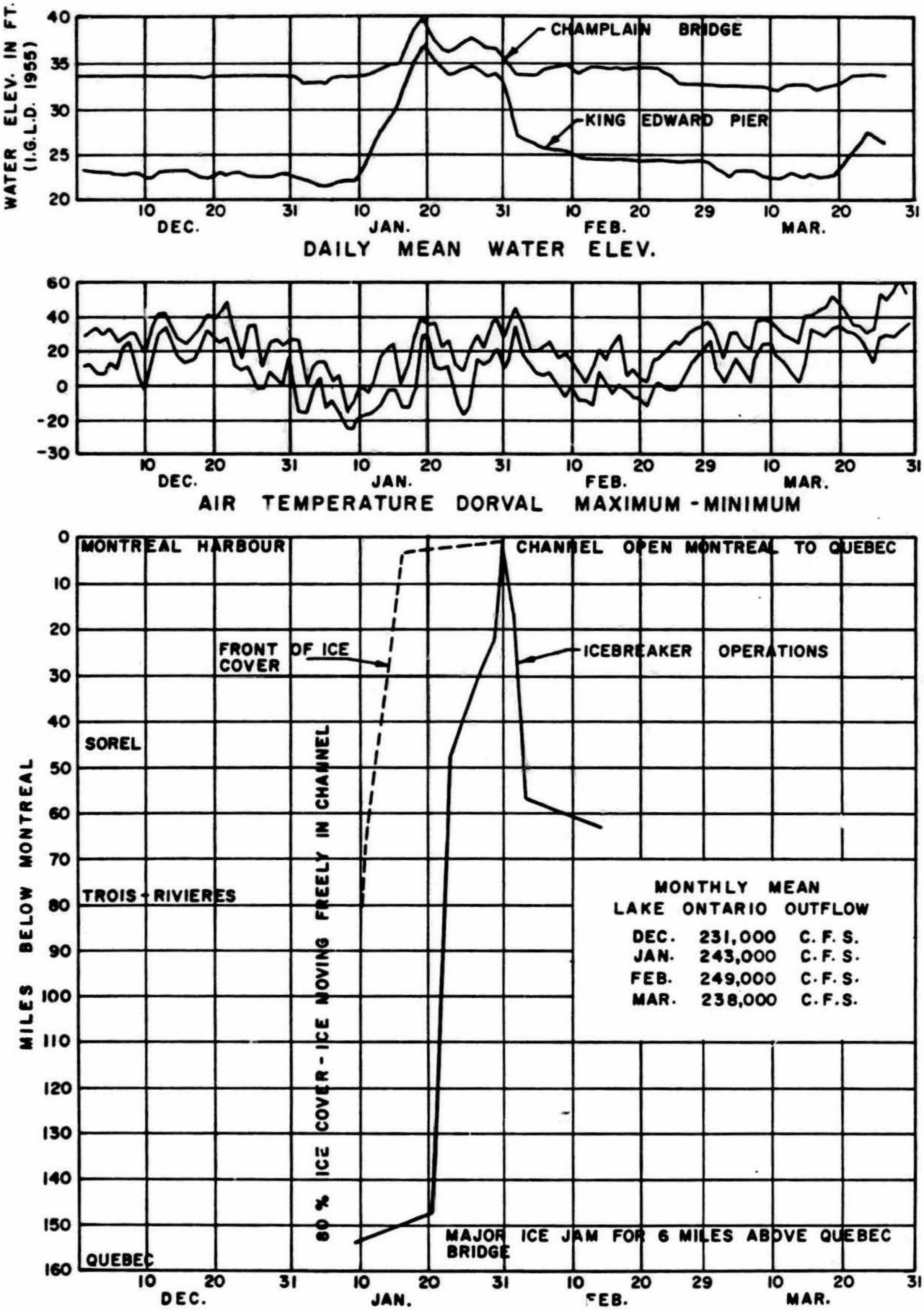
ICE BREAK-OFF IN LAKE ST. PETER

FIG. No. 7



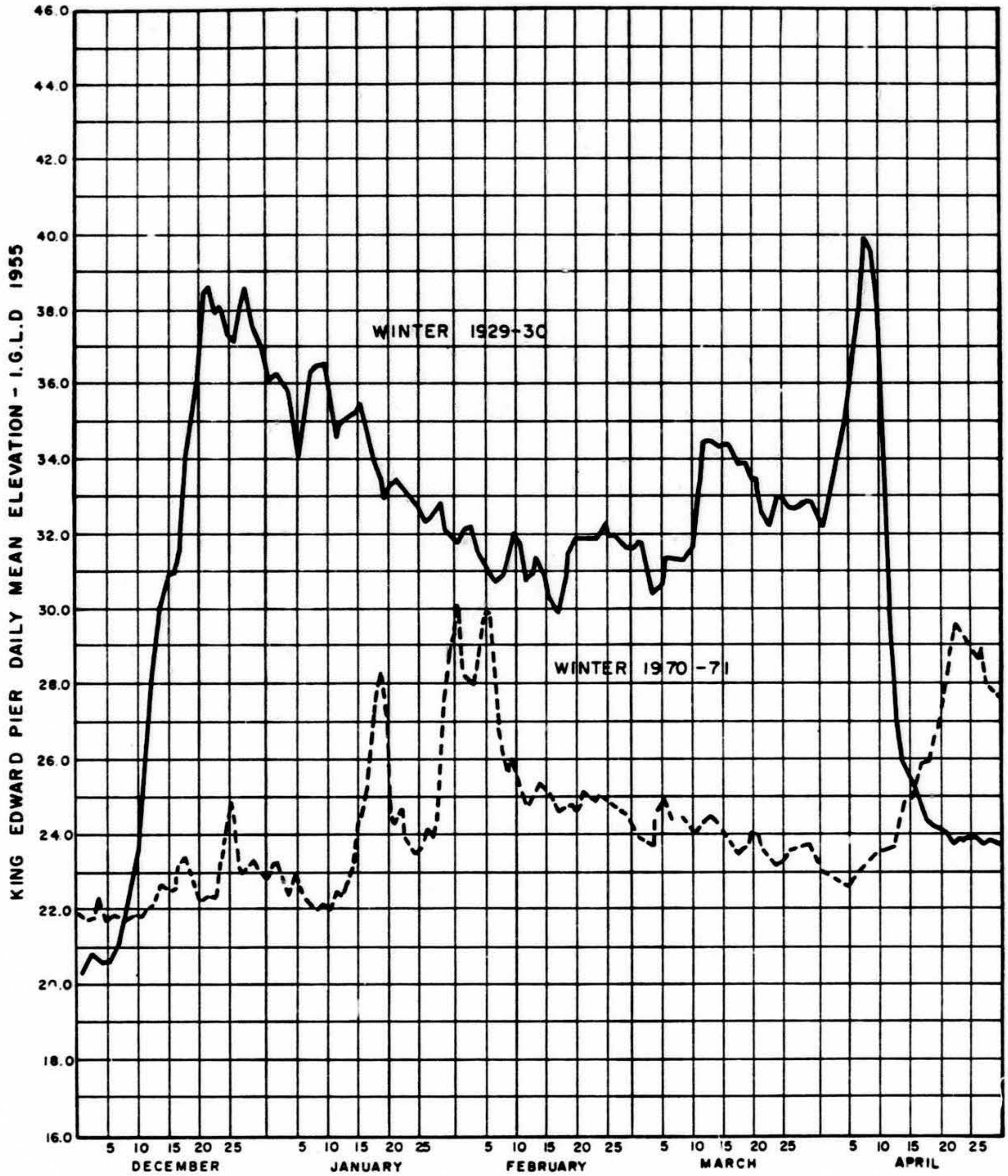
RECORD OF VESSELS ARRIVING IN MONTREAL 1. JANUARY TO 1. APRIL

FIG. No. 8



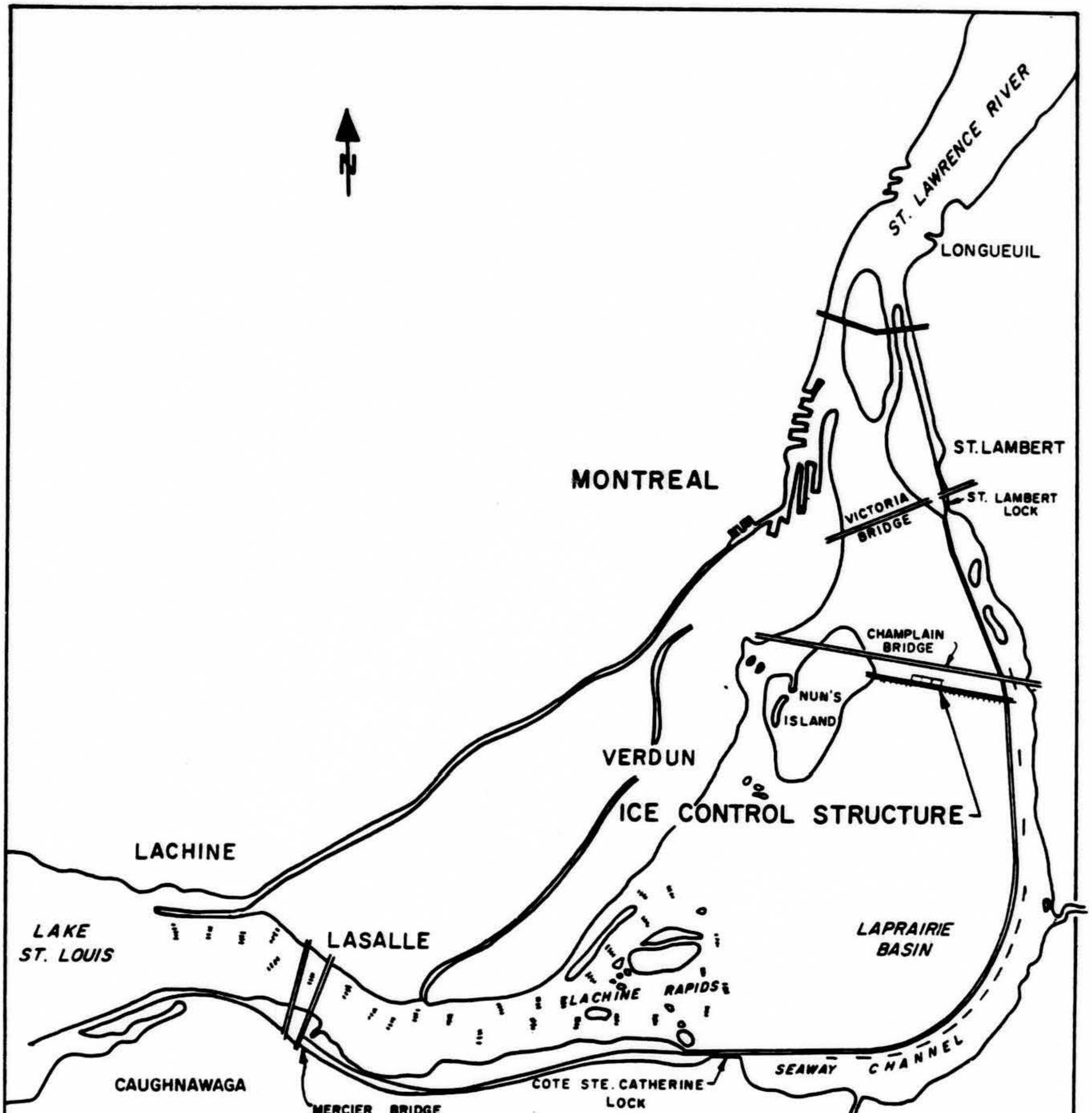
HISTORIC RECORD OF ICE COVER FORMATION AND ICEBREAKING
ST. LAWRENCE RIVER 1967 - 68

FIG. No. 9



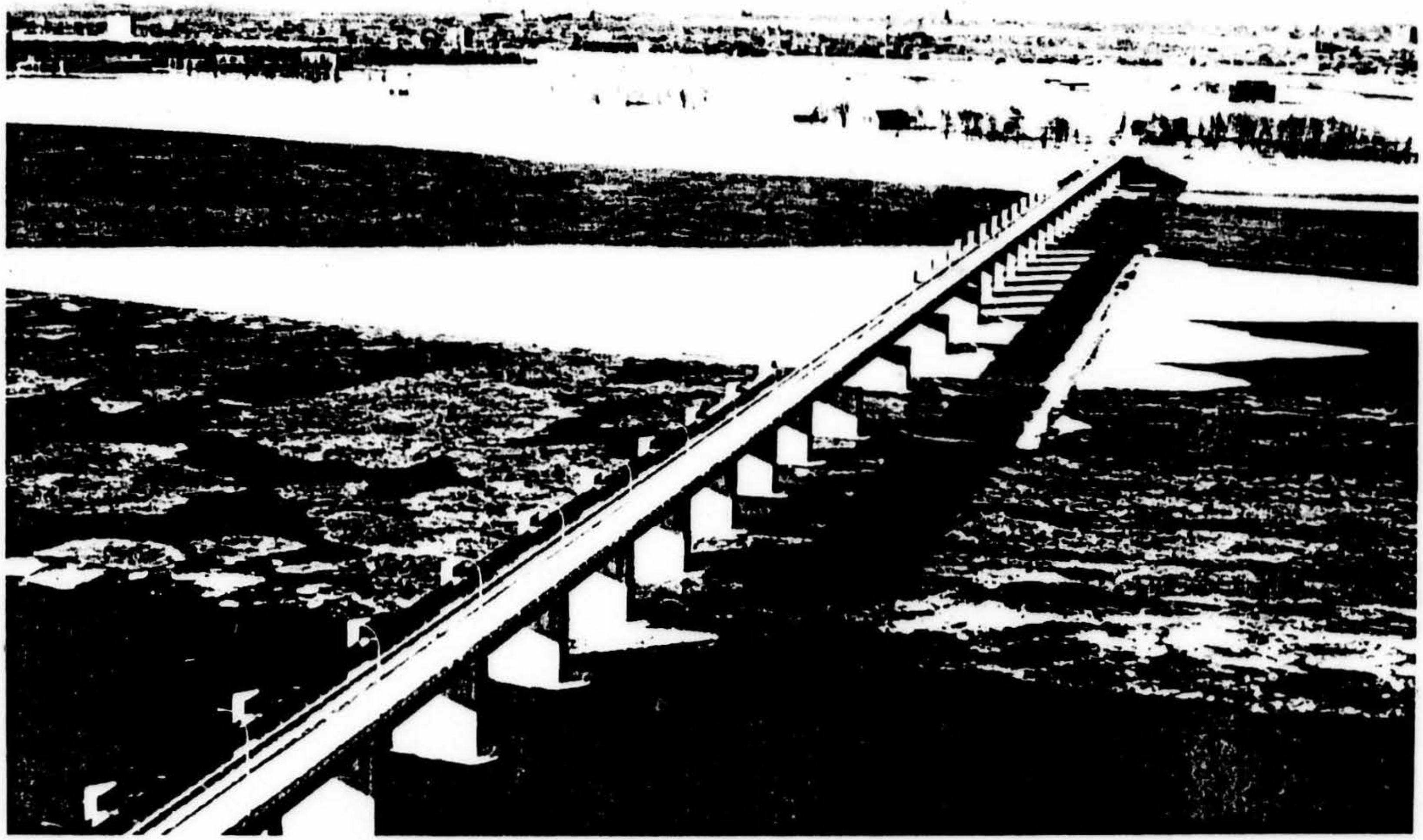
INFLUENCE OF ICEBREAKER OPERATION ON WATER LEVELS IN MONTREAL HARBOUR

FIG. No. 10



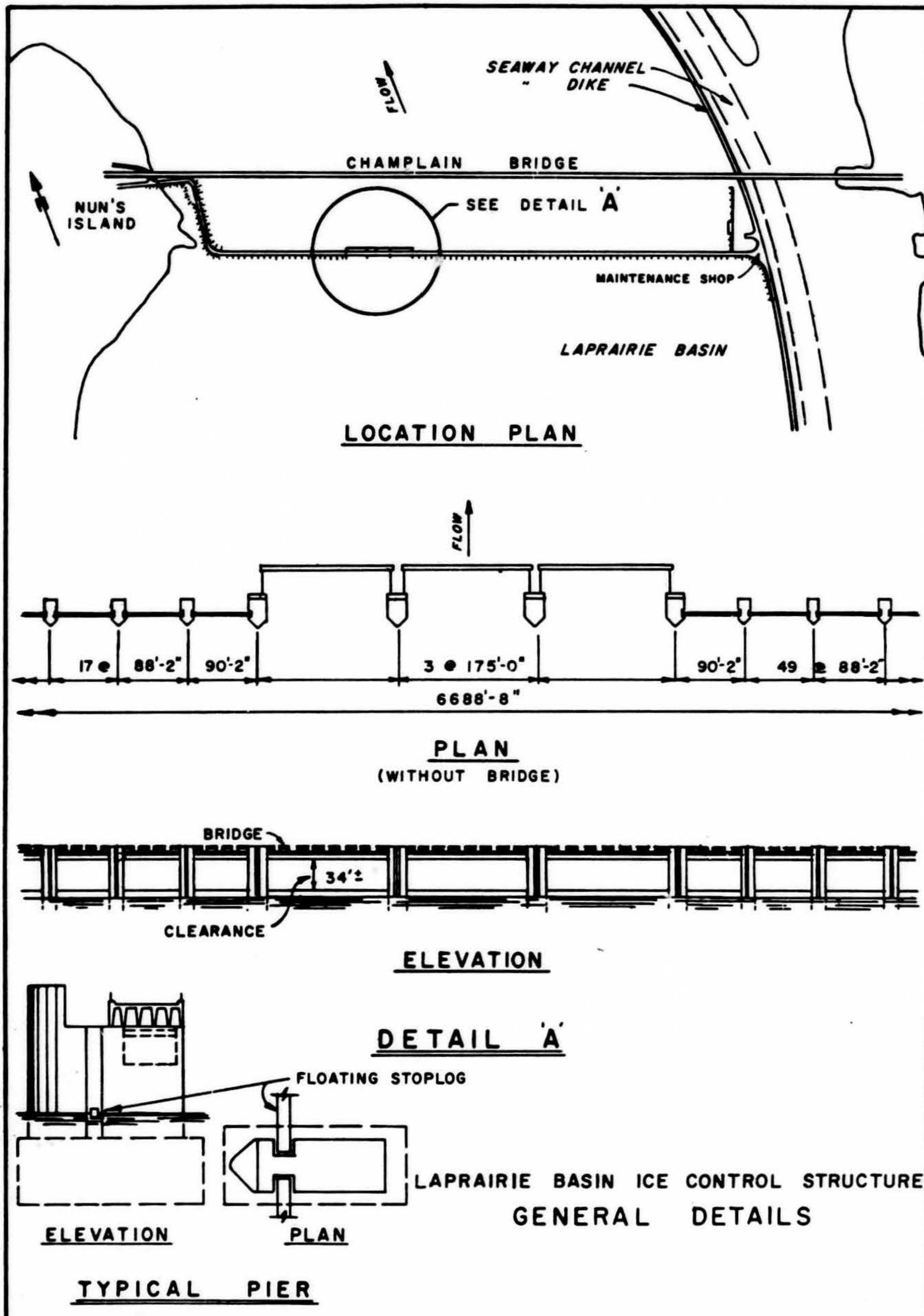
LOCATION OF ICE CONTROL STRUCTURE

FIG. No. 11



AERIAL VIEW OF ICE CONTROL STRUCTURE
LAPRAIRIE BASIN

FIG. No. 12

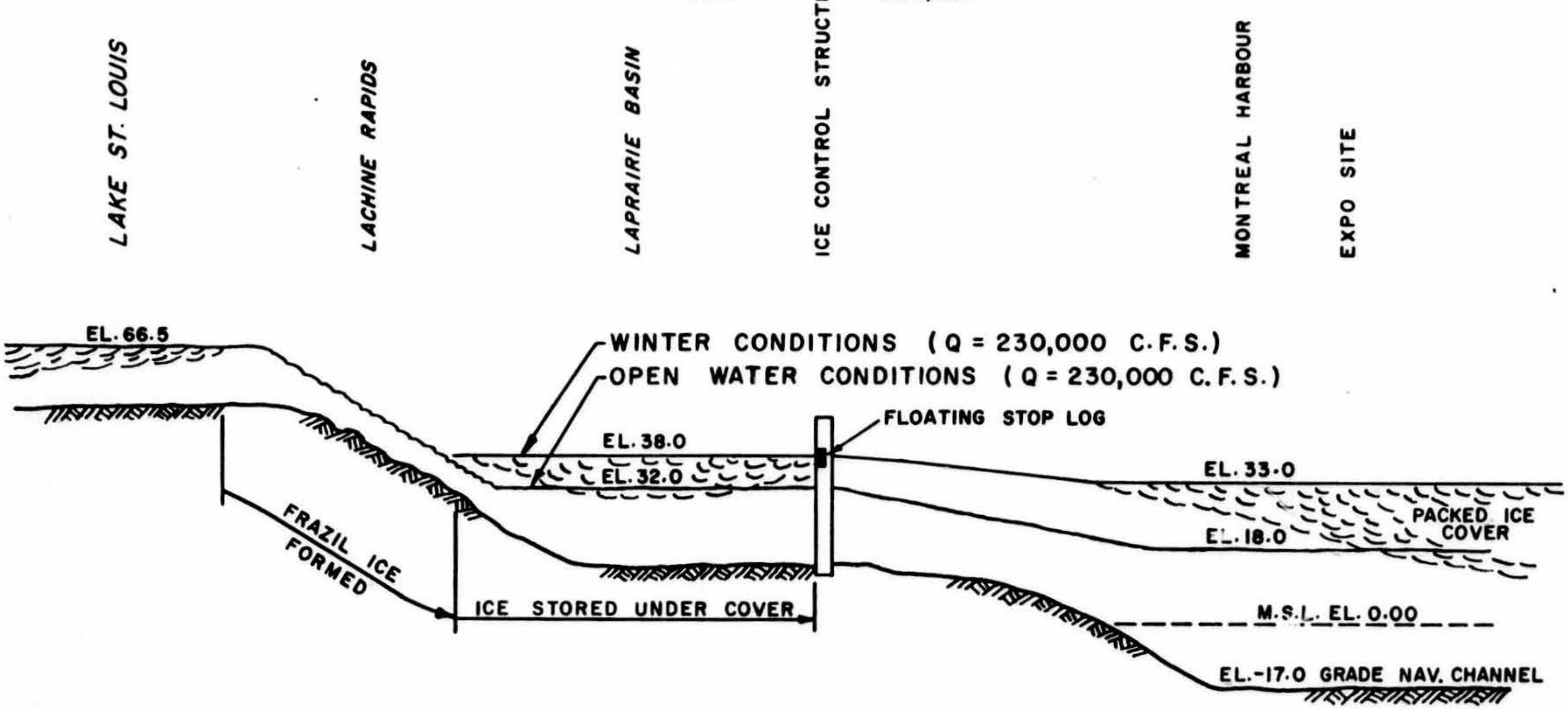


LAPRAIRIE BASIN ICE CONTROL STRUCTURE
GENERAL DETAILS

FIG. No.13

RECOMMENDED CRITERIA FOR
OPERATING STRUCTURE

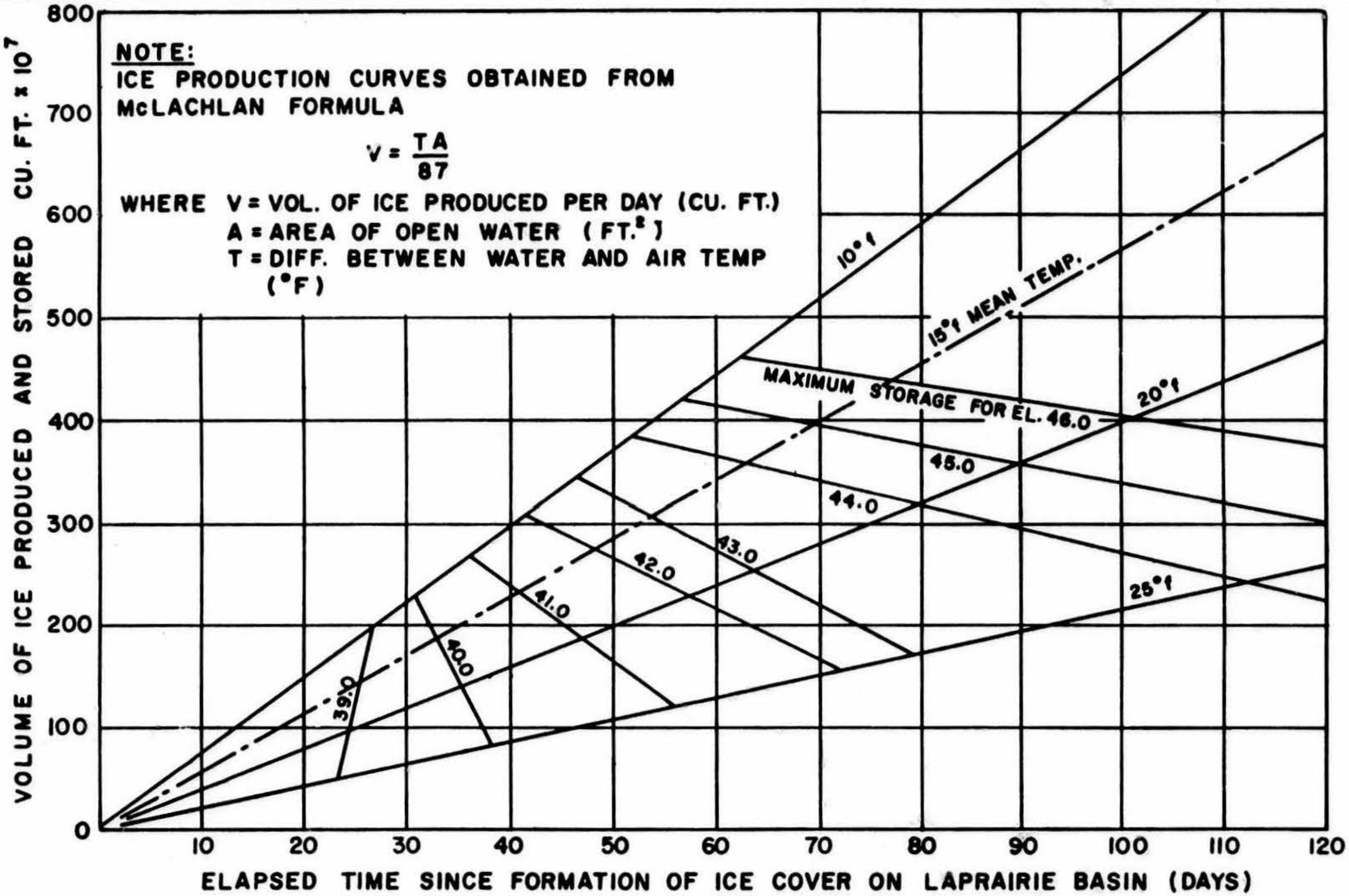
WATER ELEV (I.G.L.D. 1955)	DISCHARGE (C.F.S.)
37.0	200,000
38.0	250,000
40.0	300,000



LAPRAIRIE BASIN ICE CONTROL STRUCTURE

DIAGRAM SHOWING TYPICAL WATER SURFACE PROFILES IN THE ST. LAWRENCE RIVER AT MONTREAL FOR OPEN WATER AND WINTER CONDITIONS - INDICATING EFFECT OF ICE COVER DOWNSTREAM ON WATER LEVELS AT THE STRUCTURE.

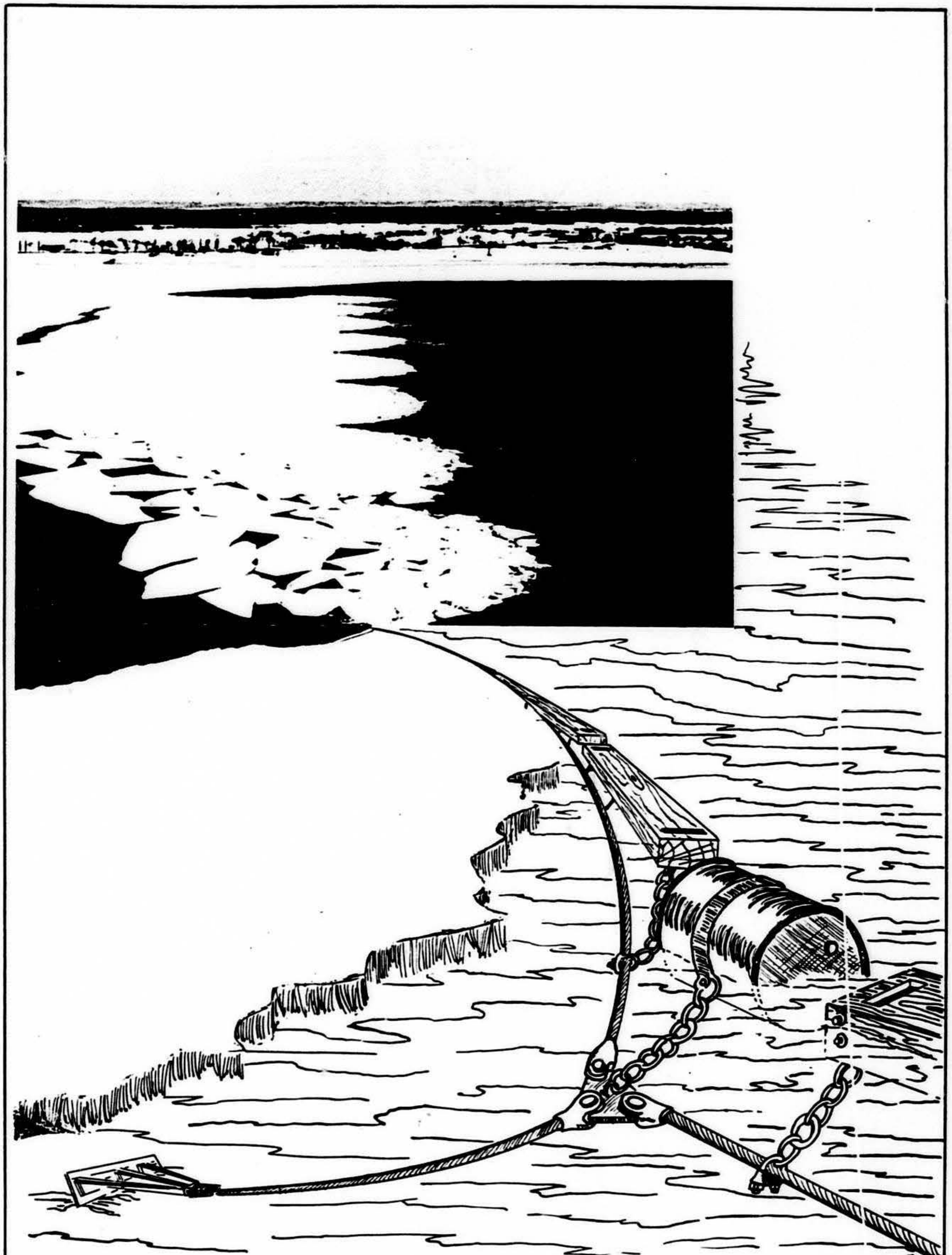
FIG. No. 14



MAXIMUM STORAGE CAPACITY OF LAPRAIRIE BASIN AT VARIOUS WATER ELEVATIONS RELATED TO ICE PRODUCTION IN LACHINE RAPIDS.

FROM: HYDRAULIC MODEL STUDIES
BY LASALLE HYDRAULIC LABORATORY

FIG. No. 15



TYPICAL ICE BOOM ARRANGEMENT

FIG. No. 16

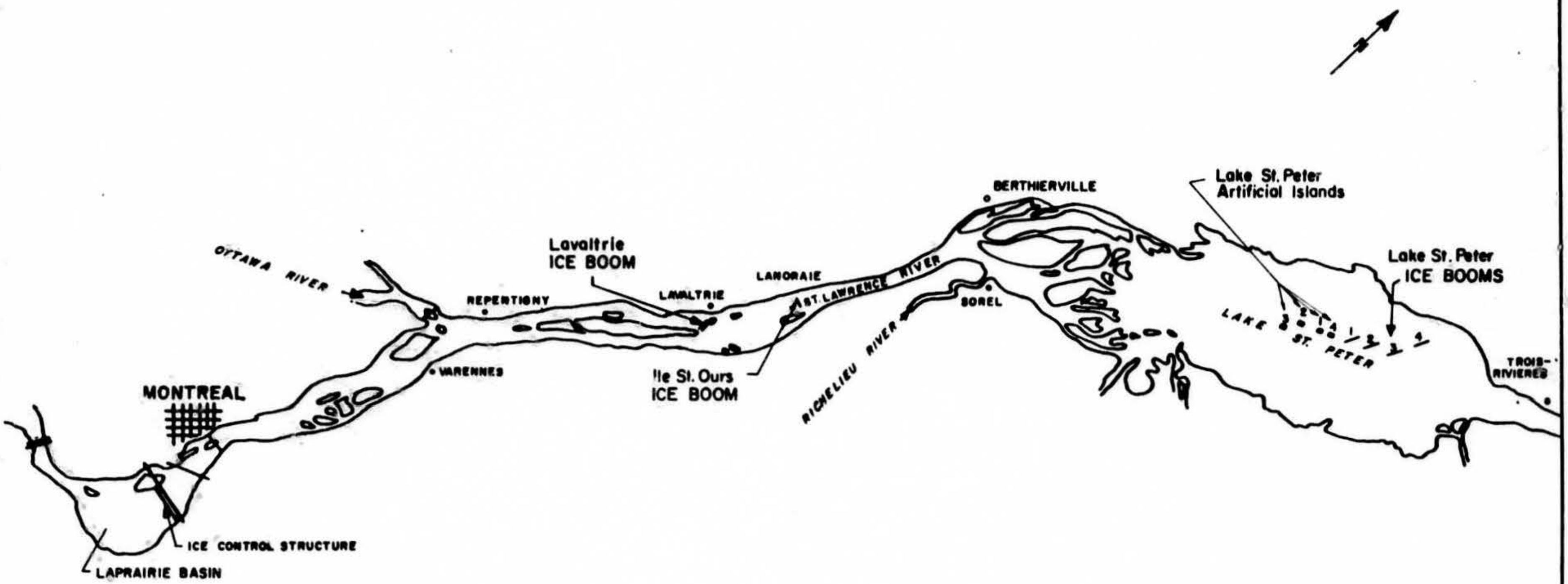


FIG. No.17

MINISTRY OF TRANSPORT

ST. LAWRENCE RIVER

LOCATION OF ICE BOOMS LAVALTRIE AND LAKE ST. PETER
AND ARTIFICIAL ISLANDS LAKE ST. PETER

APPROX. SCALE 1" = 10 MILES

INTRODUCTION

Le fleuve Saint-Laurent, une des grandes artères commerciales de l'Amérique du Nord, est affronté à un problème sérieux auquel sont liées de graves incidences économiques et que l'on a longtemps cru insurmontables, c'est-à-dire la présence de glace durant plusieurs mois et à des degrés divers.

En janvier 1966, le Ministère des transports, par l'entremise de sa division du Chenal maritime du Saint-Laurent, entreprit un vaste programme de contrôle des glaces, dans le but principal d'éliminer les inondations et par la même occasion promouvoir la navigation d'hiver sur le Saint-Laurent jusqu'à Montréal, tout en gardant la priorité absolue sur les fonctions de recherches et de sauvetage.

Le premier geste posé dans ce sens fut de mettre sur pied une section qui se spécialiserait dans ce domaine et dont les responsabilités majeures seraient les suivantes:

I - <u>Les opérations de déglaçage</u>
Diriger et coordonner les opérations de déglaçage entre Montréal et Notre-Dame de Portneuf.

II - <u>Les ouvrages de retenue des glaces</u>
a) Gérer et administrer le Régulateur des glaces du Bassin de Laprairie qui entrait en opération à l'hiver 1965-66
b) Planifier l'installation de nouveaux ouvrages de retenue des glaces, en diriger les opérations et en assurer l'entretien.

III - <u>Les recherches et études de glace</u>
Planifier et coordonner les travaux d'études et de recherches sur la glace.

Nous allons tenter d'expliquer ici en quoi consiste chacune des responsabilités précitées.

I - Les opérations de déglaçage

Secteur

Montréal à Notre-Dame de Portneuf
Voie maritime du Saint-Laurent (ouverture printanière - Entrée de la Voie maritime, Port de Montréal, au Lac St-François)

Objectifs

1 - Eliminer les inondations
2 - Promouvoir la navigation d'hiver

Origine

Les opérations de déglaçage sur le Saint-Laurent en amont de Trois-Rivières débutèrent en 1928 alors qu'une inondation particulièrement désastreuse à Montréal amena le Gouvernement fédéral à implanter des méthodes pour le contrôle des inondations. Des brise-glace furent assignés pour ouvrir un chenal en partant de Trois-Rivières et en remontant le fleuve durant le mois de février. Le but était d'arriver à Montréal avant la débâcle printanière, ouvrant ainsi une voie d'échappement au surplus d'eau.

Cause

Laissé à lui-même, le fleuve avait tendance à élever son niveau jusqu'au point d'inonder les basses terres riveraines de certaines régions. Ceci se produisait dès la formation de glace et du mouvement des glaçons à la surface de l'eau et se poursuivait durant tous les mois d'hiver de même qu'au moment de la crue printanière.

Lorsque viennent les températures froides de l'hiver, généralement dans les premiers jours de décembre, la glace commence à se former dans les baies et les petites rivières qui se jettent dans le fleuve. Cette glace nouvellement formée tend à se briser sous l'action du vent et des vagues et dérive ensuite dans le régime du fleuve. Avec la baisse constante des températures d'air, elle s'épaissit graduellement et c'est ainsi que se forment les glaçons.

L'amorce du premier champ de glace complet sur le fleuve se faisait lorsque les glaçons atteignaient la sortie du Lac St-Pierre, qui, à cause de son rétrécissement en forme d'entonnoir, provoquait l'arrêt de ceux-ci lorsqu'ils arrivaient en quantité suffisante. L'entassement combiné à l'effet du froid résultait en la fusion de tous ces glaçons. Il se formait alors ce que l'on appelle un pont de glace à partir duquel le champ se formait en progressant vers l'amont.

Ajouté à l'effet de l'entonnoir du Lac St-Pierre, il faut souligner la faible vélocité du courant. Dans le port de Montréal, les vitesses varient généralement entre trois et sept noeuds; dans la région de Sorel elles sont d'environ deux noeuds tandis que sur le Lac St-Pierre elles sont d'un noeud ou moins et parfois, avec l'effet de vents provenant de l'est et du nord-est, le courant de surface peut s'arrêter totalement et même remonter.

Lorsque le champ de glace était complètement formé sur la partie avale du Lac St-Pierre et que le mouvement des glaçons dérivant à la surface était arrêté, il progressait vers l'amont jusqu'à ce que le lac soit complètement couvert et puis continuait de progresser jusqu'à ce que le fleuve en entier soit couvert jusqu'à Montréal. Lors d'un hiver normal ce processus prenait environ dix jours mais dans des conditions rigoureuses il pouvait être complété en quatre jours ou moins.

L'effet que produisait ce champ de glace en eau peu profonde est qu'il réduisait la surface d'écoulement par laquelle le même volume d'eau devait se frayer un chemin et que le niveau normal du fleuve en hiver tendait à être plus élevé qu'en été.

En outre, ce champ de glace n'était pas d'une épaisseur régulière puisque la glace flottante pouvait être transportée sous la surface glacée par le courant puis à son tour voir son mouvement arrêté et ainsi s'empiler sous le champ de glace jusqu'à former une sorte de barrage. De plus, le mouvement de la glace à cause de la pression de la glace elle-même ou à cause du vent ou du courant provoquait un empilement de glace qui pouvait s'élever jusqu'à vingt pieds ou même plus au-dessus de la surface glacée. Ces accumulations de glace étaient aggravées par la neige épaisse qui, mélangée à la glace, agissait tel un ciment, et par le frasil, consistant en de millions de lamelles cristallisées, qui se forme dans les nappes d'eau peu profonde, où le courant est trop rapide pour permettre la formation de glace solide. Ce frasil étant entraîné par le courant aide à la fusion des glaçons entre eux.

La combinaison de ces deux phénomènes produit cette situation que la surface permettant l'écoulement de l'eau est si restreinte que le niveau du fleuve peut s'élever très rapidement et ainsi sortir de son lit, s'il est laissé à lui-même, causant ainsi de désastreuses inondations.

Endroits menacés d'inondation

Les endroits en amont du Lac St-Pierre qui sont particulièrement susceptibles d'être inondés sont le port de Montréal, Repentigny, Lanoraie et les rivières se jetant dans le Lac St-Pierre. Avant que ne soit entrepris le contrôle des inondations par la méthode du déglaçage, il y a une longue liste d'inondations survenues dans ces localités.

Principe d'opération pour enrayer les inondations employé jadis

Le mode d'opération de déglaçage employé jadis était que les brise-glace commencent à découper un chenal en remontant le fleuve jusqu'à Montréal aussitôt que le champ de glace était formé et suffisamment ancré aux rives et haut-fond pour que le déglaçage ou mouvement des navires ne puisse dégager la couche de glace couvrant le Lac St-Pierre et ainsi permettre la formation d'un embâcle à sa sortie. Le but de cette méthode était de créer et maintenir un chenal dans le centre du fleuve qui permettrait l'évacuation des morceaux de glace vers l'aval sans bloquer et former des embâcles qui arrêteraient l'écoulement de l'eau.

Principe d'opération employé aujourd'hui

Aujourd'hui, les opérations de déglaçage en plus de tenter d'éliminer les inondations ont pour objectif la promotion de la navigation d'hiver.

Avec la construction de plusieurs ouvrages de retenue des glaces, le mode d'opération a été amélioré même si fondamentalement il est demeuré le même. Ainsi, la saison opérationnelle commence habituellement à la mi-décembre et se termine à la mi-avril. En moyenne, trois brise-glace sont assignés à la région.

Le port d'attache de ces navires est Trois-Rivières, à cause de sa proximité en aval du Lac St-Pierre. Toutes les patrouilles quotidiennes et opérations de déglaçage pour la saison sont effectuées de ce port et dictées par les rigueurs du climat et les conditions de glace dans la voie navigable.

Le principe d'opération en cas d'embâcle est de briser l'amoncellement de glace par l'aval, de façon à ce que les morceaux de glace puissent être évacués par le courant au fur et à mesure qu'ils sont détachés. Des brise-glace qui attaqueraient l'embâcle par sa partie amont seraient complètement inefficaces et ne serviraient qu'à empiler davantage la glace. Il est à noter que de forts vents à contre-courant pendant une longue période de temps peuvent entraver sérieusement l'évacuation de la glace cassée et même parfois faire remonter cette glace. Dans ces conditions le travail des brise-glace devient extrêmement hardu. Encore là, la construction d'ouvrages de retenue des glaces et des améliorations apportées au chenal navigable ont grandement aidé à diminuer ces inconvénients.

La procédure normale employée pour briser un embâcle est que le plus puissant des brise-glace en opération brise l'amoncellement de glace tandis que les deux autres, à l'arrière, dégagent les voies moins congestionnées, brisent les trop grands morceaux et maintiennent un déblaiement rapide. Quand l'évacuation de la glace en aval de l'embâcle ne pose aucun problème, une autre méthode très efficace est souvent employée: deux brise-glace situés parallèlement, un de chaque côté du chenal, travaillent simultanément, attaquant l'embâcle tout à tour et dégageant ainsi la glace qu'il y a entre eux.

Plusieurs fois durant la saison, et quand les conditions le permettent, un brise-glace est envoyé dans le port de Montréal pour y dégager certains bassins ou quais, aussi des entailles sont ordinairement effectuées au milieu de l'hiver ou tôt au printemps dans le champ de glace de l'entrée de la Voie maritime du Saint-Laurent pour en faciliter l'ouverture éventuelle. Cette ouverture printanière est effectuée par les plus légers des brise-glace dès que les conditions météorologiques le permettent et que la situation en aval est favorable.

Effectifs

Les brise-glace qui travaillent aux opérations de déglaçage font partie de la Garde côtière canadienne, un service exploité par le Ministère fédéral des transports, qui défraie toutes les dépenses encourues par les travaux de déglaçage. L'on estime le coût annuel des opérations de déglaçage à environ $850,000.

Les brise-glace qui sont assignés au secteur de Montréal à Notre-Dame de Portneuf sont généralement choisis parmi ceux qui apparaissent dans le tableau suivant.

<u>Figure 1</u>. **Secteur des opérations de déglaçage**

II - Les ouvrages de retenue des glaces

Objectifs

Les ouvrages de retenue des glaces que la section Contrôle des glaces opèrent et entretiennent sur le fleuve Saint-Laurent furent construits dans le but d'atteindre les objectifs suivants:

1) Former un champ de glace beaucoup plus tôt que ne le ferait normalement la nature.
2) Retenir ce champ de glace durant toute la saison d'hiver malgré les vents à contre-courant, les vagues de même que les vagues créées par les navires.
3) Diminuer la superficie des régions à l'eau libre réduisant par le fait même la production continuelle de nouvelle glace.
4) Arrêter la glace flottante sous un couvert de glace formé, afin de l'empêcher de pénétrer dans le régime du fleuve.
5) Accélérer les vitesses du courant dans le chenal navigable, facilitant ainsi une évacuation rapide de la glace flottante et apportant une aide considérable aux opérations de déglaçage.

Ces ouvrages de retenue des glaces peuvent se diviser en trois catégories:

A) Structures permanentes
   1. Le Régulateur des glaces.
B) Estacades flottantes
   1. Estacades du Lac St-Pierre
   2. Estacade de Lavaltrie
   3. Estacade de l'Ile St-Ours
C) Iles artificielles
   1. Les îles artificielles du Lac St-Pierre

B-1) Les estacades flottantes du Lac St-Pierre

La section Contrôle des glaces, ayant été chargée de la planification pour l'installation de nouveaux ouvrages de retenue des glaces, fit construire en 1967 à titre expérimental des estacades flottantes sur le Lac St-Pierre.

Des études antécédentes avaient démontré que le champ de glace qui se formait du côté nord du Lac St-Pierre était très instable. De forts vents, les vagues causées par les navires, une baisse rapide des températures pouvaient arracher de grandes battures de glace provenant de ce champ et ensuite celles-ci ,dérivant dans le chenal ,venaient bloquer la sortie du Lac St-Pierre et ainsi causer de graves embâcles.

Emplacement

Les estacades sont situées sur le Lac St-Pierre, à environ 1,000 pieds, au nord du chenal maritime du Saint-Laurent, entre l'ancrage de Yamachiche et la courbe numéro 3.

Description

La longueur totale des estacades (distance en ligne droite, entre les deux extrémités) est d'environ 8,000 pieds, s'échelonnant en tronçons de 2,000 pieds chacun. Elles sont constituées de poutres de bois flottantes attachées par des chaînes à des câbles d'acier enfouis dans le fond du lac.

Une estacade flottante est composée de sections de 500 pieds de longueur comprenant chacune 13 poutres de bois espacées de 6 pieds l'une de l'autre. Un câble d'acier galvanisé de 2 pouces de diamètre d'une longueur de 500 pieds, court à quatre pieds en dessous de ces poutres auxquelles il est relié par des chaînes de 4 pieds de longueur. En tout les estacades comportent 20 de ces sections reliées les unes aux autres au moyen de plaques de raccord. Chacune de ces plaques est à son tour reliée à une ancre de modèle spécial, pesant environ 6 tonnes; ces ancres sont enfouies sous le lit du fleuve à 400 pieds de distance l'une de l'autre. Les extrémités de l'estacade sont assujetties à des ancres plus lourdes, du type jumelé. A chaque plaque de raccord, des bouées cylindriques sont attachées aux estacades pour en assurer la flottabilité et pour neutraliser le poids de la plaque de raccord et du câble d'ancrage.

Chaque ancre de type "B-1" pour usage dans la glaise et dans la matière inconsistante est mise en place avec précision et soumise au préalable à des tractions afin d'assurer un contact ferme avec le lit du fleuve. Dans la matière très molle, il peut être nécessaire de placer deux ancres ensemble. La longueur du câble d'ancrage est déterminée par la profondeur de l'eau. On a trouvé que pour une profondeur de 40 pieds la longueur optimale était d'environ 500 pieds; pour les estacades du Lac Saint-Pierre où la profondeur varie entre 10 et 15 pieds, un câble de 250 pieds paraît approprié.

Modifications

Ayant été installées à titre expérimental en 1967, plusieurs modifications furent apportées depuis. Du matériel plus solide et résistant a été substitué à des pièces vulnérables, ce qui en plus d'en augmenter l'efficacité a réduit considérablement le coût d'entretien.

Résultats obtenus

Les résultats obtenus sont pour le moins fantastiques comme le démontrent les figures 16 et 17. Maintenant le champ de glace du côté nord du Lac Saint-Pierre peut être maintenu en place durant tout l'hiver, réduisant le risque d'embâcle et contribuant par la même occasion à la diminution de temps que doivent passer les brise-glace aux opérations de déglaçage.

à son tour reliée à une ancre de modèle spécial, pesant environ 6 tonnes; ces ancres sont enfouies sous le lit du fleuve à 400 pieds de distance l'une de l'autre. Les extrémités de l'estacade sont assujetties à des ancres plus lourdes, du type jumelé. A chaque plaque de raccord, des bouées cylindriques sont attachées aux estacades pour en assurer la flottabilité et pour neutraliser le poids de la plaque de raccord et du câble d'ancrage.

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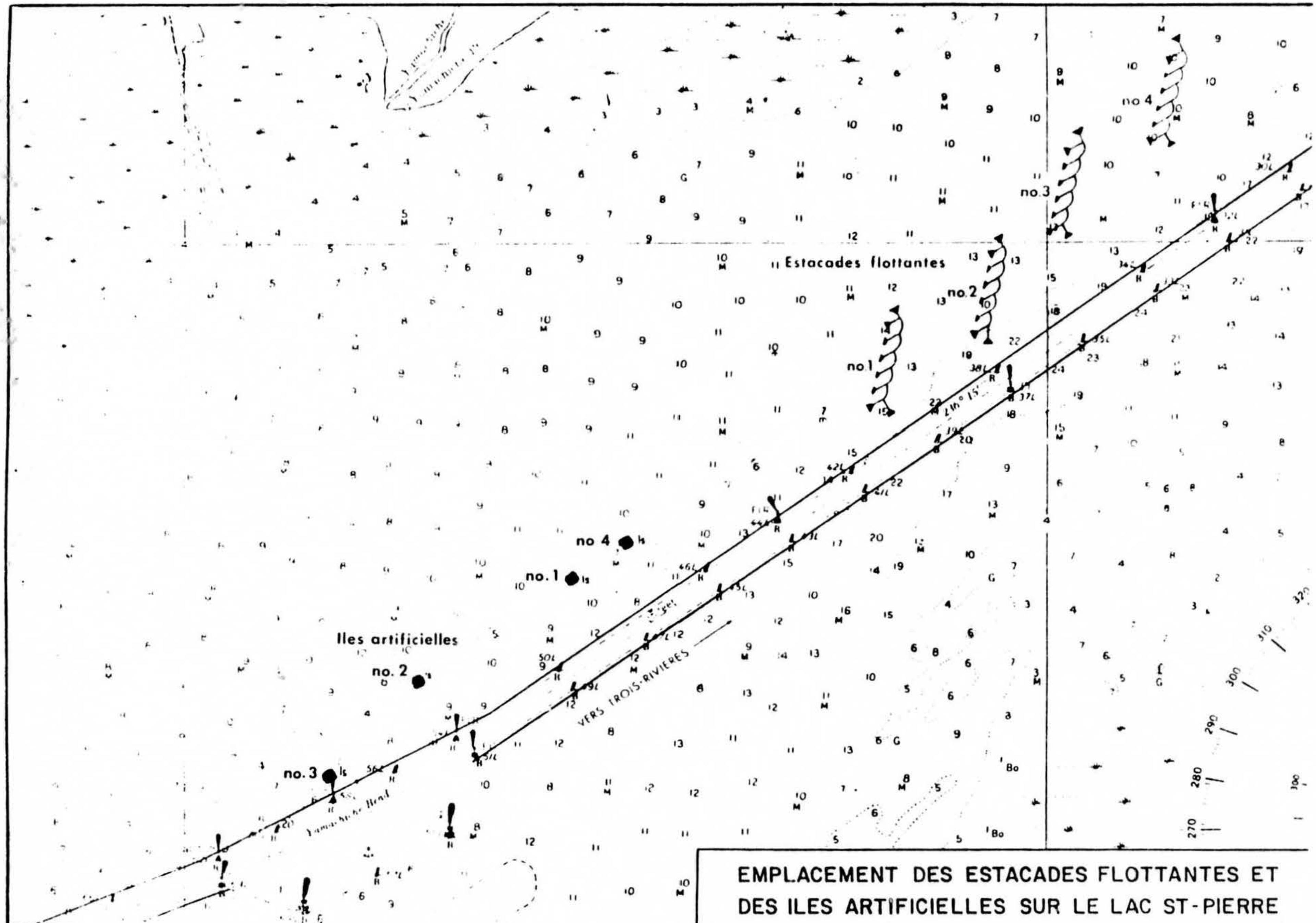
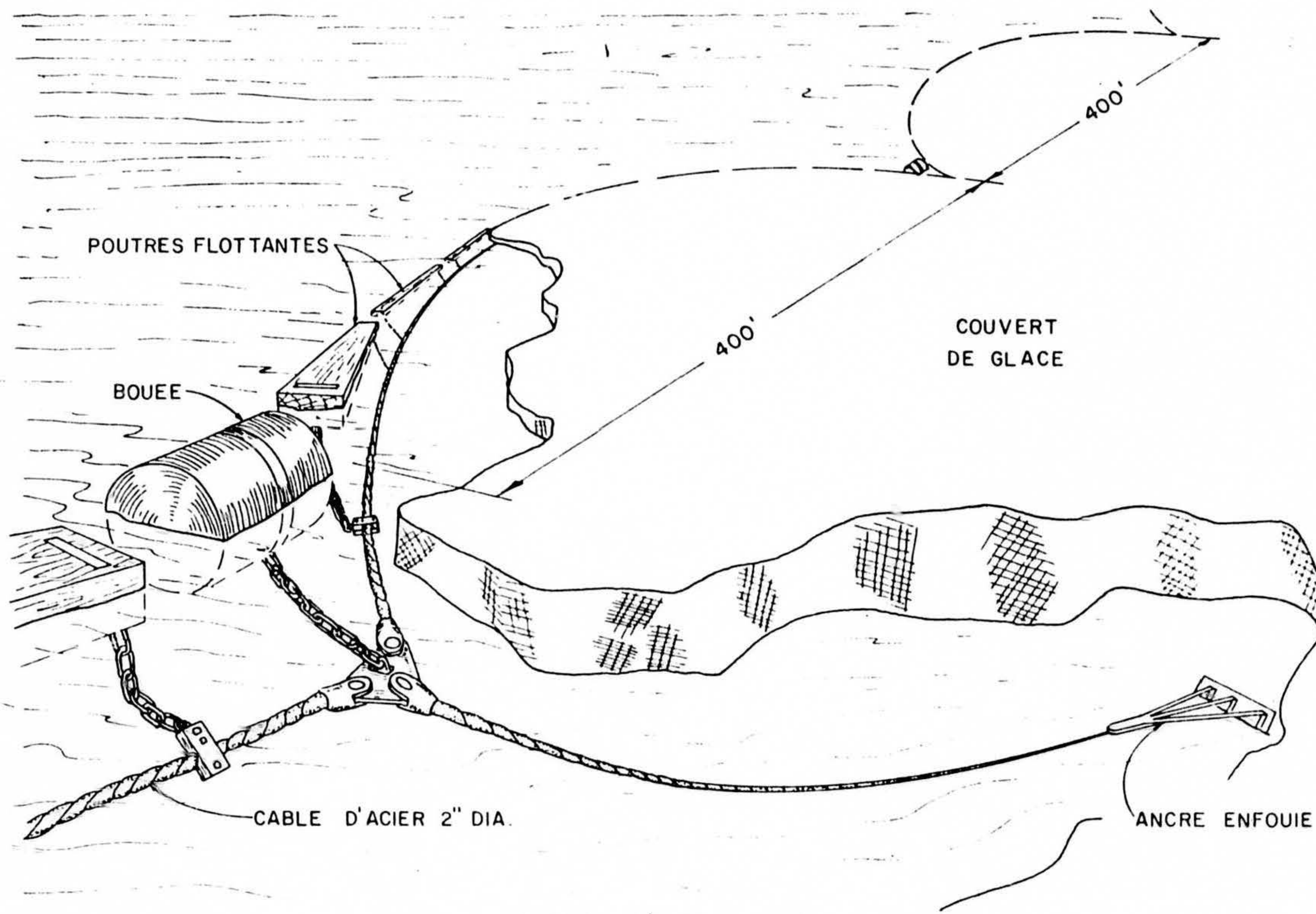


EMPLACEMENT DES ESTACADES FLOTTANTES ET DES ILES ARTIFICIELLES SUR LE LAC ST-PIERRE



CROQUIS D'UNE SECTION
D'ESTACADE FLOTTANTE

B-2) L'Estacade de Lavaltrie

A la suite des succès remportés par les estacades flottantes du lac Saint-Pierre, une autre région, soit le chenal de Repentigny ou "Chenal du Nord", qui s'étend entre Lavaltrie et l'Ile Ste-Thérèse, a fait l'objet d'études en vue de l'installation d'une estacade flottante.

Il était depuis longtemps reconnu que cette surface d'environ 200 millions de pieds carrés était une grande productrice de nouvelle glace (1,200 pi.cu./sec. à -20^{o} F) qui dérivait vers le Lac St-Pierre pour venir le congestionner. Aussi lorsqu'un champ de glace réussissait finalement à se former dans cette région, souvent tard dans l'hiver, il était fréquemment arraché par gros morceaux qui allaient ensuite causer des embâcles en aval.

La section Contrôle des glaces fit donc construire, en 1969, une estacade qui allait former un champ de glace dans cette région dès le début de l'hiver et le maintenir en place jusqu'à la débâcle printanière.

Cette estacade entra en opération dès le début de l'hiver 1969-70.

Emplacement

L'estacade est située sur le fleuve St-Laurent, environ 2 milles en amont de Lavaltrie, sur une ligne perpendiculaire au "chenal du nord" et s'étendant entre l'extrémité amont de l'Ile Mousseau et le déblai à l'extrémité aval de l'Ile Bouchard.

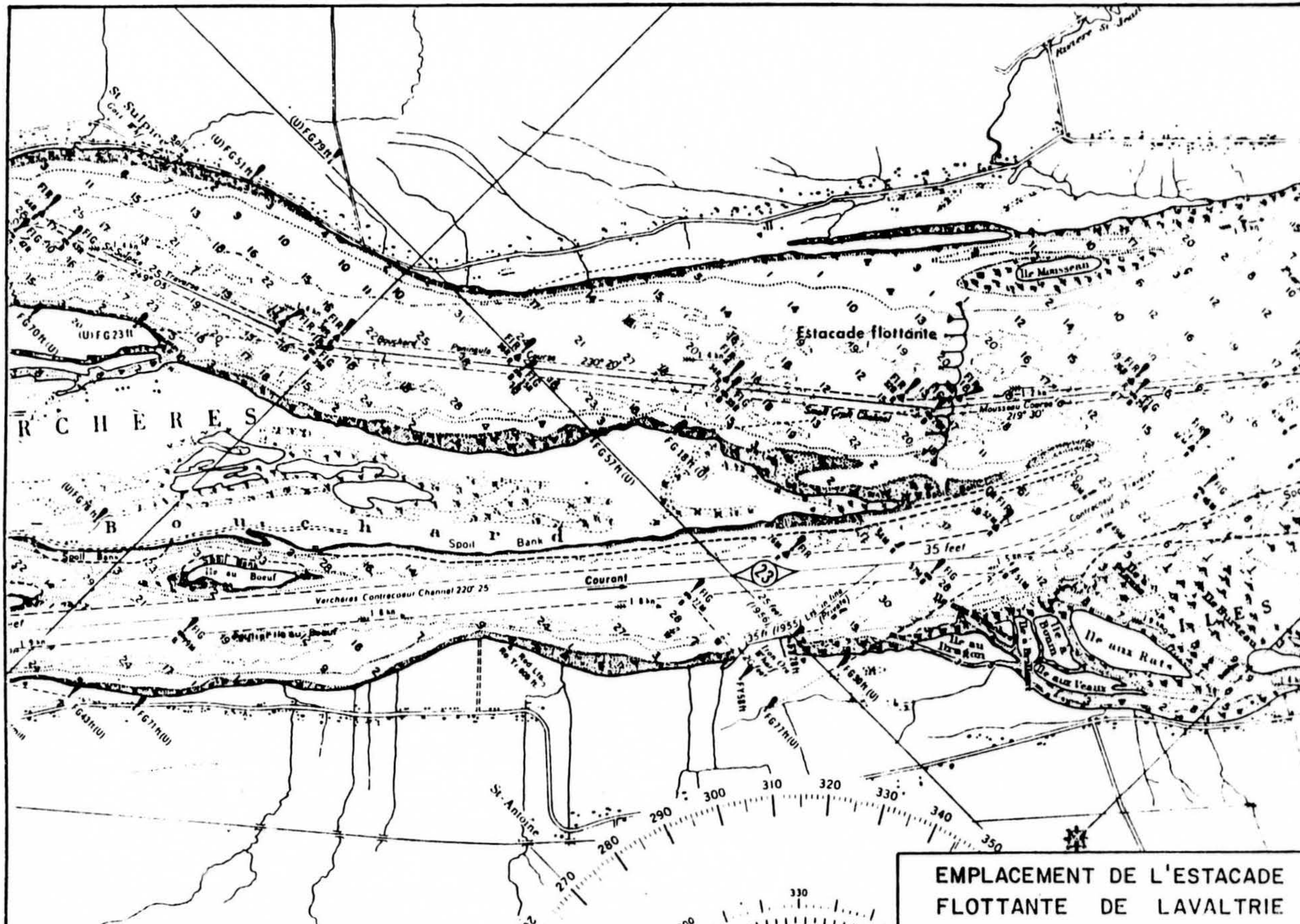
La profondeur de l'eau à cet endroit varie généralement entre 12 pieds et 30 pieds et les vitesses de courant sont de 2.5 pieds par seconde. Le lit du fleuve se compose généralement d'argile grise de grande plasticité avec quelques bandes étroites de sable vaseux.

Description

L'estacade flottante de Lavaltrie est du même type que celles installées sur le Lac St-Pierre. Le fleuve, à cet endroit, étant d'environ 5,000 pieds de large, il fut décidé de limiter la longueur de l'estacade à 3,200 pieds en prenant pour acquis que le champ de glace se formerait naturellement sur les sections laissées ouvertes entre les extrémités de l'estacade et le rivage.

Résultats obtenus

Encore ici les résultats sont fantastiques. Le champ de glace, qui auparavant était très lent à se former dans ce chenal, est en place maintenant dès le début de l'hiver pour y demeurer jusqu'à la débâcle printanière. Les figures 20, 21 et 22 illustrent bien le travail effectué par cette estacade.



EMPLACEMENT DE L'ESTACADE FLOTTANTE DE LAVALTRIE

FIGURE

B-3) L'Estacade de l'Ile St-Ours

Afin de compléter l'excellent travail effectué par l'estacade de Lavaltrie sur le "Chenal du Nord" une autre estacade du même type fut construite et installée en 1971 en vue d'arrêter la production de nouvelle glace (1,200 pi.cu./sec. à -20°,F) sur le secteur compris entre l'extrémité aval de l'Ile St-Ours et l'estacade de Lavaltrie. La surface affectée par l'estacade serait d'environ 180 millions de pieds carrés.

Emplacement

L'estacade est située sur le fleuve St-Laurent environ 1 mille 1/2 en amont de Lanoraie sur une ligne perpendiculaire au "Chenal du Nord" et s'étend entre l'extrémité aval de l'Ile St-Ours et l'extrémité aval d'une batture au nord du "Chenal du Nord".

La profondeur de l'eau à cet endroit varie généralement entre 4 pieds et 30 pieds et les vitesses de courant sont de 2.5 pieds par seconde. Le lit du fleuve se compose généralement d'argile grise de grande plasticité avec quelques bandes étroites de sable vaseux.

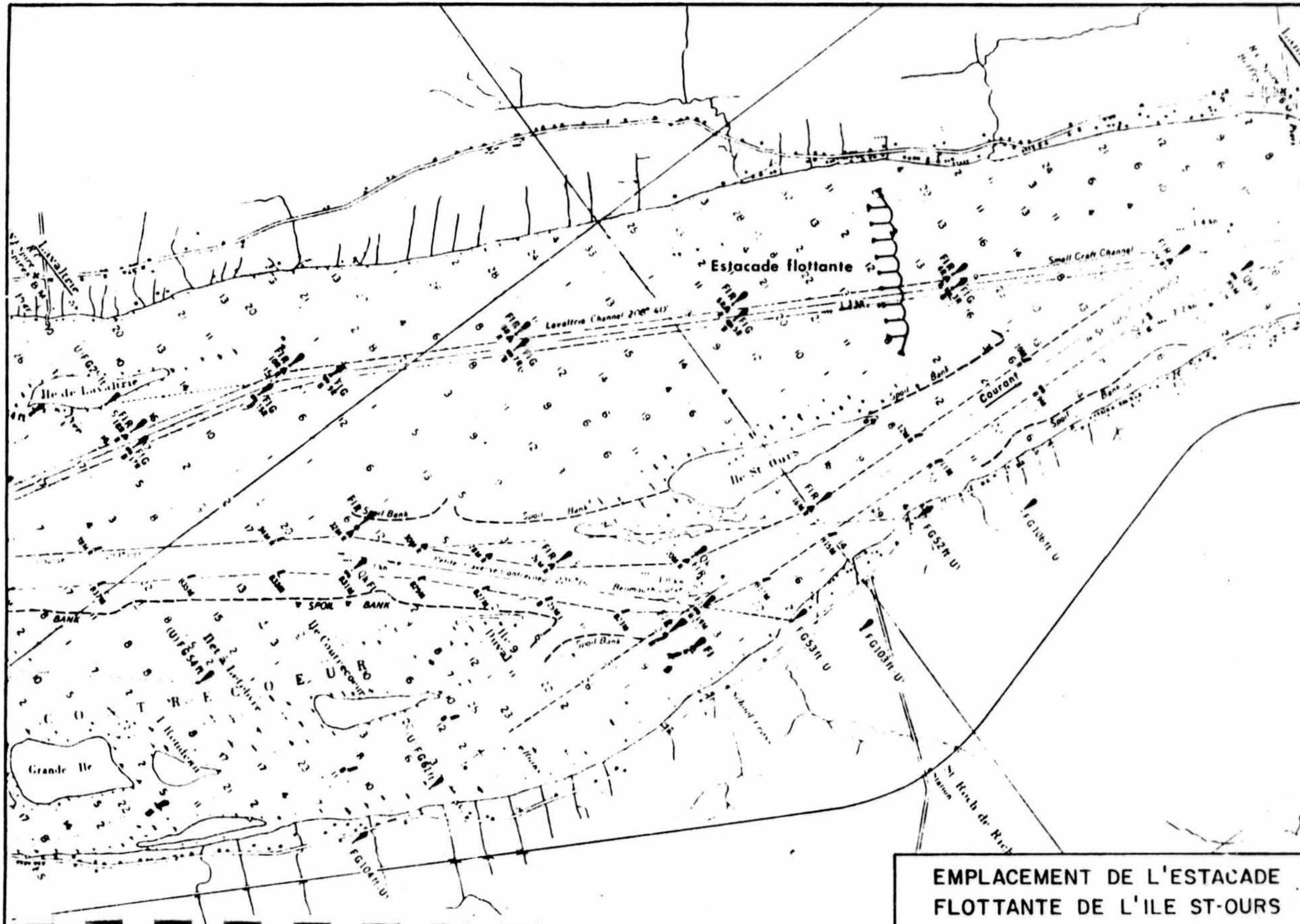
Description

L'estacade flottante de l'Ile St-Ours est du même type que celles installées sur le Lac St-Pierre et à Lavaltrie. Le fleuve à cet endroit a une largeur de 4,600 pieds. Une estacade de 3,200 pieds de longueur fut choisie, c'est-à-dire 8 sections de 500 pieds chacune.

Résultats obtenus

On ne peut que répéter ce qui a été dit pour les estacades du même type installées sur le Lac St-Pierre et à Lavaltrie, l'estacade de l'Ile St-Ours a fait sa part valable sur le total des efforts entrepris pour contrôler la glace, et justifie certainement l'opération annuelle de cette estacade. Les figures 20, 21 et 22 ne peuvent que prouver les succès remportés par ce genre d'installation.

Malgré tout, l'addition d'une section supplémentaire du côté nord de l'estacade serait souhaitable puisque cette surface ouverte a présenté des difficultés à geler lors de la deuxième année d'opération.



EMPLACEMENT DE L'ESTACADE
FLOTTANTE DE L'ILE ST-OURS

FIGURE

C) Les îles artificielles du Lac St-Pierre

Les 4 îles artificielles du Lac St-Pierre furent construites à titre expérimental à l'automne de 1967 et 1968, dans le but d'étudier les possibilités d'emplois de telles structures pour contrôler la glace sur le fleuve St-Laurent.

Elles furent intégrées à l'étude entreprise pour trouver des moyens de retenir le champ de glace du côté nord du Lac St-Pierre. Elles ont donc été disposées de façon à compléter le travail des estacades flottantes.

Emplacement

Les îles sont situées sur le côté nord du chenal maritime du St-Laurent. Trois de ces îles sont à environ 1,700 pieds de la bande nord du chenal navigable. L'autre sur laquelle fut construit un phare servant d'aide à la navigation a été construite à seulement 500 pieds de cette bande. L'île la plus en aval (île No 4) est à environ 6 milles 1/2 de Pointe du Lac. L'île suivante (île No 1) est à 1,500 pieds en amont et l'autre, qui suit (île No 2), est à un autre 4,500 pieds en amont. L'île qui sert d'aide à la navigation (île No 3) est située en face de l'ancrage de Yamachiche.

Description

Les îles furent construites à partir de matériel dragué dans le chenal maritime du St-Laurent et furent par la suite recouvertes avec de la pierre de carrière.

Les îles 1,2 et 4 ont une élévation de 19 pi. (IGLD) et leur surface supérieure est de 34 pi. x 34 pi. tandis que leur surface inférieure est de 140 pi. x 140 pi.

L'île 3 a une élévation de 25 pi. (IGLD). Sa surface supérieure est de 24 pi. x 24 pi. et sa surface inférieure est de 244 pi. x 244 pi.

Résultats obtenus

L'on peut maintenant affirmer hors de tout doute que les îles artificielles ont grandement contribué, en compagnie des estacades flottantes, à la formation et la retenue du champ de glace du côté nord du Lac St-Pierre. Les figures 16 (avant la construction) et 17 (après la construction) nous prouvent que les objectifs visés par la construction de ces îles sont maintenant atteints.

SURFACE WATER QUALITY

MANAGEMENT PROPOSAL

Volume 1: Surface Water Quality Objectives

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Safety and Health

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SUMMARY

The surface waters of Manitoba are used for numerous purposes including domestic consumption, industrial uses and agricultural purposes such as irrigation and livestock watering. In addition, many surface waters are used for recreational pursuits such as swimming, water skiing, boating and the enjoyment of pleasant scenery. Most waters are also inhabited by fish life, amphibians (frogs), reptiles (turtles), aquatic insects and algae. Large forms of wildlife, small furbearing mammals, water fowl and some birds of prey rely upon surface waters for drinking purposes, habitat and sources of food supplies.

The quality of surface water has the potential to become degraded through many other uses such as the disposal of industrial and municipal effluents, development of hydroelectrical generating sites and land use practices such as agriculture and forestry.

In order to achieve harmony between the various uses, surface water quality objectives were developed which define minimum levels of quality for each of the uses that requires protection. The objectives, when not exceeded, will protect an organism, a community of organisms, a prescribed water use, or a designated multiple purpose water use with an adequate degree of safety. Specific objectives have been developed for over eighty substances.

These objectives affect all Manitobans, since if they are under protective, surface water quality may become degraded, or if they are over protective, an unnecessary burden may be imposed on taxpayers and industry in order to pay for additional waste treatment facilities.

Surface water quality objectives are primarily used by government agencies, such as the Environmental Management Division and the Clean Environment Commission, in order to assist in developing effluent discharge restrictions for industrial and municipal waste discharges. In addition, other government agencies may use the objectives to control land use practices that may have effects on water quality, such as cottage development.

The objectives are also used by the Environmental Management Division, in combination with environmental monitoring programs, to determine if pollution control measures are successful in preventing water pollution. For example, the objectives may be used to determine if the waste treatment provided by a municipal sewage lagoon is successful in preventing water pollution of a fish spawning stream.

The objectives are also used by other government agencies to determine if certain waters are suitable for uses such as irrigation.

If the objectives are exceeded, there is no direct legal recourse to the source of the pollution. However, the Environmental Management Division may conduct the necessary studies in order to determine the cause of the pollution. Should the cause be waste effluents, Clean Environment Commission Orders may be reviewed and revised in order to provide the necessary protection.

Occasionally, water quality parameters may exceed these objectives due to natural conditions. In these cases, the objectives do not apply. It is the intent that these objectives are applicable to conditions in the water that are caused by man's activities. However, if a certain parameter exceeds the objectives due to natural conditions, it would be unwise to further increase that parameter by man-made activities.

It is important to realize that scientific information is limited on all the possible effects of a pollutant in the environment. New information, however, is continually being reported. Thus, the objectives must be revised periodically in order to include the most recent scientific knowledge. Based upon the available information, these objectives are designed to afford adequate protection without an unreasonable amount of over protection or under protection.

Thousands of substances could potentially pollute Manitoba's surface waters. These include, for example, agricultural chemicals, or hazardous goods that may be transported through Manitoba. Objectives have not been developed for all possible substances that could affect water quality. However, given reasonable information that such substances are present, objectives will be developed using the best available scientific information.

Because specific numerical objectives cannot reasonably be developed for every possible chemical, physical or biological parameter, general statements concerning environmental quality are also used to protect water quality. These requirements, although written in general terms, are nevertheless water quality objectives. For example, these may be used to establish effluent limits even though there may be no specific numerical objectives applicable in the receiving water. General statements have been developed for colour, odour, taste, turbidity, deposits, floating materials, flow, litter, nutrients, oil and grease and toxic substances.

Ideally, objectives should be maintained at all times. It is however, generally accepted that to require objective maintenance at all times is unreasonable. Thus, a specific low flow level has been chosen below which the objectives do not have to be met. This flow, for large streams and rivers, is the lowest flow which, on a statistical basis, would occur for a seven consecutive day period once every ten years. For small intermittent streams this minimum flow is $0.003 m^3/s$. The objectives should be maintained at all times in lakes.

Mixing zones are areas adjacent, for example, to a discharge, where the stream or lake may not meet all the water quality objectives. This is allowed for practical reasons, since for most contaminants, it would be unreasonable to expect the objectives to be met at the end of the discharge pipe. Mixing zones are therefore recognized as areas subject to a loss of value, but nevertheless, certain guidelines should be followed to ensure that the loss is kept as small as possible. These include, for example, ensuring that the entire width of rivers are not completely influenced by a discharge in such a manner that fish movement is prohibited or that bathing areas are not included in mixing zones.

Certain pristine waters support important major uses, such as recreation on surface waters within Provincial Parks. These waters may be given a "High Quality" designation. It is the intent that discharges or other activities that may affect the water quality of these areas should be very strictly controlled. Thus, development within "High Quality" surface water areas will likely be more costly that in other areas of the Province, since all available measures should be used to control environmental disturbances.

Some pristine waters of the Province may be preserved in their natural state for the future. These waters will be given an "Exceptional Value" designation. Development of any type that may affect water quality should be discouraged from these areas.

Objectives have been developed for each of the general surface water uses within Manitoba that requires protection. These are designated as classes and include domestic consumption, aquatic life and wildlife, industrial consumption, agricultural consumption, recreation and other uses. Where possible, these general classes are further divided into categories to provide protection, for example, to the different types of recreation.

CLASS 1: **DOMESTIC CONSUMPTION** will ensure the protection of waters that are suitable for human consumption after treatment. Since all surface waters of Manitoba are susceptible to uncontrolled microbiological contamination, for example, by wildlife, minimum treatment consisting of disinfection is required for all surface waters prior to consumption. Objectives are included for substances that may have harmful health effects, such as pesticides, toxic metals and radioactive materials and for substances that may present a nuisance to the consumer, such as excessive hardness and iron.

CLASS 2: **AQUATIC LIFE AND WILDLIFE** will ensure the protection of waters that are suitable for aquatic life such as fish, amphibians (frogs), reptiles (turtles) and other forms of life including aquatic insects and algae. By ensuring protection of the aquatic communities, protection is indirectly offered to those forms of wildlife that rely upon surface waters for habitat and for food supplies. These include ducks, geese, furbearing mammals such as the muskrat and birds of prey such as the eagle and osprey. Protection is also provided to those animals that use these waters for drinking purposes.

Objectives are included for numerous parameters including dissolved oxygen, toxic metals and pesticides. The presence of dissolved oxygen in water is essential for aquatic life, and the type of aquatic community is dependent to a large extent on the amount of dissolved oxygen present. Toxic metals, such as zinc and cadmium, in small concentrations, can have harmful effects on growth and reproduction, and in large concentrations, can be lethal. Others, such as mercury and PCB's, even though present in small quantities, can slowly bio-accumulate in the tissue of organisms, until higher harmful levels are reached.

Some metals, such as cadmium, are more or less toxic depending upon the hardness of the water. For this reason, a mathematical equation is used to establish an objective based upon the relationship between toxicity and hardness.

The existence and composition of an aquatic community also depends upon temperature characteristics. An excessive increase in temperature can be harmful by interfering with fish spawning cycles, causing changes in growth and respiration, and causing more heat tolerant species to replace heat sensitive ones. Heat related winter fish kills can occur when a heated discharge is suddenly stopped. Fish that have been attracted to a heated area are suddenly exposed to the cold ambient temperature.

Developing site-specific temperature objectives is complicated and time consuming. Therefore, a method is included by which temperature objectives will be developed for specific discharges.

CLASS 2: AQUATIC LIFE AND WILDLIFE is subdivided into two categories in order to provide specific protection to different general groups of aquatic life in Manitoba.

CATEGORY A: COLD WATER AQUATIC LIFE, COOL WATER AQUATIC LIFE AND WILDLIFE will provide protection to all types of aquatic life inhabiting the surface waters of Manitoba, including the protection of wildlife.

CATEGORY B: COOL WATER AQUATIC LIFE AND WILDLIFE will provide protection to cool water aquatic life such as walleye, sauger and pike, including the protection of wildlife. This category, however, will not provide adequate protection to cold water aquatic life such as trout and whitefish.

CLASS 3: INDUSTRIAL CONSUMPTION will ensure the protection of waters that are used for industrial purposes. However, objectives will not be developed at present due to the large number of present and potential industrial users, each with different quality requirements for water.

CLASS 4: AGRICULTURAL CONSUMPTION will provide protection to waters used by the market garden and farming industry for irrigation and livestock watering purposes. Objectives are included for parameters, such as sodium, that will protect various textured soils. Other objectives, such as boron, will protect sensitive plants. In addition, others, for example, fecal coliform bacteria are included that are intended to protect humans following consumption of raw vegetables irrigated with waters of this class.

This class is subdivided into four categories in order to provide protection to three different general irrigation practices plus to provide protection for livestock watering.

CATEGORY A: IRRIGATION will provide protection to waters that are used by the greenhouse industry where such water is the only source of moisture for the greenhouse plants.

CATEGORY B: IRRIGATION will provide protection to waters that are used to irrigate field crops, where such water is used to supplement natural rainfall.

CATEGORY C: IRRIGATION will provide protection to waters that are used to irrigate field crops, where such water is used to supplement natural rainfall. These waters, however, may damage certain soil types if used for long periods of time.

CATEGORY D: LIVESTOCK will provide protection to waters that are used by livestock.

CLASS 5: RECREATION will ensure that surface waters may be safely used for swimming and boating purposes and also may provide for the enjoyment of pleasant scenery. These waters provide outdoor recreational opportunities for both Manitoba residents and for tourists.

This class is further subdivided into two categories in order to provide protection to the different types of water related recreation depending upon the extent of contact with the water.

CATEGORY A: PRIMARY RECREATION will ensure the protection of waters that may be used for purposes such as swimming and water skiing, where contact with the water is an important aspect of the activity.

CATEGORY B: SECONDARY RECREATION will ensure the protection of waters that may be used for purposes such as fishing and boating, where contact with the water is only incidental to the activity.

Manitoba's surface waters may be used for other purposes that do not require protection through the establishment of objectives. These include, for example, the disposal of wastes or the generation of hydroelectrical power. Because of social or economic reasons, certain waters may be used only for these uses. Such waters may be given a **CLASS 6: OTHER USES** classification.

PREAMBLE

In 1976, the Environmental Management Division prepared a proposal outlining a system of surface water quality objectives and watershed classifications for the Province of Manitoba that would form the basis of a surface water quality management program. This proposal was critically reviewed by the Clean Environment Commission through public hearings held in 1977, under Section 13.1 of the Clean Environment Act. It was subsequently implemented with several revisions resulting from the public hearings.

Volume I, Surface Water Quality Objectives, herein, contains revised objectives which reflect current scientific knowledge and which delineate numerous changes in the water use classes and categories in order to better reflect the surface water use within Manitoba. Specific problem areas encountered in the original objectives have been further defined.

Volume 2, Watershed Classifications, under separate cover, contains a revised procedure for the application of surface water quality objectives. The surface water quality objectives will be appended for each of the nineteen watersheds within Manitoba as they become available. This document also contains a procedure for utilizing the surface water quality objectives, on an interim basis for watersheds that have not been classified.

The attached documents have been prepared by the Water Standards and Studies Section of the Environmental Management Division and have been reviewed by the Water Pollution Control Section of the Environmental Management Division, the Manitoba Departments of Agriculture, Municipal Affairs, Health, Urban Affairs, Economic Development and Tourism, Energy and Mines and the Parks, Wildlife, Resource Allocation, Fisheries and Water Resources Branches of the Manitoba Department of Natural Resources.

FOR FURTHER INFORMATION ON THE SURFACE WATER QUALITY OBJECTIVES OR WATERSHED CLASSIFICATIONS, PLEASE CONTACT:

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ABBREVIATIONS

(a) Bq/L = Becquerels per liter
(b) µg/L = micrograms per liter
(c) mg/L = milligrams per liter
(d) µS/cm = microsiemens per centimeter
(e) SAR = Sodium Adsorption Ratio

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1. INTRODUCTION

There are many diverse uses of surface waters within Manitoba, such as domestic, industrial and agricultural consumption, propagation and maintenance of aquatic life, wildlife, waterfowl, shorebirds and furbearing animals and recreation. These often require waters of differing physical, chemical and biological quality. These uses, plus others such as the disposal of wastes, generation of hydroelectric power, removal of excess precipitation, hydraulic alterations to natural watercourses and land use practises compete for the quality resources of provincial surface waters.

The acceptability of water is directly related to the needs of the user. Water containing a certain combination of constituents may be suitable for one use but may be totally unsuitable for another use.

The Environmental Management Division is striving to maintain, enhance and protect the chemical, physical and biological integrity of all surface waters within the Province of Manitoba. Achievement of this goal will ensure that the present and potential surface water uses are maintained in concordance with the social and economic development of the Province. To this end, surface water quality objectives were formulated which define minimum levels of quality required for the various uses. Provincial waters designated "High Quality" will be afforded greater protection and waters designated "Exceptional Value" will be maintained in their natural or non-degraded state.

If the objectives are under protective, surface water quality may become degraded, or if they are over protective, an unnecessary burden may be imposed on taxpayers and industry in order to pay for additional waste treatment facilities. These objectives, therefore, impact all Manitobans, since they may affect the operation of industries, municipalities and certain aspects of agriculture.

Figure 1: Pisew Falls, located on the Grass River, is one of Manitoba's many picturesque water-related sites.

Surface water quality objectives are designated concentrations of constituents that, when not exceeded, will protect an organism, a community of organisms, a prescribed water use, or a designated multiple purpose water use with an adequate degree of safety.

Where water quality characteristics could not be defined in scientifically defensible quantitative terms, general narrative statements were developed that reflected the necessary and desirable quality.

1.1 APPLICATION OF SURFACE WATER QUALITY OBJECTIVES

Surface water quality objectives are used as a management tool suited to protecting surface water quality. The objectives are used in conjunction with ambient monitoring data:

(a) to develop effluent discharge limitations for the protection of specific uses, through determining the capability of a receiving water course to assimilate waste contaminants. The assimilative capacity is usually the difference between objective levels and ambient levels, where the ambient levels are lower,

(b) to develop rational policies to guide those agencies having legislative authority, in co-operation with the Environmental Management Division, for projects involving resource apportionment, such as hydrological alterations and land use practises that may contribute to water quality deterioration. Such projects include, but are not limited to:

- (i) water flow augmentation,
- (ii) lake level regulation,
- (iii) water flow regulation,
- (iv) inter-basin water transfer,
- (v) extraction and apportionment of water for agriculture, municipal and industrial purposes,
- (vi) construction activities,
- (vii) resource harvesting or extraction operations (timber harvest, wild rice harvest, mineral exploration, mining, etc.),
- (viii) apportionment of crown, municipal, or private lands for recreational or other purposes through lease, sale, subdivision, etc.

(c) to develop best management practises to control non-point or diffuse sources of pollution,

(d) to assess the effectiveness of pollution control measures in protecting beneficial uses, and

(e) to identify if the ambient water can sustain specific uses.

Paramount among the above applications, is the utilization of the objectives for consideration by the Clean Environment Commission, in order to develop effluent discharge limitations necessary to make discharges compatible with specific surface water uses. The water quality objectives should not be construed as permitting any waste amenable to treatment or control to be discharged in any surface waters without treatment or control that could reasonably be expected.

If the water quality objectives are not met, there is no direct legal implication to the source of the contamination. Such a situation however, would indicate that administrative action is required to determine the cause and if remedial action is required. For example, remedial action may involve the review and revision of Clean Environment Commission orders, if point source discharges are the causative agents.

Utilization of surface water quality objectives is but one integral facet of surface water quality protection. Water quality objectives are used in conjunction with other provincial and federal guidelines and regulations on quantities, rates and concentrations of chemical, physical, biological and other constituents to which dischargers are subject.

1.2 NATURAL CHARACTERISTICS OUTSIDE THE OBJECTIVES

Waters may have, on occasion, natural characteristics outside the objectives in which case the objectives do not apply. The objectives contained herein apply to man-induced alterations. Withdrawal and subsequent discharge of such waters without alteration of the physical, chemical or biological characteristics into the same or similiar water body will not constitute violation of these objectives. The reduction in water quantity following withdrawal but prior to discharge however, should not cause exceedences of the general or specific surface water quality objectives such that other riparian uses may be adversely affected.

It should be noted that where the assimilative capacity is utilized by inferior natural quality, further deterioration by the introduction of contaminants should not be allowed, unless such additions will not jeopardise any beneficial use as shown through site specific investigations.

1.3 LIMITATIONS AND INTERPRETATION OF SURFACE WATER QUALITY OBJECTIVES

The surface water quality objectives contained herein are based upon current scientific knowledge. Thus, they must be reviewed and revised regularly to ensure that they reflect new information on criteria and limitations and that existing or potential uses are accurately identified.

There is a great deal of uncertainty of specific cause effect relationships between all concentrations of contaminants and all environmental variables. In addition, information is scarce on the antagonistic, synergistic and additive effects of combinations of contaminants.

The objectives necessary to protect aquatic life, for example, were adopted from criteria that were developed utilizing an array of data from organisms, both plant and animal, occupying various trophic levels. Others were adopted after the application of safety factors to a limited data base. These objectives then, are designed to protect most aquatic organisms most of the time, but not necessarily all organisms all of the time.

Similarly, water used for domestic consumption which contains substances at concentrations less than the maximum acceptable objectives should be suitable for lifelong consumption. These objectives were developed in consideration of other exposure routes, such as dietary intake.

These objectives are designed to afford adequate protection, without an unreasonable amount of over protection or under protection. Hence, adverse effects may be discernible should exceedences of the objectives be prolonged.

The objectives are expressed in terms of total concentrations of constituents in whole unfiltered water, except where otherwise specified and as maximum acceptable concentrations. Maximum acceptable concentrations should be construed to mean instantaneous maximum (or minimum) concentrations not to be exceeded at any time in any place. It should be noted that certain objectives are below the present technical detection capabilities of analytical instruments. This is justifiable, however, since these objectives are useful in calculating waste load limitations.

1.4 HIGH QUALITY WATERS

Waters designated as "High Quality" should have biological, chemical and physical quality better than the established objectives. Such waters should support a high quality beneficial use. The designation "High Quality" will be used in conjunction with the respective beneficial use class or category that is determined to be of high quality. Waters suitable for inclusion are as follows:

(a) waters that flow through or that are bounded by Provincial or National Parks,

(b) waters within relatively undisturbed watersheds,

(c) waters possessing outstanding quality characteristics,

(d) waters that support a diverse or unique flora and fauna which are sensitive to man-induced water quality alterations.

Measurable or calculable degradation should not occur as a result of human activity, that will jeopardize the designated high quality use unless:

(a) the proposed new, additional or increased discharge or discharges of pollutants is justified,

(b) such proposed discharges will not preclude any use presently possible in such waters and downstream from such waters, and will not result in exceedences of the water quality objectives, and

(c) any project or development which will result in new, additional or increased discharges of pollutants into such waters should be required to utilize the best available combination of treatment, land disposal, re-use and discharge technologies to control such wastes, including the use of best management practises to curb soil erosion.

1.5 EXCEPTIONAL VALUE WATERS

Any water whose quality is better than the established objectives and whose value as a resource for the support of a combination of aquatic life and wildlife and recreational uses is such that the waters are of exceptional recreational and ecological value will be given an "Exceptional Value" designation. Waters suitable for inclusion are as follows:

(a) wild and scenic rivers or lakes,

(b) waters or watersheds providing habitat for rare or endangered flora and fauna,

(c) waters considered sensitive such that irreversible harm will result following human impact,

(d) waters whose exceptional quality and value as a future resource precludes the assignment of present uses.

The above waters will be given a single "Exceptional Value" designation. This designation will be used to replace all other possible beneficial use designations. Water courses designated as such should not receive any alterations that result in measurable, calculable or perceived water quality degradation or degradation of other values deemed exceptional.

1.6 DEVELOPMENT OF SPECIFIC SURFACE WATER QUALITY OBJECTIVES

The list of specific water quality objectives does not include all possible substances that could affect water quality. Technology however, exists for the development of analytical procedures and for water quality objectives for all possible contaminants.

For example, over 600 substances are registered by Agriculture Canada as active ingredients within pest control products. These products are used for various purposes including wood preservatives, insecticides, herbicides, fungicides, materials preservatives, plant growth regulators, etc., and are available for use in Manitoba. In addition, thousands of other substances are recognized by the Environmental Management Division as hazardous goods, and are subject to manufacture, use, storage or transport within Manitoba. These products therefore, have the potential to contaminate surface waters. Analytical techniques and objectives will be developed given reasonable information that such substances are present.

For substances not listed, the general requirement from Section 1.7 that these should not be present in concentrations or combinations that injure, be toxic to, or produce adverse physiological or behavioral responses in humans, aquatic, semi-aquatic and terrestrial life should apply.

Specific numerical water quality objectives will be developed utilizing the best available scientific information. Objectives will be developed as follows for the protection of aquatic life:

1. Objectives will be developed utilizing the minimum data base concept published by the United States Environmental Protection Agency in the Federal Register, Vol. 45, No. 231, Friday, November 28, 1980, or subsequent similiar methods.

2. In those cases where it has been determined that there is insufficient available data to establish a safe concentration for a pollutant, the safe concentration value should be determined by applying the appropriate application factor to the 96 hr LC_{50} value. The 96 hr LC_{50} is defined as that concentration of a toxic material or materials which kills fifty percent of bioassay test organisms in ninety-six hours. If an experimentally derived application factor does not exist for a pollutant, the following values should be used in the determination of safe concentration values:

(a) concentrations should not exceed 0.2 of the lowest chronic dose level at which subtle and deleterious effects were noted,

(b) concentrations of pollutants that are non-persistent or have non-cumulative effects should not exceed 0.05 of the 96 hr LC_{50},

(c) concentrations of pollutants that are persistent or have accumulative effects should not exceed 0.01 of the 96 hr LC_{50},

(d) concentrations of pollutants with known synergistic or antagonistic effects with pollutants in the effluent or receiving water will be established on a case-by-case basis utilizing the best available scientific data.

Objectives for other uses will be developed utilizing the best available scientific information on exposure-response data, ingestion rates, risk extrapolation techniques, exposure from sources other than surface water, and appropriate application factors dependent upon the quantity and quality of data.

1.7 GENERAL REQUIREMENTS

Because specific numerical objectives cannot reasonably be developed for every possible chemical, physical or biological parameter, the following general statements are also used to protect water quality. These statements, although written in general terms, are nevertheless water quality objectives. For example, these may be used to establish effluent limits even though there may be no specific numerical objectives applicable in the receiving water.

All waters of the Province of Manitoba should be free of constituents attributable to sewage, industrial, agricultural and other land use practices, or other man-induced point or non-point source discharges such that the following general objectives are met as minimum conditions at all times and in all places:

Parameter, Substance, or Condition	Objectives
Colour, odour, taste, turbidity	Free from materials that produce colour, odour, taste, turbidity or other conditions in such a degree as to be objectionable or to impair any beneficial use.
Deposits	None that will cause the formation of putrescent or otherwise objectionable sludge deposits.
Floating materials	Free from floating debris, oil, scum, and other floating materials in sufficient amounts to be unsightly or deleterious.
Flow	Water quantities (flows and lake levels) should not be altered to a degree which will cause exceedences of the general or specific surface water quality objectives such that beneficial uses may be adversely affected.
Litter	Free from materials such as garbage, rubbish, trash, cans, bottles, or any unwanted or discarded solid material.
Nutrients	Nitrogen, phosphorus, carbon and contributing trace elements should be limited to the extent necessary to prevent the nuisance growth and reproduction of aquatic rooted, attached and floating plants, fungi or bacteria or to otherwise render the water unsuitable for other beneficial uses. For general guidance, unless it can be demonstrated that total phosphorus is not a limiting factor, considering the morphological, physical, chemical or other characteristics of the water body, total phosphorus should

not exceed 0.025 mg/L in any reservoir, lake, pond, or in a tributary at the point where it enters such bodies of water. In addition, total phosphorus should not exceed 0.05 mg/L in any stream except those identified in the immediately preceding statement. It should be noted that maintenance of such concentrations may not guarantee that eutrophication problems will not develop.

Oil and grease	Free from oil and grease residues which causes a visible film or sheen upon the waters or any discolouration of the surface of adjoining shorelines or causes a sludge or emulsion to be deposited beneath the surface of the water or upon the adjoining shorelines.
Toxic substances	Free from substances in concentrations or in combinations that injure, be toxic to, or produce adverse physiological or behavioral responses in humans, aquatic, semi-aquatic and terrestrial life.

1.8 MINIMUM AND MAXIMUM FLOWS AND LEVELS

Ideally, objectives should be maintained at all times. It is however, generally accepted that to require objective maintenance at all times is unreasonable. Thus, specific low flow levels have been chosen below which the objectives do not have to be met.

1.8.1 RIVERS AND STREAMS

The specific numerical water quality objectives should apply at all times except during periods when flows are less than the average minimum seven day flow which occurs once in ten years (Q_{7-10}). Should the average minimum seven day flow with a recurrence interval of once in ten years be $0.003m^3/s$ or less, then the criteria from the following Section 1.8.2, INTERMITTENT STREAMS should apply. In cases where the stream flow is highly regulated it may not be possible to calculate the average minimum seven day flow with a recurrence interval of once in ten years. In such instances the specific numerical objectives should apply for all periods above the minimum daily discharge for the period of record after the stream flow was altered.

The specific numerical objectives should, however, apply at all times if the beneficial uses are supported because of pooling of water during periods of low natural flows. The general requirements from the preceding Section 1.7 should apply at all times regardless of the amount of flow. Minimum and maximum stream flow criteria may be developed on a site specific basis should fluctuating stream flows influence water quality such that beneficial uses will be jeopardized.

1.8.2 INTERMITTENT STREAMS

Intermittent streams and natural or man-made drainage channels receive water from precipitation from small watersheds (usually less than 1 km^2 in area), and from ground water sources, hence usually flow during short periods. Such streams however, are an integral part of the surface water resources of the Province of Manitoba. The specific numerical water quality objectives should apply to all such streams when the flow is $0.003m^3/s$ or greater. When the intermittent stream does not contain this flow, the objectives to be maintained should be those pertaining to the water body to which the intermittent stream is tributary. The specific numerical objectives should however, apply at all times if the beneficial uses are supported because of pooling during periods of low natural flows. The general requirements from the preceding Section 1.7 should apply at all times regardless of the amount of flow. Minimum and maximum stream flow criteria may be developed on a site specific basis should fluctuating stream flows influence water quality such that beneficial uses will be jeopardized.

Figure 2: This tributary to the Shell River flows for only a short period of time each year, thus, is considered an intermittent stream.

Figure 3: Water for domestic consumption should be safe, palatable and aesthetically pleasing.

1.8.3 LAKES, BAYS AND IMPOUNDMENTS

The surface water quality objectives should apply at all times to lakes, bays and impoundments. Site specific water quality management strategies may be developed, delineating maximum and minimum levels, when natural or man-made lake level fluctuations cause water quality deterioration such that beneficial uses are jeopardized.

1.9 MIXING ZONES

Mixing zones are areas adjacent to a discharge or to an activity that may affect water quality where a receiving water may not meet all the water quality objectives. This is allowed for practical reasons, since for most contaminants, it would be unreasonable to expect the objectives to be met at the end of the discharge pipe. Wastes and water are given an area to mix such that the water quality objectives are met at the boundaries of the mixing zone. Mixing zones are recognized as areas subject to loss of value, however they should not be construed as a substitute for waste discharge treatment.

Mixing zones should be determined on a case-by-case basis utilizing a thorough knowledge of local conditions. Normally, geometric size constraints will not be assigned due to the complex nature of the mixing properties of liquids. The following guidelines should apply to mixing zones where applicable:

(a) the mixing zone should be as small as practicable and should not be of such size or shape as to cause or contribute to the impairment of water uses,
(b) the mixing zone should contain not more than 25% of the cross-sectional area/volume of flow at any transect in the receiving water during all flow regimes when the specific water quality objectives are applicable,
(c) the mixing zone should be designed to allow an adequate zone of passage for the movement or drift of all stages of aquatic life,

(d) mixing zones should not interfere with the migratory routes, natural movements, survival, reproduction, growth or increase the vulnerability to predation of any representative aquatic species,

(e) mixing zones should not interfere with spawning and nursery areas,

(f) when two or more mixing zones are in close proximity, they should be so defined that a continuous passageway for aquatic life is available,

(g) in lakes and other surface impoundments, the volume of mixing zones should not exceed 10% of the volume of that portion of the receiving waters available for mixing,

(h) mixing zones should not cause an irreversible organism response,

(i) mixing zones should not intersect the mouths' of rivers,

(j) the 96 hr LC_{50} for indigenous fish species should not be exceeded at any point in the mixing zones,

(k) mixing zones should not contaminate natural sediments so as to cause or contribute to exceedences of the water quality objectives outside the mixing zone,

(l) mixing zones should not intersect domestic water supply intakes or bathing areas,

(m) the general requirements from the preceding Section 1.7 should apply at all points within the mixing zones.

2. BENEFICIAL USES

The following sections are separated into classes that represent general surface water uses within Manitoba. These include domestic consumption, aquatic life and wildlife, industrial consumption, agricultural consumption. recreation and other uses. Where possible, these general classes are further divided into categories to provide protection, for example, to the different types of recreation.

2.1 CLASS 1: DOMESTIC CONSUMPTION

This class will ensure the protection of waters that are suitable for human consumption, culinary or food processing purposes, and other household purposes after treatment in order that the treated water will not exceed the maximum acceptable concentrations of the "Guidelines for Canadian Drinking Water Quality," 1978, published by Health and Welfare Canada, and any revisions, amendments or supplements, thereto.

The specific requirements listed hereunder are a combination of maximum acceptable concentrations and objective concentrations as set forth in the above mentioned guidelines and guidelines utilized by the Province of Manitoba. Inherent in these requirements is the necessity to disinfect all raw surface water supplies as minimum treatment prior to consumption, since all surface waters of Manitoba are susceptible to uncontrolled microbiological contamination, for example, by wildlife.

Some surface waters of the Province of Manitoba normally exceed several of these requirements due to natural or background occurences. Partial, complete or a combination of conventional and unit processes then become necessary in order to produce potable water from such raw water supplies. Specific requirements will not be developed for every possible combination of existing or available treatment processes.

Rather, it is the intent that man-induced water quality alterations not cause an unacceptable increased risk to public health or an unacceptable increased treatment cost to the water user or supplier. The following maximum acceptable objectives should be used, on a site specific basis, to assist in determining when increased health risks or increased treatment costs may be expected, in conjunction with information concerning:

(a) the chemical, physical or biological quality of the proposed discharge or alteration being considered,
(b) ambient or background surface water quality,
(c) design of downstream water treatment facilities,
(d) other pertinent information.

2.1.1 SPECIFIC REQUIREMENTS

2.1.1.1 RECOMMENDED LIMITS FOR PHYSICAL CHARACTERISTICS

Parameter	Maximum Acceptable
Colour	15 True Colour Units
Odour	Inoffensive
pH	6.5 - 8.5 pH Units
Taste	Inoffensive
Turbidity	5 Nephelometric Turbidity Units

2.1.1.2 RECOMMENDED LIMITS FOR CHEMICAL SUBSTANCES RELATED TO HEALTH

Substance[1]	Maximum Acceptable	
Inorganic		
Antimony	0.0002	mg/L
Arsenic	0.05	mg/L
Barium	1.0	mg/L
Boron	5.0	mg/L
Cadmium	0.005	mg/L
Chromium	0.05	mg/L
Cyanide (Free)	0.2	mg/L
Fluoride	1.5	mg/L
Lead	0.05	mg/L
Mercury	0.001	mg/L
Nitrate (as N)[2]	10.0	mg/l
Nitrite (as N)[2]	1.0	mg/L
Selenium	0.01	mg/L

Silver	0.05	mg/L
Sodium	400.	mg/L
Sulphate	500.	mg/L
Uranium	0.02	mg/L

Organic

Nitrilotriacetic Acid (NTA)	0.05	mg/L
Pesticides (Total)[3]	0.1	mg/L
Trihalomethanes (Total potential)[4]	0.35	mg/L

1. Unless otherwise stated the limits refer to the sum of all forms present.

2. Where both nitrate and nitrite are present, the total nitrate, plus nitrite-nitrogen should not exceed 10 mg/L.

3. For maximum acceptable concentrations of individual pesticides see Section 2.1.1.3.

4. Comprise chloroform, bromodichloromethane, chlorodibromomethane and bromoform.

2.1.1.3 RECOMMENDED LIMITS FOR PESTICIDES

Pesticide[1]	Maximum Acceptable	
Aldrin and Dieldrin	0.0007	mg/L
Carbaryl	0.07	mg/L
Chlordane (Total Isomers)	0.007	mg/L

DDT (Total Isomers)	0.03	mg/L
Diazinon	0.014	mg/L
Endrin	0.0002	mg/L
Heptachlor and Heptachlor Epoxide	0.003	mg/L
Lindane	0.004	mg/L
Methoxychlor	0.1	mg/L
Methyl Parathion	0.007	mg/L
Parathion	0.035	mg/L
Toxaphene	0.005	mg/L
2, 4-D	0.1	mg/L
2, 4, 5 - TP	0.01	mg/L
Total Pesticides[2]	0.1	mg/L

1. The limits for each pesticide refer t[illegible] [illegible]e sum of all forms present.

2. The "Total Pesticides" limit applies to [illegible]r in which more than one of the above pesticides is present, [illegible] [illegible]ich case, the sum of their concentrations should not exceed 0.[illegible] [illegible]L.

2.1.1.4 RECOMMENDED LIMITS FOR SUBSTANCES RELATE[illegible] [illegible] [illegible]STHETIC AND OTHER CONSIDERATIONS

Contaminant	Maximum Acc[illegible]le	
Chloride	250.	[illegible]
Copper	1.0	[illegible]
Hardness (as $CaCO_3$)	200.	[illegible]
Hydrogen Sulfide (as H_2S)	0.05	[illegible]

Iron	0.3	mg/L
Manganese	0.05	mg/L
Phenols	0.002	mg/L
Total Dissolved Solids	500.	mg/L
Zinc	5.0	mg/L

2.1.1.5 RECOMMENDED LIMITS FOR RADIONUCLIDES

Radionuclides	Maximum Acceptable	
Cesium - 137	50	Bq/L
Iodine - 131	10	Bq/L
Radium - 226	1	Bq/L
Strontium - 90	10	Bq/L
Tritium	40000	Bq/L

Where two or more radionuclides affecting the same organ or tissue are found to be present, the following relationship, based on the International Commission on Radiological Protection publication 26, should be satisfied:

$$\frac{c_1}{C_1} + \frac{c_2}{C_2} + \ldots \frac{c_i}{C_i} \leq 1$$

Where c_1, c_2, c_i are the observed concentrations, and C_1, C_2, C_i are the maximum acceptable concentrations for each contributing radionuclide.

2.1.1.6 RECOMMENDED LIMITS FOR MICROBIOLOGICAL CHARACTERISTICS

Indicator Organism	Maximum Acceptable
Fecal coliform bacteria	90 percentile: 10 organisms/100mL
Total coliform bacteria	90 percentile: 100 organisms/100mL

2.2 CLASS 2: AQUATIC LIFE AND WILDLIFE

2.2.1 CATEGORY A: COLD WATER AQUATIC LIFE, COOL WATER AQUATIC LIFE AND WILDLIFE

This category will ensure the passage, maintenance and propagation of fish species including the family Salmonidae (char, trout, whitefish, grayling) and additional flora and fauna which are indigenous to a cold water habitat. This category will also ensure the passage, maintenance and propagation of fish species and additional flora and fauna which are indigenous to a cool water habitat (mooneye, goldeye, pike, perch, walleye, sauger). Additional flora and fauna includes other aquatic organisms but not limited to bacteria, fungi, algae, aquatic insects, other aquatic invertebrates, reptiles, amphibians and fishes.

By ensuring protection of the aquatic communities, protection is indirectly offered to those forms of wildlife that rely upon surface waters for habitat and for food supplies.

This category will therefore ensure the protection of streams, lakes, marshes, swamps, lowlands, etc. which are satisfactory as habitat for aquatic and semi-aquatic wild animals, such as waterfowl, shorebirds, furbearing mammals and other water oriented wildlife including the necessary aquatic organisms in their food chain. Protection of waters suitable for watering wild animals will be provided.

Objectives are included for numerous parameters including dissolved oxygen, toxic metals and pesticides. The presence of dissolved oxygen in water is essential for aquatic life, and the type of aquatic community is dependent to a large extent on the amount of dissolved oxygen present. Toxic metals, such as zinc and cadmium, in small concentrations, can have harmful effects on growth and reproduction, and in large concentrations, can be lethal. Others, such as mercury and PCB's, even though present in small quantities, can slowly bio-accumulate in the tissue of organisms, until higher harmful levels are reached.

Some metals, such as cadmium, are more or less toxic depending upon the hardness of the water. For this reason, a mathematical equation is used to establish an objective based upon the relationship between toxicity and hardness.

The existence and composition of an aquatic community also depends upon temperature characteristics. An excessive increase in temperature can be harmful by interfering with fish spawning cycles, causing changes in growth and respiration, and causing more heat tolerant species to replace heat sensitive ones. Heat related winter fish kills can occur when a heated discharge is suddenly stopped. Fish that have been attracted to a heated area are suddenly exposed to the cold ambient temperature.

Developing site-specific temperature objectives is complicated and time consuming. Therefore, a method is included by which temperature objectives will be developed for specific discharges.

2.2.1.1 SPECIFIC REQUIREMENTS

Parameter	Maximum Acceptable Concentration	Reference
Ammonia (un-ionized, NH_3)	0.02* mg/L	EPA (1976)

*the percent un-ionized ammonia can be
calculated for any temperature and pH by using

the following formula taken from Thurston, R.V., et al, 1974. Aqueous ammonia equilibrium calculations. Technical report number 74-1, Fisheries Bioassay Laboratory, Montana State University, Bozeman, Montana, 18 pages.

$$f = \frac{1}{10^{(pKa - pH)} + 1} \times 100$$

where:

f = the percent of the total ammonia in the un-ionized state,

pKa (dissociation constant for ammonia) =

$$0.0901821 + \frac{2729.92}{T}$$

T = temperature in degrees Kelvin
(273.16° Kelvin = 0° Celsius)

Arsenic (Total)	0.05 mg/L	Environment Canada (1979)
Cadmium (Total)	$e^{(1.05 (\ln (hardness))-8.53)}$, where hardness is expressed in mg/L as $CaCO_3$ (eg) 50 mg/L $CaCO_3$ ≡ 0.012 µg/L 100 mg/L $CaCO_3$ ≡ 0.025 µg/L 200 mg/L $CaCO_3$ ≡ 0.051 µg/L	EPA (1980)
Chlordane	0.0043 µg/L	EPA (1980)
Chlorine (Total residual)	0.002 mg/L	EPA (1976)
Chromium (Total hexavalent)	0.29 µg/L	EPA (1980)
Copper (Total)	5.6 µg/L	EPA (1980)
Cyanide, Free (sum of HCN and CN^-, expressed as CN)	3.5 µg/L	EPA (1980)
DDT and metabolites	0.0010 µg/L	EPA (1980)
Dieldrin	0.0019 µg/L	EPA (1980)
Dissolved oxygen	60% saturation (instantaneous minimum)	Davis (1975)

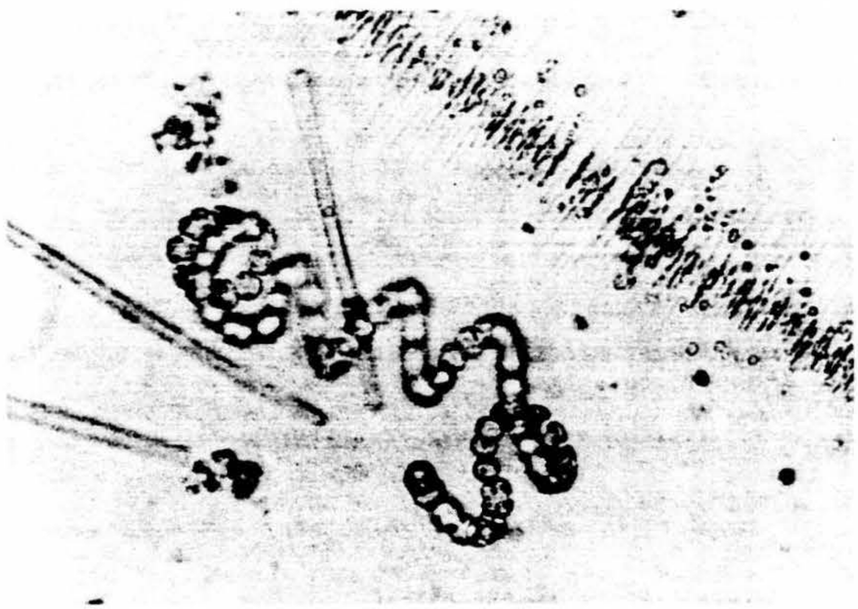

Figure 4: These various species of algae form an important link in the food chain of higher organisms, such as fish. However, given the necessary enriched conditions, algae may proliferate until nuisance conditions are reached.

Figure 5: Manitoba's surface waters abound with species of fish such as this pearl dace. Larger forage fish, such as walleye, rely upon these plus many other species as a source of food supply.

Endosulfan	0.056 µg/L	EPA (1980)
Endrin	0.0023 µg/L	EPA (1980)
Heptachlor	0.0038 µg/L	EPA (1980)
Hexachlorobutadiene (HCBD)	0.1 µg/L	Environment Canada (1983)
Hydrogen Sulphide (H_2S)	0.002 mg/L	EPA (1976)
Iron (Total)	1.0 mg/L	EPA (1976)
Lead (Total)	$e^{(2.35 (\ln (\text{hardness}))- 9.48)}$, where hardness is expressed in mg/L as $CaCO_3$ (eg)50 mg/L $CaCO_3$ ≡ 0.75 µg/L 100 mg/L $CaCO_3$ ≡ 3.8 µg/L 200 mg/L $CaCO_3$ ≡ 20.0 µg/L	EPA (1980)
Lindane	0.080 µg/L	EPA (1980)
Mercury (Total)	0.00057 µg/L	EPA (1980)
Nickel (Total)	$e^{(0.76 (\ln(\text{hardness}))+ 1.06)}$, where hardness is expressed in mg/L as $CaCO_3$ (eg)50 mg/L $CaCO_3$ ≡ 56 µg/L 100 mg/L $CaCO_3$ ≡ 96 µg/L 200 mg/L $CaCO_3$ ≡ 160 µg/L	EPA (1980)
Non-filterable residue	25 mg/L	Alabaster & Lloyd (1982)
pH	6.5 - 9.0 pH units	Alabaster & Lloyd (1982)
Phenols	1.0 µg/L	EPA (1976)
Phthalic acid esters		
(i) Dibutylphthalate (DBP)	4.0 µg/L	Environment Canada (1983)
(ii) Di-(2-ethylhexyl) phthalate (DEHP)	0.6 µg/L	Environment Canada (1983)
(iii) Other phthalates	0.2 µg/L	Environment Canada (1983)
Polychlorinated biphenyls	0.014 µg/L	EPA (1980)

Selenium (Total selenite)	35 μg/L	EPA (1980)
Silver (Total)	0.1 μg/L	Environment Canada (1979)
Temperature	Site specific objectives will be developed considering the following:	EPA (1976)

1. Thermal additions should be such that thermal stratification and subsequent turnover dates are not altered from those existing prior to the addition of heat from artificial origin.

2. One limit which consists of a maximum weekly average temperature that:
   (a) In the warmer months is determined by adding to the physiological optimum temperature (usually for growth) a factor calculated as one-third of the difference between the ultimate upper incipient lethal temperature and the optimum temperature for the most sensitive important species (and appropriate life stages) that normally is found at that location and time,

   (b) in the colder months is an elevated temperature that would still ensure that important species would survive if the temperature suddenly dropped to the normal ambient temperature, or

   (c) during reproduction seasons meets specific site requirements for successful migration, spawning, egg incubation, and other reproductive functions of important species, or

   (d) at a specific site is found necessary to preserve normal species diversity or prevent undesirable growths of nuisance organisms.

3. A second limit which is the time-dependent maximum temperature for short exposures.

Figure 6: Amphibians, such as this leopard frog, rely upon surface waters for habitat and for sources of food supplies.

Figure 7: Canada geese are often seen in association with Manitoba's lakes and marshes.

4. Maximum limits may be specified for incremental temperature fluctuations necessary to protect aquatic life from sudden temperature changes.

Toxaphene	0.013 µg/L	EPA (1980)
Zinc (Total)	47 µg/L	EPA (1980)

2.2.2 CATEGORY B: COOL WATER AQUATIC LIFE AND WILDLIFE

This category will ensure the passage, maintenance and propagation of fish species and additional flora and fauna which are indigenous to a cool water habitat (mooneye, goldeye, pike, perch, walleye, sauger). This category also includes other aquatic organisms but not limited to bacteria, fungi, algae, aquatic insects, other aquatic invertebrates, reptiles, amphibians and fishes that are indigenous to a cool water habitat.

This category will also ensure the protection of streams, lakes, marshes, swamps, lowlands, etc. which are satisfactory as habitat for aquatic and semi-aquatic wild animals, such as waterfowl, shorebirds, furbearing mammals and other water oriented wildlife including the necessary aquatic organisms in their food chain. Protection of waters suitable for watering wild animals is provided.

2.2.2.1 SPECIFIC REQUIREMENTS

All parameters and limitations from Section 2.2.1.1 should apply except as follows:

Parameter	Maximum Acceptable Concentration	Reference
Chlorine (Total residual)	0.01 mg/L	EPA (1976)
Dissolved oxygen	47% saturation (instantaneous minimum)	Davis (1975)

2.3 CLASS 3: INDUSTRIAL CONSUMPTION

This class will ensure the protection of all waters which are or may be used as a source of supply for industrial processes or cooling water, or any other industrial or commercial purposes and for which quality control is or may be necessary.

Discharges or alterations to waters classified as CLASS 3: INDUSTRIAL CONSUMPTION, should not be permitted such that downstream present or potential industrial users will incur unacceptable additional treatment costs.

2.3.1 SPECIFIC REQUIREMENTS

Selective limits will be imposed for any specific water as required. Objectives will not be formulated at present due to the large number of present and potential industrial water users, each with varying requirements for quality control of water.

2.4 CLASS 4: AGRICULTURAL CONSUMPTION

2.4.1 CATEGORY A: IRRIGATION (Sole Source of Water)

This category will ensure the long term protection of fine, medium and coarse textured soils from the accumulation of substances that may be harmful or cause a reduction in fertility; will ensure the protection of sensitive, semi-tolerant and tolerant species of plants; and will ensure the protection of humans from the harmful effects caused by the accumulation of substances on marketable produce that may not be processed prior to consumption. This category will provide protection for intensive horticultural crop production, where irrigation is used as the only source of water.

Figure 8: The La Salle River is used as a source of irrigation water for vegetable crops.

Figure 9: Many streams, such as the Shell River, are used for watering livestock. It is however, considered environmentally unacceptable to allow livestock direct access to the river.

Objectives are included for parameters such as sodium, that will protect various textured soils. Other objectives, such as boron, will protect sensitive plants. In addition, others, for example, fecal coliform bacteria are included that are intended to protect humans following consumption of raw vegetables irrigated with water of this class.

2.4.1.1 SPECIFIC REQUIREMENTS

Parameter	Maximum Acceptable Concentration	Reference
Aluminum (Total)	5.0 mg/L	NAS/NAE (1973)
Arsenic (Total)	0.1 mg/L	Environment Canada (1979)
Beryllium (Total)	0.1 mg/L	NAS/NAE (1973)
Boron (Total)	0.5 mg/L	Best professional, judgement (Manitoba Agriculture)
Cadmium (Total)	0.01 mg/L	Environment Canada (1979)
Chloride (Soluble)	68 mg/L	Best professional judgement (Manitoba Agriculture)
Chromium (Total)	0.1 mg/L	Environment Canada (1979)
Cobalt (Total)	0.05 mg/L	NAS/NAE (1973)
Conductivity	1000 µS/cm	Best professional judgement (Manitoba Agriculture)
Copper (Total)	0.2 mg/L	Environment Canada (1979)
Fecal coliform bacteria	(a) geometric mean: 1000 organisms /100 mL (b) individual maximum: 2000 organisms/100 mL (sufficient samples should be collected in order to permit valid interpretation).	NAS/NAE (1973)

	Should contact with the irrigation water by field staff be probable, the fecal coliform bacteria characteristics from the following Section 2.5, CLASS 5: RECREATION, CATEGORY A: PRIMARY RECREATION should apply.	
Filterable residue	700 mg/L	Best professional judgement (Manitoba Agriculture)
Fluoride (Total)	1.0 mg/L	NAS/NAE (1973)
Hydrogen sulfide (H_2S)	3.0 mg/L	Best professional judgement (Manitoba Agriculture)
Iron (Total)	5.0 mg/L	NAS/NAE (1973)
Lead (Total)	5.0 mg/L	Environment Canada (1979)
Lithium (Total)	2.5 mg/L	NAS/NAE (1973)
Manganese (Total)	0.2 mg/L	NAS/NAE (1973)
Molybdenum (Total)	0.01 mg/L	NAS/NAE (1973)
Nickel (Total)	0.2 mg/L	Environment Canada (1979)
pH	6.0-8.5 pH units	Best professional judgement (Manitoba Agriculture)
Radionuclides	The limits from the preceding Section 2.1.1.5, RECOMMENDED LIMITS FOR RADIONUCLIDES should apply.	Health and Welfare Canada (1978)
Selenium (Total)	0.02 mg/L	Environment Canada (1979)
Sodium	(a) 20 mg/L	Best professional judgement (Manitoba Agriculture)
	(b) 4.0 Sodium Adsorption Ratio (SAR)	NAS/NAE (1973)

$$SAR = \frac{.043\ Na^{+}}{\sqrt{.025\ Ca^{++} + .04\ Mg^{++}}},$$

expressed in mg/L

Sulfates	250 mg/L	Best professional judgement (Manitoba Agriculture)
Vanadium (Total)	0.1 mg/L	NAS/NAE (1973)
Zinc (Total)	2.0 mg/L	NAS/NAE (1973)

2.4.2 CATEGORY B: IRRIGATION (Supplemental Source of Water)

This category will ensure the long term protection of fine, medium and coarse textured soils from the accumulation of substances that may be harmful or cause a reduction in fertility; will ensure the protection of sensitive, semi-tolerant and tolerant species of plants; and will ensure the protection of humans from harmful effects caused by the accumulation of substances on marketable produce that may not be processed prior to consumption. This category will provide protection for field crop production where irrigation water is used to supplement natural precipitation. This category may be applicable only during the irrigation season.

2.4.2.1 SPECIFIC REQUIREMENTS

The maximum acceptable concentrations for parameters listed in the preceding Section 2.4.2.1 should apply with the following exceptions:

Parameter	Maximum Acceptable Concentration	Reference
Boron (Total)	1.0 mg/L	Best professional judgement (Manitoba Agriculture)
Chloride (Soluble)	150 mg/L	Best professional judgement (Manitoba Agriculture)

Conductivity	1,500 µS/cm	Best professional judgement (Manitoba Agriculture)
Filterable residue	1,050 mg/L	Best professional judgement (Manitoba Agriculture)
pH	5.0-9.0 pH units	Best professional judgement (Manitoba Agriculture)
Sodium	6.0 Sodium Adsorption Ratio (SAR)	NAS/NAE (1973)

$$SAR = \frac{.043\ Na^{+}}{\sqrt{.025\ Ca^{++} + .04\ Mg^{++}}},$$

expressed in mg/L

2.4.3 CATEGORY C: IRRIGATION (Qualified Use of Water)

This category will ensure the long term protection of coarse soils, protection up to twenty years of medium to fine textured soils, and short term protection of fine textured soils from the accumulation of substances that may be harmful or cause a reduction in fertility; will ensure the protection of sensitive, semi-tolerant and tolerant species of plants; and will ensure the protection of humans from harmful effects caused by the accumulation of substances on marketable produce that may not be processed prior to consumption. This category will provide protection for permanent irrigation installations on coarse soils and for temporary irrigation installations on medium to fine soils where irrigation water is used to supplement natural precipitation. This category may be applicable only during the irrigation season.

2.4.3.1 SPECIFIC REQUIREMENTS

The maximum acceptable concentrations for parameters listed in the preceding Section 2.4.2.1 should apply with the following exceptions:

Parameter	Maximum Acceptable Concentration	Reference
Aluminum (Total)	20.0 mg/L	NAS/NAE (1973)
Arsenic (Total)	2.0 mg/L	NAS/NAE (1973)
Beryllium (Total)	0.5 mg/L	NAS/NAE (1973)
Boron (Total)	2.0 mg/L	NAS/NAE (1973)
Chromium (Total)	1.0 mg/L	NAS/NAE (1973)
Cobalt (Total)	5.0 mg/L	NAS/NAE (1973)
Conductivity	2000 µS/cm	Best professional judgement (Manitoba Agriculture)
Copper (Total)	5.0 mg/L	Environment Canada (1979)
Filterable residue	1400 mg/L	Best professional judgement (Manitoba Agriculture)
Fluoride (Total)	15.0 mg/L	NAS/NAE (1973)
Iron (Total)	20.0 mg/L	NAS/NAE (1973)
Lead (Total)	10.0 mg/L	Environment Canada (1979)
Manganese (Total)	10.0 mg/L	NAS/NAE (1973)
Nickel (Total)	2.0 mg/L	Environment Canada (1979)
Selenium (Total)	0.05 mg/L	Environment Canada (1979)
Sodium	8.0 Sodium Adsorption Ratio (SAR)	NAS/NAE (1973)

$$SAR = \frac{.043\ Na^{+}}{\sqrt{.025\ Ca^{++} + .04\ Mg^{++}}},$$

expressed in mg/L

Parameter	Maximum Acceptable Concentration	Reference
Vanadium (Total)	1.0 mg/L	NAS/NAE (1973)
Zinc (Total)	10.0 mg/L	NAS/NAE (1973)

2.4.4 CATEGORY D: LIVESTOCK

This category will ensure the protection of all classes and ages of livestock and poultry from inhibitory effects following water consumption. Adjustment problems, such as mild and temporary diarrhea, may result when the livestock are introduced to the water, but should not affect their health or performance. Disinfection may be required for waters heavily contaminated with wastes of fecal origin in order to provide a suitable supply for ingestion by monogastric animals (poultry, swine, horses).

2.4.4.1 SPECIFIC REQUIREMENTS

Parameter	Maximum Acceptable Concentration	Reference
Alkalinity (Total, as $CaCO_3$)[1]	1000 mg/L	Manitoba Agriculture
Aluminum (Total)	5.0 mg/L	NAS/NAE (1973)
Arsenic (Total)	0.5 mg/L	Environment Canada (1979)
Boron (Total)	5.0 mg/L	NAS/NAE (1973)
Cadmium (Total)	0.02 mg/L	Environment Canada (1979)
Chromium (Total)	1.0 mg/L	Environment Canada (1979)
Cobalt (Total)	1.0 mg/L	NAS/NAE (1973)
Conductivity	4300 µS/cm	Manitoba Agriculture
Copper (Total)	1.0 mg/L	Environment Canada (1979)
Dissolved oxygen	maintain aerobic conditions	NAS/NAE (1973)
Filterable residue	3000 mg/L	NAS/NAE (1973)
Fluoride (Total)	2.0 mg/L	NAS/NAE (1973)
Lead (Total)	0.5 mg/L	Environment Canada (1979)

Mercury (Total)	0.003 mg/L	Environment Canada (1979)
Nickel (Total)	5.0 mg/L	Environment Canada (1979)
Nitrate - Nitrite (NO_3 - NO_2)	10.0 mg/L	Best professional judgement (Manitoba Agriculture)
pH	5.5-9.0 pH units	Manitoba Agriculture
Radionuclides	The limits from the preceding Section 2.1.1.5, RECOMMENDED LIMITS FOR RADIONUCLIDES, should apply.	Health and Welfare Canada (1978)
Selenium (Total)	0.05 mg/L	Environment Canada (1979)
Sulfate (SO_4)[1]	1000 mg/L	Manitoba Agriculture
Toxic algae	Waters bearing heavy growths of blue green algae should be avoided.	Manitoba Agriculture
Vanadium (Total)	0.1 mg/L	NAS/NAE (1973)
Zinc (Total)	50.0 mg/L	Environment Canada (1979)

1. Alkalinity (Total, as $CaCO_3$) plus Sulfate (SO_4) should not exceed 1000 mg/L (Best professional judgement, Manitoba Agriculture).

2.5 CLASS 5: RECREATION

Surface waters provide outdoor recreational opportunities for both Manitoba residents and tourists. This class will ensure that such waters may be safely used for swimming and boating purposes and also may provide for the enjoyment of pleasant scenery.

2.5.1 CATEGORY A: PRIMARY RECREATION

This category will ensure the protection of waters which are suitable for primary recreational uses where the human body may come in direct contact with the water, to the point that water may be ingested accidently or water may contact certain sensitive organs such as the eyes, ears and nose. Examples could include wading and dabbling, swimming, diving, water skiing, surfing and intimate contact with water directly associated with shoreline activities. Due to climatic conditions in Manitoba, primary recreation is usually restricted to the period from May 1 to September 30 of the same year.

2.5.1.1 SPECIFIC REQUIREMENTS

Parameter	Maximum Acceptable Concentration	Reference
Clarity (Secchi disc visibility)	1.2 meters (minimum)	Health and Welfare Canada (1983)
Dissolved oxygen	maintain aerobic conditions	Best professional judgement (Environmental Management Division)
Fecal coliform bacteria	(a) geometric mean: 200 organisms /100 mL (b) individual maximum: 400 organisms/100 mL (sufficient samples should be collected in order to permit valid interpretation)	Health and Welfare Canada (1983)
pH	6.5-8.5 pH units	Health and Welfare Canada (1983)
Turbidity	50 Jackson Turbidity Units	Health and Welfare Canada (1983)

2.5.2 CATEGORY B: SECONDARY RECREATION

This category will provide protection to waters which are suitable for boating, fishing and other water related activities other than immersion recreation, including navigation and aesthetic enjoyment of scenery. This category includes activities in which water use is incidental, accidental or sensory, and includes fishing, boating, camping, hunting and hiking.

Figure 10: Manitoba's surface waters provide an aesthetically pleasing setting for the enjoyment of secondary recreational pursuits, such as angling.

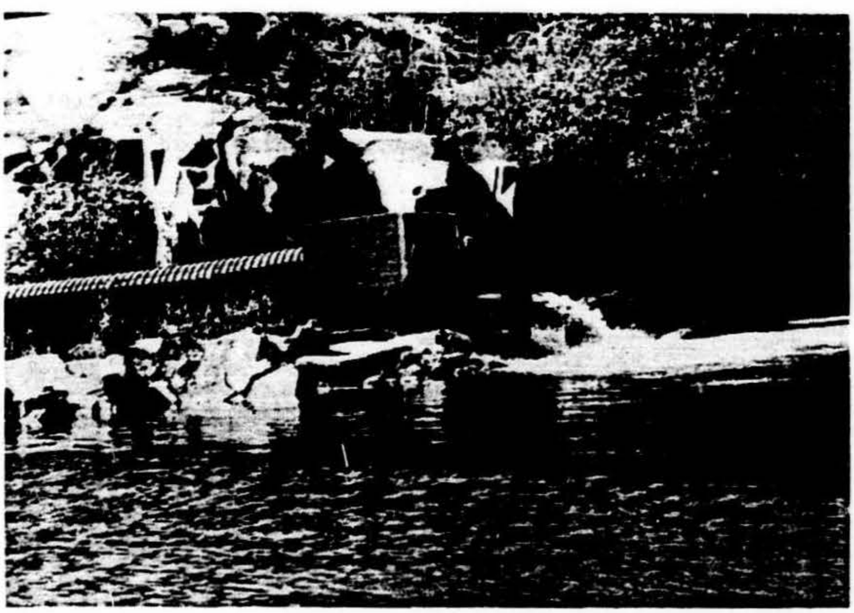

Figure 11: The assimilative capacity of streams, such as the Winnipeg River, is used to dispose of liquid waste effluents.

2.5.2.1 SPECIFIC REQUIREMENTS

Parameter	Maximum Acceptable Concentration	Reference
Dissolved oxygen	maintain aerobic conditions	EPA (1976)
Fecal coliform bacteria	(a) geometric mean: 1000 organisms /100 mL (b) individual maximum: 2000 organisms/100 mL (sufficient samples should be collected in order to permit valid interpretation).	Best professional judgement (Environmental Management Division)
Navigational hazards	There should be no unmarked submergent or emergent obstructions that pose a threat to navigation.	Best professional judgement (Environmental Management Division)
Temperature	There should be no temperature changes that interfere with the natural freezing patterns or dates that pose a threat to navigation.	Best professional judgement (Environmental Management Division)

2.6 CLASS 6: OTHER USES

The uses to be protected in this class may be under other jurisdictions and in other areas to which the waters of the Province are tributary and may include any or all of the uses listed in the preceding categories, plus any other possible beneficial uses. The Environmental Management Division, therefore, reserves the right to designate any objective necessary for the protection of this class, consistent with legal limitations.

The Environmental Management Division is also cognizant of the fact that other uses of streams, rivers or lakes may also include the removal of excess precipitation, production of hydroelectric power or disposal of wastes. These are beneficial uses, however, they do not require protection through the designation of objectives. These uses, however may be chosen on the basis of social or economic conditions to take precedence over the other preceding designated beneficial uses.

3. REFERENCES

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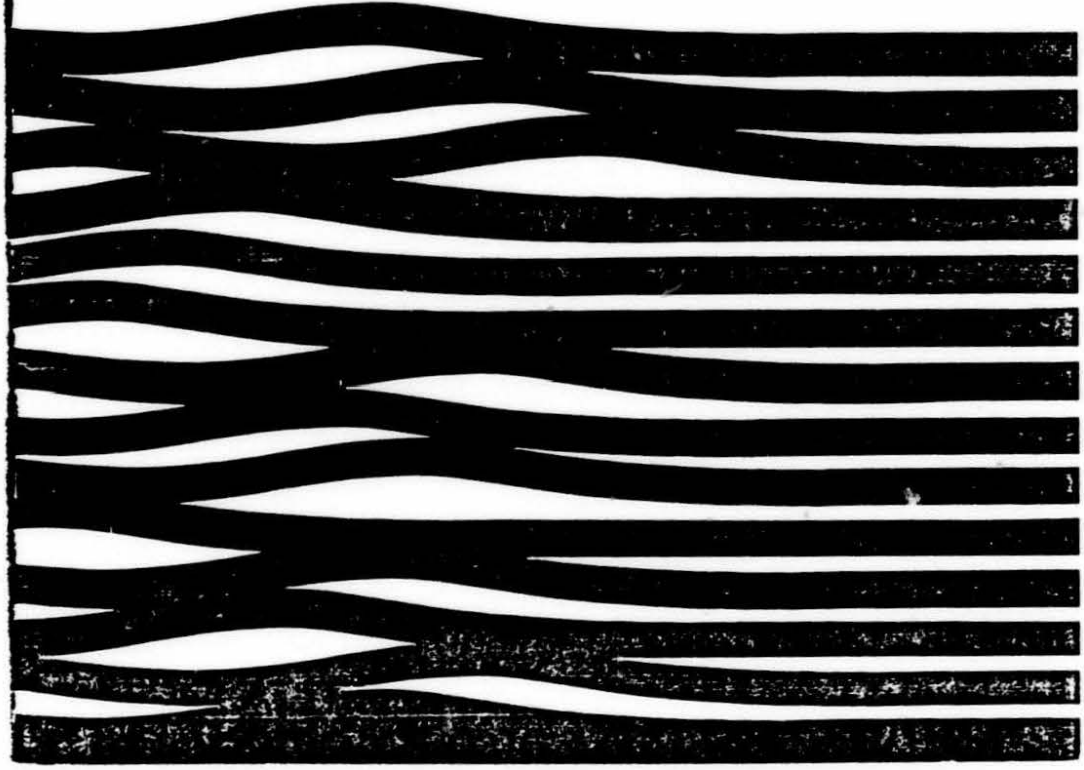
APPENDIX 1

17
20
16
(Mudry)

Red River Floodway
City of Winnipeg Dykes

FLOOD CONTROL

Portage Diversion
Shellmouth Reservoir
Fairford River
Assiniboine River Dykes

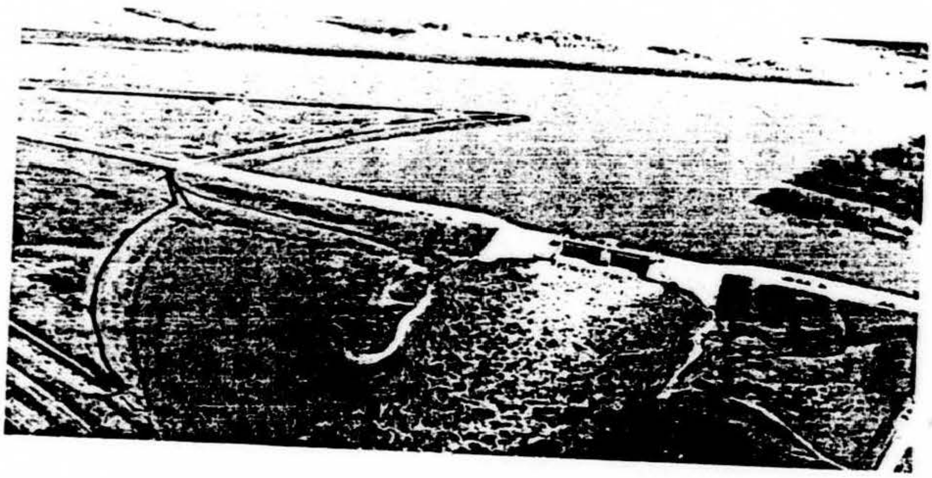


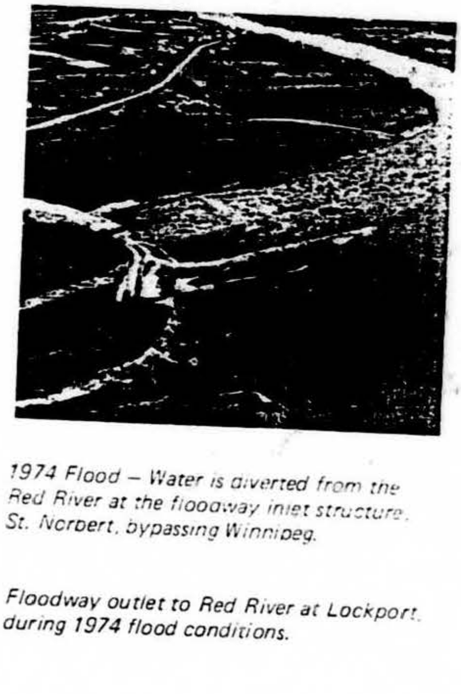
Flood Control

Manitoba lies in a gigantic drainage system which extends east to Ontario, west to Alberta and south to the headwaters of the Mississippi River. This huge area is drained by several major rivers including the Winnipeg, Red, Assiniboine and the Saskatchewan which all flow through Manitoba into Lake Winnipeg. The Lake Manitoba-Lake Winnipegosis system also drains into Lake Winnipeg through the Dauphin River. In the spring, runoff from melting snow frequently causes flooding of the land bordering these rivers and tributaries. Flooding in the mildly sloping Red River Valley exemplifies this type of problem. When the Red River overflows its banks a large area is subject to flooding. In 1950 the floodwaters extended over 15 miles in width, flooding an area of approximately 500 square miles.

Floods have been a tremendous economic burden to Manitobans. It is estimated that the average annual flood damage in Manitoba was $14 million before the major flood control works were built.

The Water Resources Division of the Manitoba Department of Mines, Resources and Environmental Management has overall responsibility for major flood control works and for the co-ordination of flood fighting activities in the Province. This is done on the basis of flood forecasting which allows the Division to evaluate the possibility of spring flooding. The total amount of rain and snow is measured and, in the spring, information is gathered about runoff conditions on a number of streams and rivers including the tributaries of the Red and Assiniboine rivers. This information is analyzed and estimates of peak river flows are determined. Action taken in operating permanent flood control works and evaluation of the need for emergency measures depends on these forecasts.





Red River Floodway

The first thing that comes to mind when we think of flood control in Manitoba, is the Red River Floodway This is Manitoba's largest flood protection project. It is also the most costly, and it provides the most obvious benefit by protecting Winnipeg from damaging floods. As over 80 per cent of the water that passes through Winnipeg during the spring runoff comes from the Red River, control of this river is the key to Winnipeg flood protection.

There were alternatives to building the Floodway. The Red River could have been deepened through the City; or a reservoir could have been

1974 Flood – Water is diverted from the Red River at the floodway inlet structure, St. Norbert, bypassing Winnipeg.

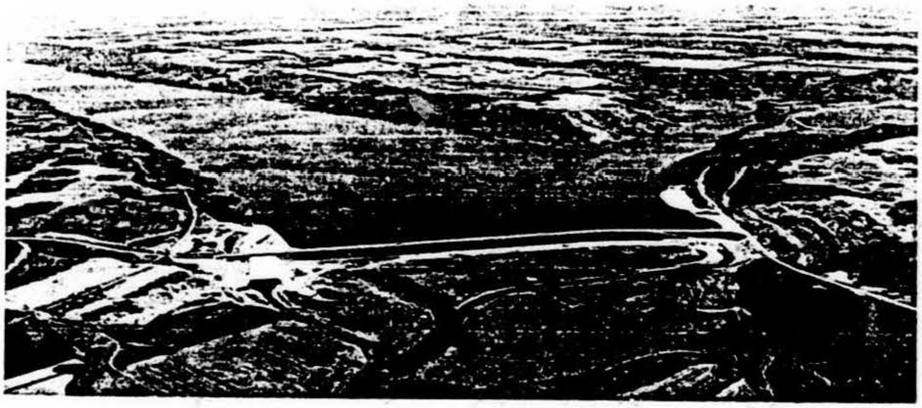
Floodway outlet to Red River at Lockport, during 1974 flood conditions.

built south of Winnipeg to hold excess spring runoff, for slow release during the summer. These were not practical alternatives, however, as dredging the channel would have cost twice as much as a floodway, and a dam built south of Winnipeg creating a flood storage reservoir, would result in heavy flooding to farmland during flood conditions.

The Red River Floodway allows all the water in the Red River to flow through Winnipeg during normal summer, fall and winter months. But in the spring, when the discharge is greater than 30,000 cubic feet per second (cfs), the water flow divides between the Red River and the Floodway.

The amount of water diverted into the Floodway is regulated by a control structure. This structure maintains the Red River's natural level upstream of the Floodway but allows up to 60,000 cfs of flood water to enter the Floodway and bypass the City of Winnipeg.

The project was completed in 1968 at a total cost of $62.7 million which was shared between the Province of Manitoba and the Federal Government.



Shellmouth Reservoir

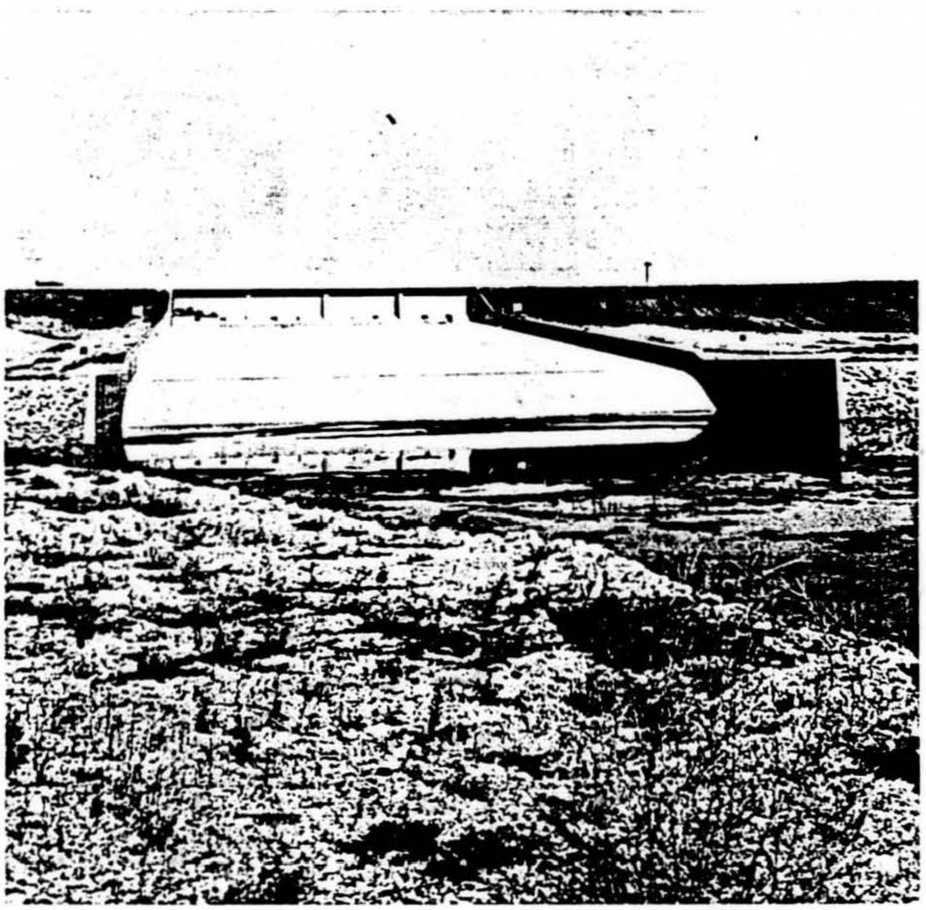
The Shellmouth Reservoir was designed to fill a need for the control of the Assiniboine River and its tributaries. The dam controls the Assiniboine River which used to flood, on average, once every ten years causing damage to a great deal of residential, agricultural and industrial property. Protection is provided to rural areas along the Assiniboine Valley and to urban centres such as Brandon. In addition, the reservoir stores water which, under natural conditions, would raise flood levels in Winnipeg.

Several sites for flood control reservoirs were studied on the Assiniboine

The reservoir stretches back 35 miles from this dam. Spring runoff is stored and released slowly throughout the year.

Gated outlet conduit controls normal river flow.

The concrete spillway, provides for flows in excess of reservoir capacity.

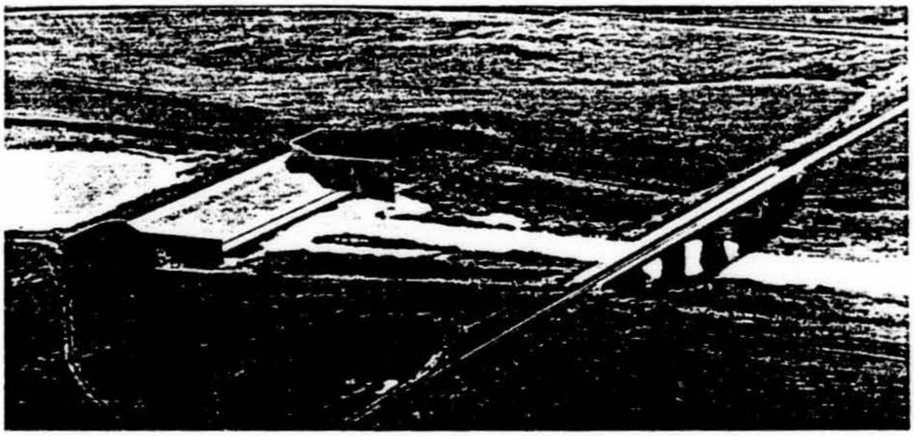


River and its tributaries. It was found more effective to build a dam on the River rather than try to control the tributaries.

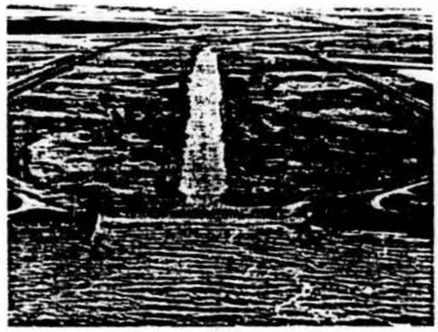
The dam which was built at the junction of the Shell and Assiniboine rivers is 70 feet high and 4,200 feet long. The reservoir stretches upstream for 35 miles, extending into the Province of Saskatchewan. At full supply level, a storage capacity of 390,000 acre-feet is provided of which 225,000 acre-feet are allocated for the storage of flood waters.

The structure is equipped with a gated conduit which controls the amount of water released throughout the year. These releases make it possible to maintain a minimum flow of 300 cfs. at Brandon as compared to the recorded natural minimum flow on the Assiniboine River of 7 cfs. This ensures that users along the Assiniboine River, particularly in urban centres such as Brandon and Portage la Prairie, will have more dependable supplies of water.

This Federal-Provincial project was completed in 1972. The total shareable cost of $10.8 million was divided equally between the two governments.



Portage Diversion



Concrete drop structures control erosion as the Diversion Channel drops between Portage la Prairie and Lake Manitoba.

Lake Manitoba outlet of the Portage Diversion Channel.

Dam in foreground permits control of river levels. Gates in background permit diversion of the Assiniboine River into Lake Manitoba.

Flooding of the Assiniboine River between Winnipeg and Portage la Prairie can be extremely severe as the surrounding land slopes away from the River. This unusual land formation results in widespread flooding and makes it difficult for water to drain back into the River following a flood crest. The water eventually drains back through other drainage systems but, in the process, delays crop planting for weeks.

To alleviate this situation and to reduce flooding in Winnipeg, a channel known as the Portage Diversion was constructed from Portage la Prairie due north to Lake Manitoba.

Diversion of water into the channel is accomplished by two control structures: a dam and spillway on the Assiniboine River, and a gated structure at the inlet to the diversion channel.

The control structure on the River creates an impoundment controlled at a summer water level 869 feet above mean sea level, by operation of the bascule spillway gates. Normal flows are released through a conduit in the structure. Whenever flood conditions exist downstream of Portage la Prairie or at Winnipeg the diversion inlet control structure is opened to permit discharge of the flood waters to Lake Manitoba. The diversion can carry up to 25,000 cfs.

The Portage Diversion is primarily a flood control project, however, the impoundment reservoir, with water levels higher than the natural river stages, could be used as an essential component of a system for delivery of water supplies to south central Manitoba.

The Federal Government contributed $9.3 million of the $20.5 million total cost of the project, which was completed in 1970.

Fairford River Control

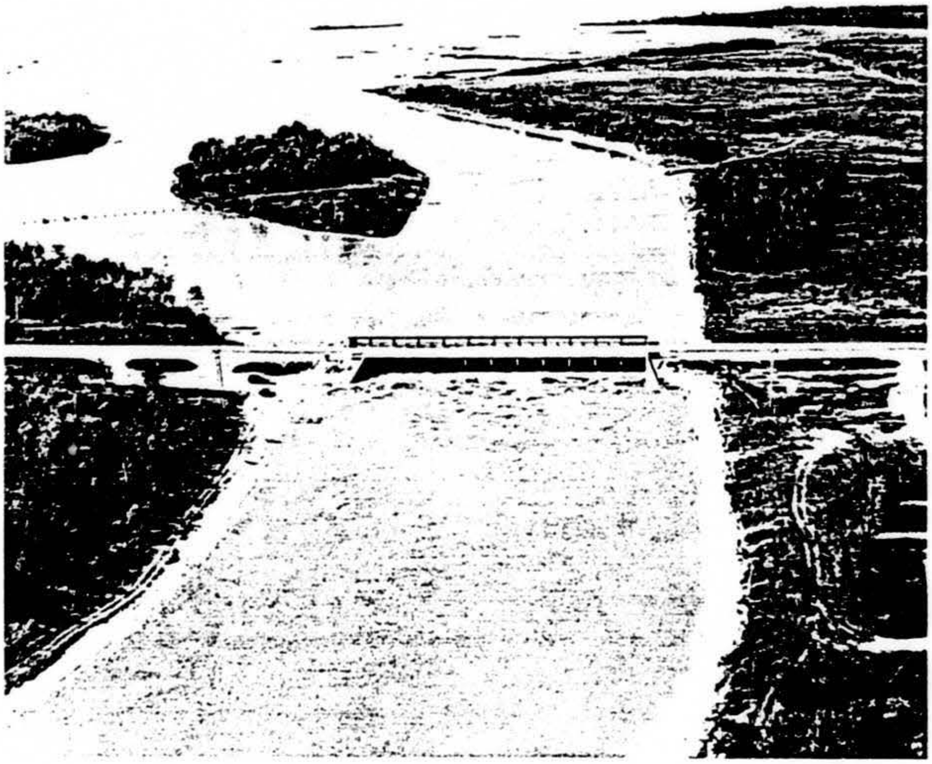
Under natural conditions Lake Manitoba water levels have fluctuated greatly from year to year. Levels, measured in feet above mean sea level, have varied from a low of 809.3 in 1942 to a high of 816.3 in 1955. These extremes have had disastrous effects on farm land and recreation property in the immediate vicinity of the Lake.

When Lake Manitoba was low, cottage owners complained of unattractive beaches and farmers complained because there was not enough water within reasonable distance for their cattle. But when the water level was high large tracts of land were

The Fairford River control is at the outlet of Lake Manitoba. This control structure permits effective regulation of Lake Manitoba water levels.

The area around the control structure is developed as a picnic area and camp site.

The Fairford River channel was widened to provide greater discharge capacity.



flooded, with meadows and pasture land reverting to marsh. This created a considerable loss to farmers. Cottage owners were flooded out and trappers and fishermen also suffered losses.

After public hearings and engineering investigations, the Lakes Winnipeg and Manitoba Board recommended that Lake Manitoba be maintained between elevations 811 and 813, by controlling the outflow from Lake Manitoba at Fairford.

A control structure was already in operation but it and the outlet channel from Lake Manitoba were too small for regulation purposes. Additional discharge capacity was obtained by building a new structure and digging a new channel one and one-half miles long with a base varying in width between 200 and 300 feet. The project was completed in 1961 at a cost of approximately $600,000 which was shared by the Federal and Provincial governments.

Dykes

Following the disastrous 1950 Flood, the Greater Winnipeg Dyking Board was formed on July 10, 1950, by agreement between the Federal and Provincial governments. The Board established a permanent system of dykes. The Federal Government contributed 75 per cent of the total cost of $4.6 million. The remainder was shared between the Provincial and Municipal governments.

The dykes were built along both sides of the Red River and the Assiniboine River to an elevation two feet above the 1948 Flood and four feet below the 1950 peak level. In most cases the dykes were designed as paved roadways throughout the city. In addition, 31 pumping stations were built. During floods these are used if necessary, to pump storm water into the rivers. The dyking system is now a vital and integral part of the flood control works protecting Winnipeg.

Between Winnipeg and Portage la Prairie the Federal Government has spent approximately $2.0 million constructing many miles of dykes along the Assiniboine River. Despite this, extensive flooding continued until the Portage Diversion was completed in 1970. Further upstream in Brandon, 14 miles of dykes were built after the 1955 Flood.

Morris is surrounded by a dyke to provide protection to the built up area.

In the City, landscaping can turn a river dyke into an attractive feature of the property.

1974 Flood — Morris, farmland outside the protective ring dyke lies under water.

Large areas in the Red River Valley are subject to periodic flooding but, for economical and physical reasons, it is not possible to provide complete protection by dyking along the Red River. Instead, protection has been provided, under a program financed by the Federal and Provincial governments, by construction of ring dykes around the communities of Emerson, Letellier, Dominion City, St. Jean Baptiste, Morris, Rosenort and St. Adolphe. Total cost of the program completed in 1972 was $2.7 million. Brunkild had been protected previously by dyking constructed by the Province.

There is an inherent risk to residents within a dyking system because dykes could fail or be overtopped under severe flood conditions. Legislation has been passed which authorizes the evacuation of dyked areas, if necessary, for reasons of health or safety of the residents.

Project Statistics

SHELLMOUTH DAM

Height	70 feet
Length	4,200 feet
Storage Capacity	390,000 acre feet
Reservoir Length	35 miles

FAIRFORD

Length	237 feet
Stop Log Bays	11
Discharge Capacity	10,000 cfs
Regulation Range	810.87 – 812.87

PORTAGE DIVERSION

Diversion

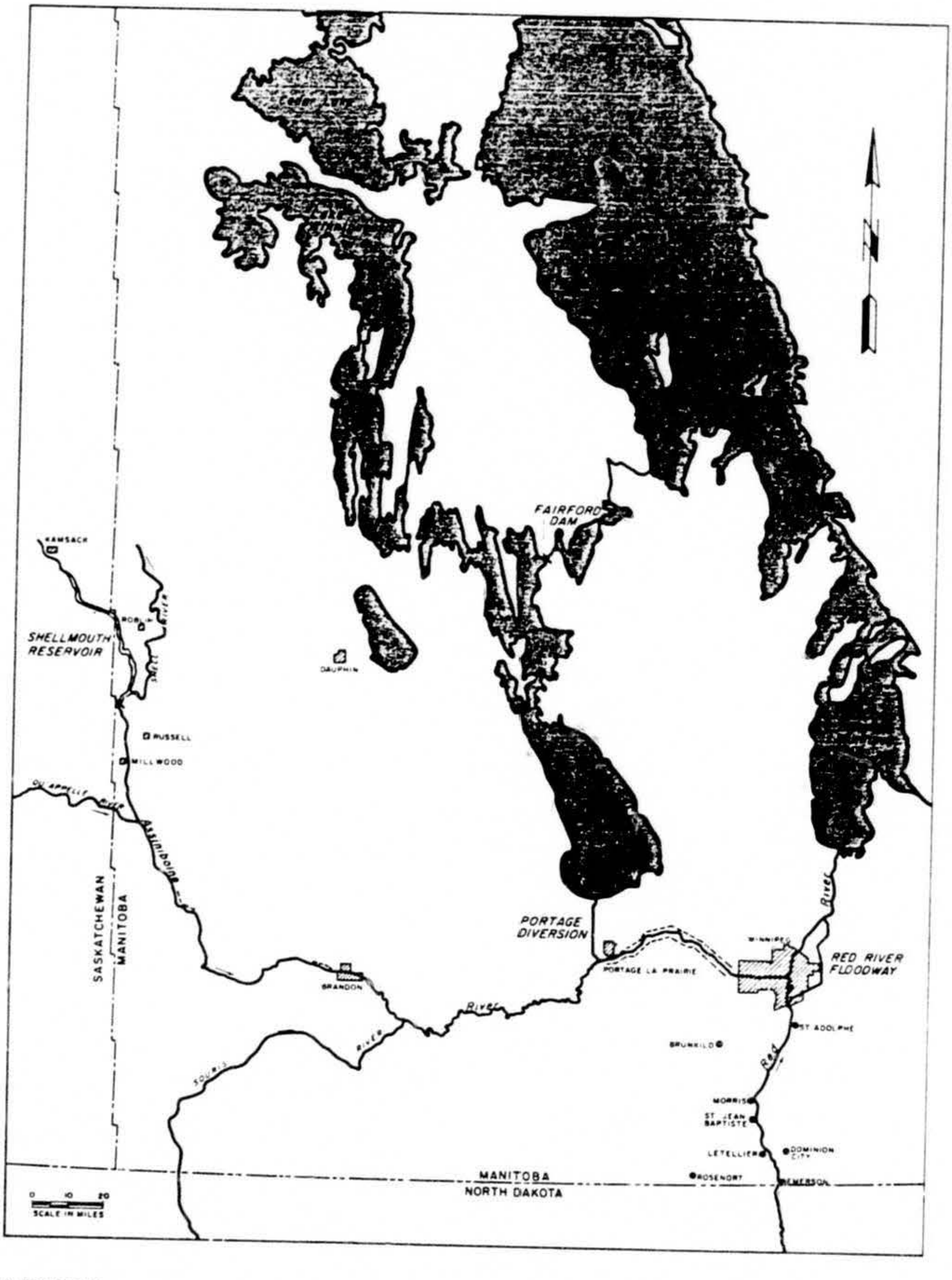
Length	18 miles
Capacity	25,000 cfs
Inlet Gates	4 – 14.5′ x 40′

Control Dam

Height	35 feet
Length	1,400 feet
Gates	2 – 13′ x 75′
Reservoir Storage	14,600 acre feet

RED RIVER FLOODWAY

Length	29.4 miles
Average Depth	30 feet
Base Width	380 – 540 feet
Design Capacity	60,000 cfs
Control Gates	2 – 34.8 x 112.5



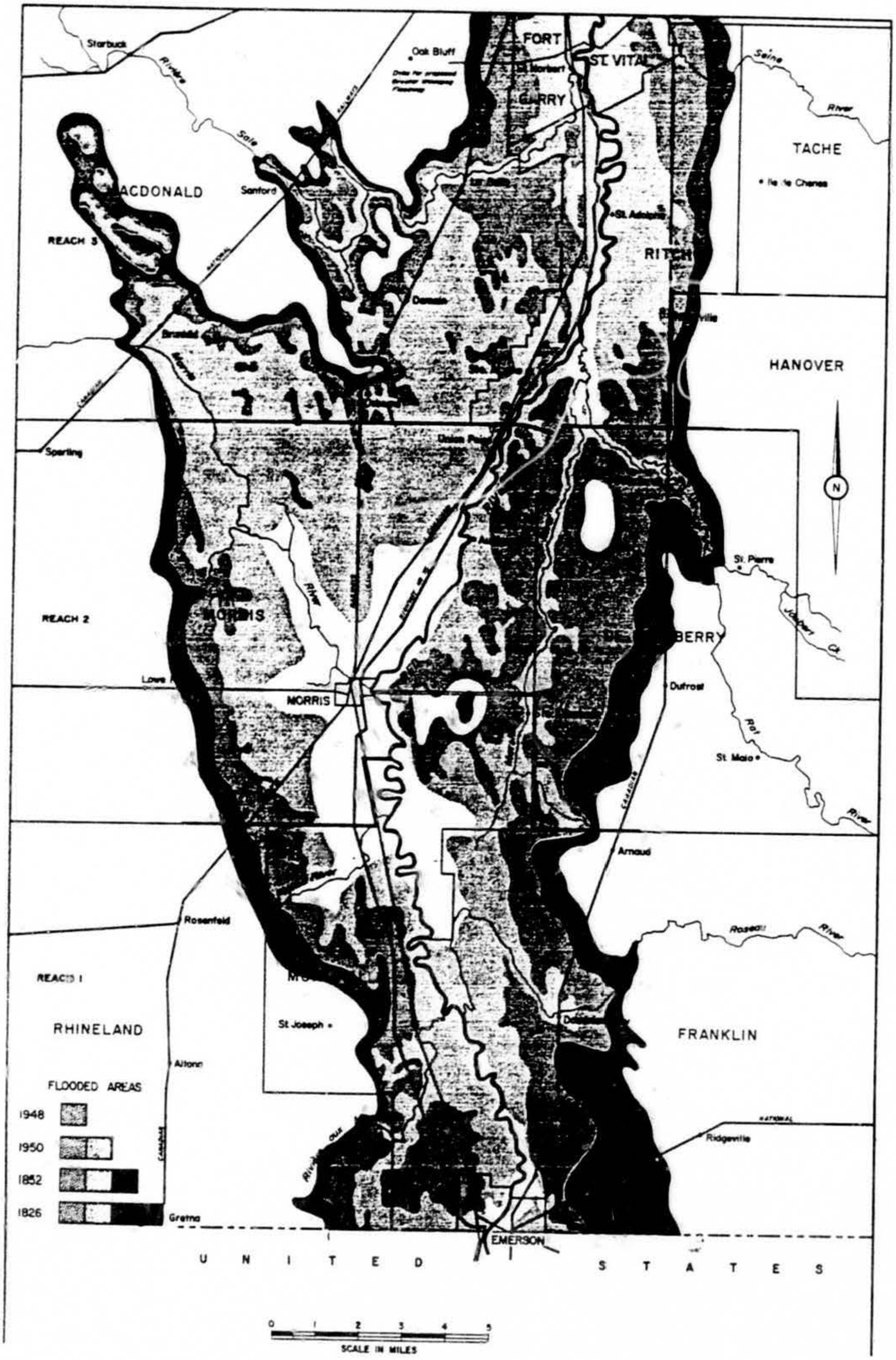
LEGEND

COMMUNITY DYKING •

DYKES —

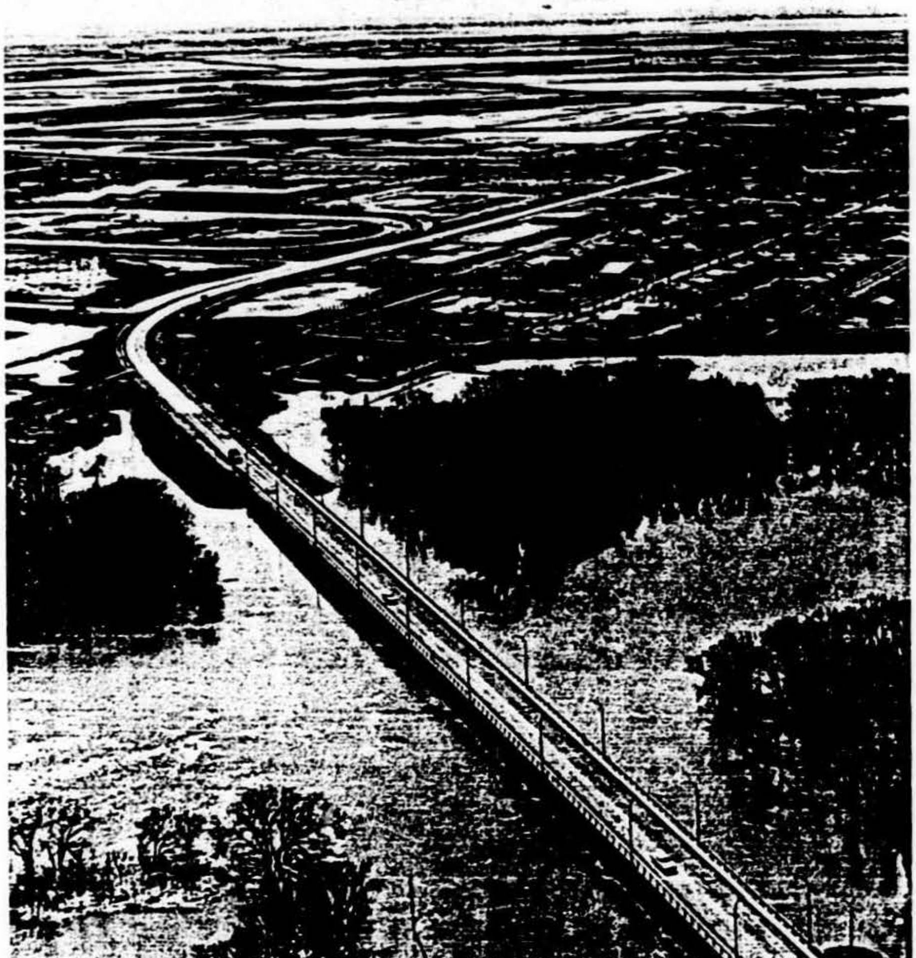
MAJOR FLOODS IN THE RED RIVER VALLEY

Year	Height at Winnipeg Above Datum (feet)	Peak Discharge (cfs)	Probable-Flood Frequency (Years)	Flooded Acreage
1948	23.1	69,000	12	67,400
1950	30.3	103,000	36	316,500
1852	35.2	165,000	150	523,000
1826	37.3	225,000	460	616,000

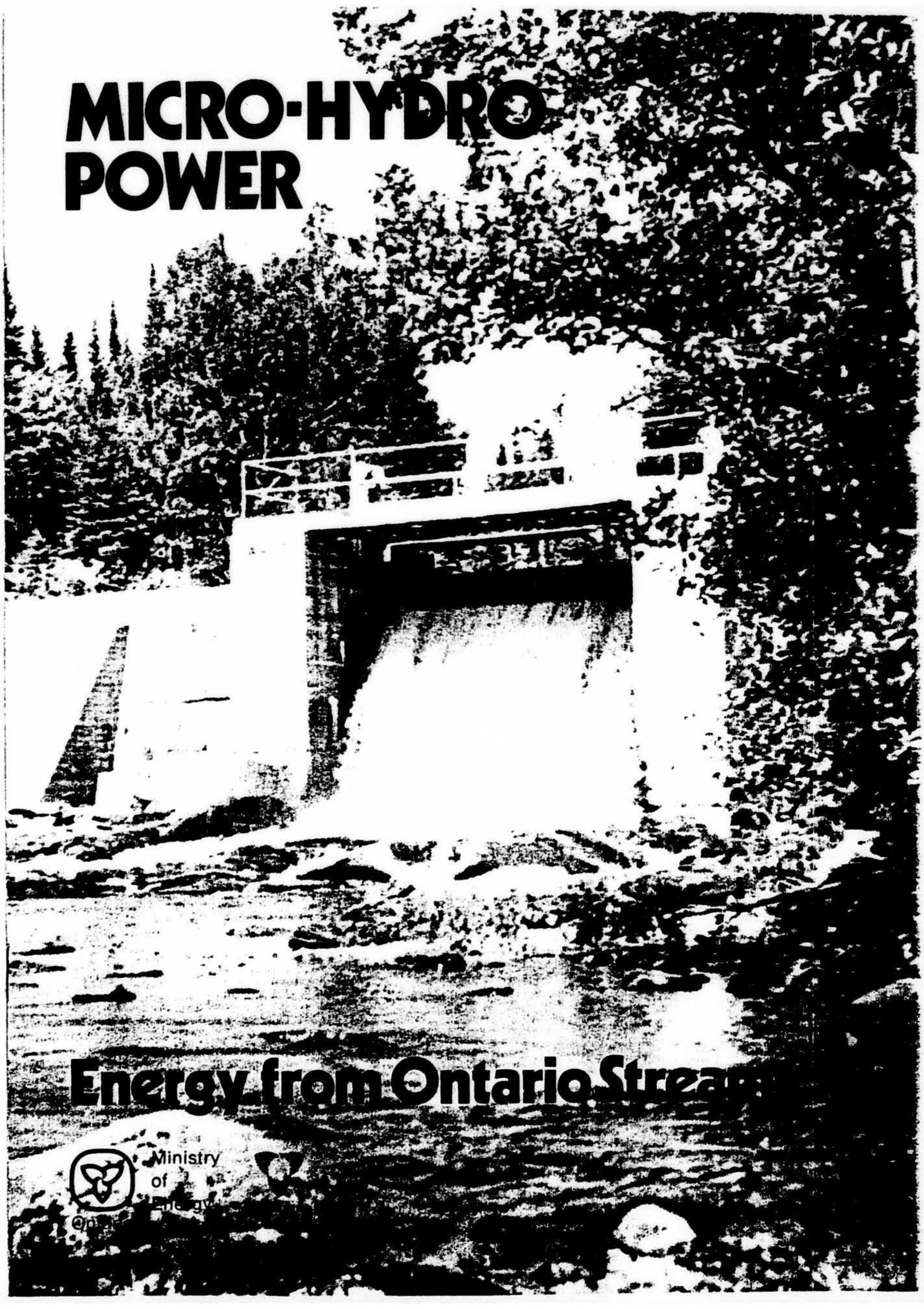


Starbuck
Oak Bluff
FORT
GARRY
ST. VITAL
St. Norbert
Rivière
Sale
Seine
River
TACHE
MACDONALD
Sanford
St. Adolphe
REACH 3
RITCHOT
HANOVER
Sperling
N
St. Pierre
REACH 2
MORRIS
River
BERRY
Joubert Cr.
Lowe
MORRIS
Dufrost
Rat
St. Malo
River
Arnaud
Rosenfeld
Roseau
River
REACH 1
RHINELAND
St. Joseph
FRANKLIN
Altona
FLOODED AREAS
1948
1950
1852
1826
Ridgeville
Gretna
EMERSON
U N I T E D S T A T E S
0 1 2 3 4 5
SCALE IN MILES

Morris, Manitoba during 1974
flooding, illustrates necessity
of dyking systems.



MICRO-HYDRO POWER



Energy from Ontario Strea

Ministry of Energy

This report, prepared for the Ministry of Energy for Ontario, is published as a public service for the information of the public. The Ministry does not, however, warrant the accuracy of its contents and cannot guarantee or assume any liability for the effectiveness or economic benefits of the devices and processes described in the report. The list of suppliers are those known to the Ministry at time of printing.

The assistance of Ontario Hydro in supplying much of the research and data contained herein, is acknowledged and appreciated.

COVER: This is one of the many small dams now in use in Ontario that could be considered suitable for micro-hydro power.

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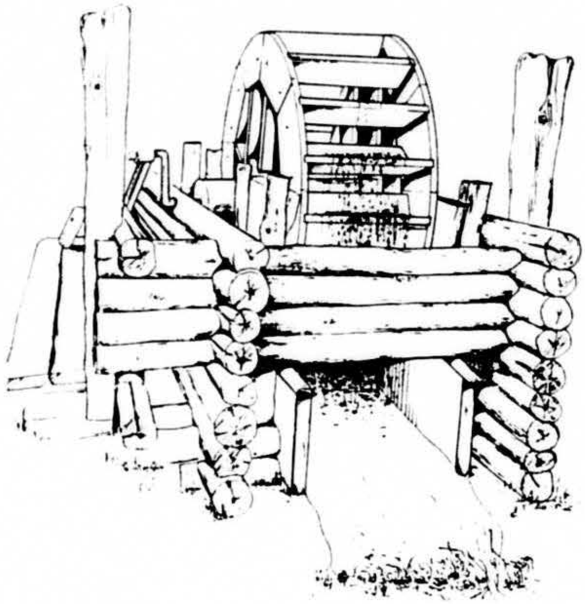

This is an artist's conception of one of the earliest methods of obtaining water power, with its origins going back to Roman times. Completely mechanical, the principle has changed little over the centuries.

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MINISTER'S MESSAGE

With the price of energy climbing, there is a growing interest throughout much of the world in micro-hydro systems — the generation of electrical power from small rivers, streams and waterfalls — to serve homes, farms, shops and even small communities of up to 25 homes.

Ontario has thousands of small sites where water power can be used to do just that.

Where environmental and community concerns can be met, such projects can be a useful supplemental source of energy for the province.

The Ontario Ministry of Energy plans to install demonstration micro-hydro generating stations at various sites throughout Ontario, to assess the technology currently available and to provide visible examples as a guide to others.

Water power, after all, was the original "fuel" for Ontario's electricity system. Even now, with oil, coal and nuclear power being used, it still provides about 36 per cent of the province's electrical power.

And it will continue to play an important role in Ontario's energy future. It is an indigenous and renewable energy source that can be tapped by proven technology. Furthermore, in many cases water power is environmentally benign and can replace the burning of fossil fuels for electrical generation.

While Ontario's untapped hydraulic potential, even if fully developed, is not sufficient to meet more than a portion of our growing electrical demand, the energy contribution from both large and small sites is welcome.

This booklet sets out the steps to follow in developing very small sites.

And while it is a practical "how-to" guide, applicable to installing almost any micro-hydro system, it is also a guide to where such projects are feasible.

The booklet shows how to evaluate the power potential of a particular site and how to determine residential and farm power requirements. To be attractive, a potential site should be capable of meeting both peak and average energy requirements.

It also includes information about typical micro-hydro components and power site layouts. Some details about alternative equipment and systems are also presented, along with general cost information and comments on the effects of various siting factors. Environmental considerations are also discussed, and there is a summary of the governmental approval process.

It is important to remember, however, that this booklet is general in nature and does not provide details of design, costs, and installation. Such information is available from other sources, some of which are listed in the bibliography, or from an appropriate hydroelectric equipment supplier or engineering consultant.

I wish the very best to all who invest their money, time, and effort in installing and operating a successful micro-hydro unit.

Yours sincerely,

Robert Welch

Hon. Robert Welch,
Minister of Energy

WHAT IS A MICRO-HYDRO SYSTEM?

Throughout this booklet, the term micro-hydro refers to an installation with a capacity of 100 kW or less.

The term system implies all of the components required to convert the potential energy in a stream into electrical energy at the user's location.

Installations with a capacity of 100 kW to 10 MW (10,000 kW) are referred to as small hydro. While much of the information in this booklet is also applicable to small hydro, t ese larger installations are more complex than micro-hydro systems.

Installations beyond 10 MW are known as medium and large hydraulic plants. This booklet is not useful for planning such installations. They require teams of specialists for feasibility studies and for engineering and planning.

No two hydroelectric developments are exactly alike. Each involves a unique set of considerations in design and construction. The heart of a micro-hydro system is the generating equipment — the turbine, generator, and control mechanisms — all of which are generally housed together.

To deliver water to the turbine, a dam to divert the stream flow and a pipeline — also known as a penstock — are typically required.

Depending on the site, other requirements may include a canal or forebay; trashracks to filter debris and prevent it from being drawn into the turbine at the pipeline entrance, and a pipeline gate or valve. A tailrace, or waterway back to the stream, must be provided if the powerhouse is not situated to permit discharge directly into the stream. The micro-hydro system must also include electrical lines to deliver the power to its destination.

Figure 1 illustrates a typical micro-hydro installation.

The type of micro-hydro plant best suited to a given site will depend upon many factors, including the head — vertical distance through which the water falls — the available flow of water, and the general topography of the area.

A concentrated-fall hydroelectric development, (Figure 1), is one in which the powerhouse is located near the dam, thus requiring a minimum length of pipeline. In such installations, which are common for low-head developments, the powerhouse may be located at one end of the dam or directly downstream from it.

In a divided-fall development, (Figure 2), the powerhouse is loacted a considerable distance from the dam, and water is delivered through a pipeline or canal. With favourable topography this type of development can make it possible to realize a high head despite a low dam.

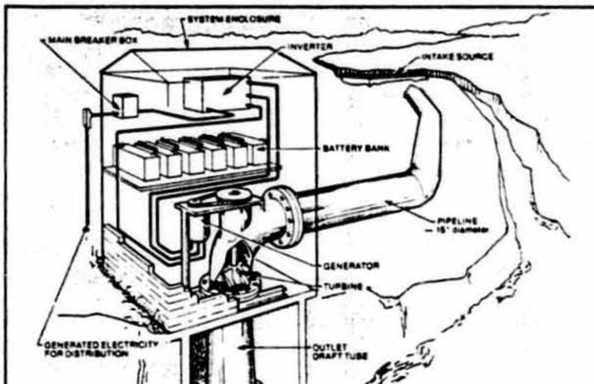


Figure 2 A Divided Fall Micro-Hydro Development.

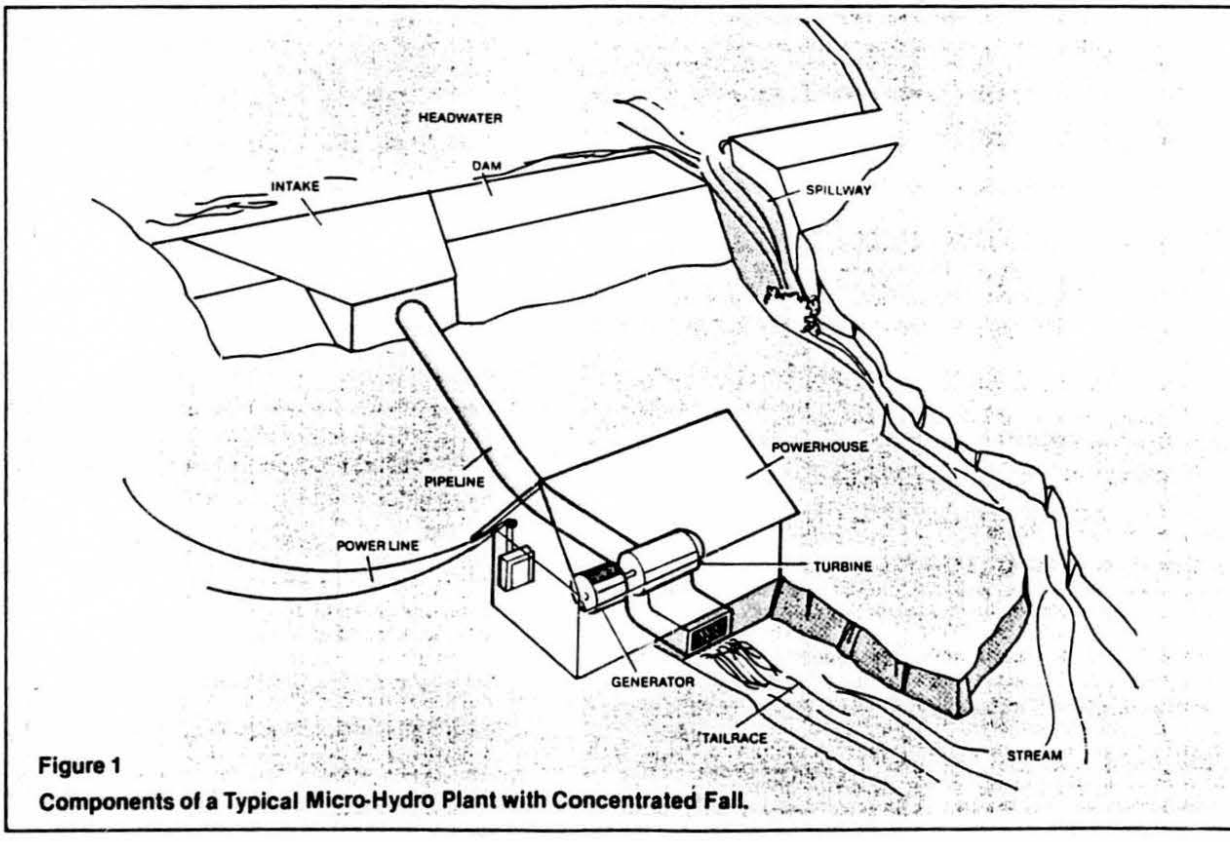


Figure 1

Components of a Typical Micro-Hydro Plant with Concentrated Fall.

CONSIDERATIONS FOR A MICRO-HYDRO SYSTEM

Seven factors must be addressed in deciding if micro-hydro would work at a specific site.

1. Hydro-Site Potential
2. Power Requirements
3. Environmental Impact and Approvals
4. Equipment Options
5. Alternative Layouts
6. Costs
7. Economics

Following are suggestions for assessing each of the seven:

1. Hydro-Site Potential

To determine the hydraulic potential of the water flowing in your stream, you must know both the water's flow rate and the head through which it can be made to fall.

The flow rate is the quantity of water moving past a point in a given time. The head is the vertical height from the headwater in the case of a dam. Where no dam exists, the head is the vertical height from the level where the water enters the intake pipe to the level where the water leaves the turbine housing.

Appendix 2 gives a detailed account of determining flow rate and head.

The technology for harnessing hydroelectric energy is more than 100 years old. In basic terms, the amount of energy that can be generated in a powerplant at a given site depends upon the following three factors:

Flow (Q) — the quantity of water available

Head (H) — the vertical distance through which the water falls

Efficiency (e) — the ability of the powerplant to utilize the available head and flow. Normally, powerplant efficiency is about 70 per cent to 90 per cent.

The capacity of the powerplant in kilowatts (P), in Imperial units, may be expressed as:

$$P = \frac{Q \times H \times e}{709}$$

where:

Q = the usable flow rate of water through the turbine in cubic feet per minute.
NOTE: Work the equation first with **mean** annual flow, then with **minimum** flow to learn the site's true potential.

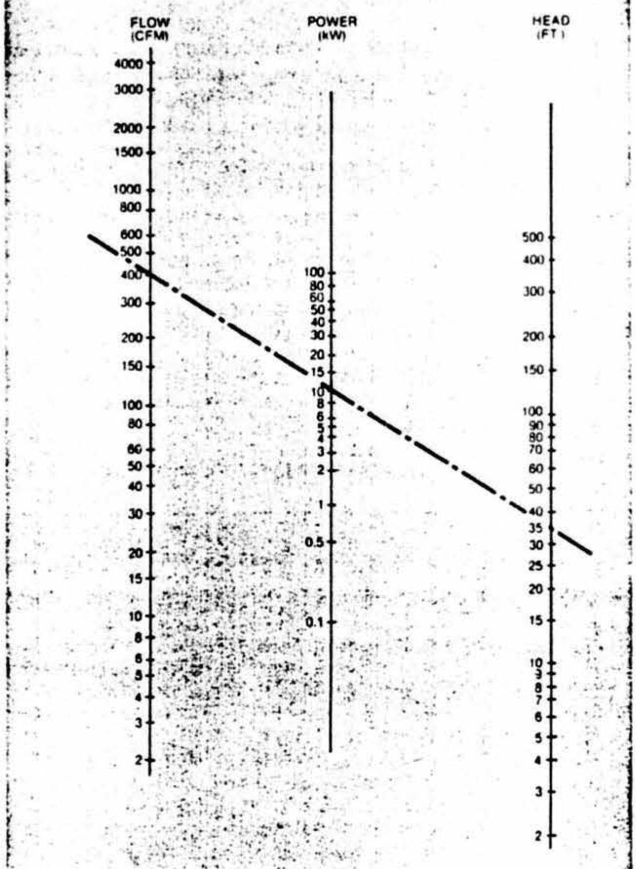
H = The available head, in feet

e = the overall powerplant efficiency

709 = a derived constant

The graph presented in Figure 3 can also be used to determine the power output of a micro-hydro system. This type of graph is called a **nomograph**. It is designed so that a straightedge, positioned at the figures corresponding to the head and flow of the potential site, allows reading the power output available in the middle column. In the example shown, a **flow** of 400 cubic feet per minute and a **net head** of 35 feet gives an estimated output of 10 kW.

Figure 3
Nomograph to determine typical output power from a micro-hydro system.



**Assumed system efficiency of 50%.*

NOTE: In metric units, the capacity may be expressed as:

$$P = \frac{Q \times H \times e}{102}$$

where:

Q = the usable flow rate of water through the turbine, in litres per second

H = The available head, in metres

e = the overall powerplant efficiency

102 = a derived constant

CONSIDERATIONS FOR A MICRO-HYDRO SYSTEM

2. Power Requirements

Early in any assessment, power needs and the characteristics of those needs should be carefully examined. This entails two separate but related determinations:

- ENERGY: The total number of kilowatt hours (kWh) used in a given period of time — a month or a year
- PEAK POWER CONSUMPTION: The maximum amount of electricity which will be needed at any one moment

Accuracy is essential. Inaccurate estimations will result in too expensive a system or one that does not meet the power needs.

If the existing power supply is from a utility company, refer to past billings to determine total consumption.

TABLE 1

Typical Household Appliance Loads

APPLIANCE	POWER (watts)	AVERAGE HOURS ON PER MONTH	ENERGY USED (kW/hours) PER MONTH
Air Conditioner	1,560	74	116
Blender	600	3	2
Car Block Heater	450	300	135
Clock	2	720	1
Clothes Dryer	4,600	19	87
Coffee Maker	800	12	10
Electric Blanket	200	80	16
Electric Heating	6,000	250	1500
Exhaust Fan (kitchen)	250	30	8
Freezer (chest 15ft³)	350	240	84
Hair Dryer (hand-held)	400	5	2
Hot Plate (1 burner)	500	14	7
Iron	1,100	12	13
Lights — each 60 watt	60	120	7
— each 100 watt	100	90	9
— each 4 ft fluorescent	50	240	12
Mixer	125	6	1
Portable Electric Heater	1,250	350	440
Radio — tube type	80	120	10
— solid state	50	120	6
Range (top only — 4 Burners)	6,000	10	60
Range & Oven	7,200	10	60
Refrigerator — standard 14 ft³	300	200	60
— frost free 14 ft³	360	500	180
Sewing Machine	100	10	1
Stereo — tube type	115	120	14
— solid state	30	120	4
SHOP TOOLS:			
— 1/4 inch Drill (1/16 hp)	250	4	1
— Skill Saw (1 hp)	1,000	6	6
— Table Saw (1 hp)	1,000	4	4
— Lathe (1/2 hp)	460	2	1
Toaster	1,150	4	5
TV — black & white	250	120	30
— colour	350	125	44
Washing Machine (15 loads/month)	700	12	8
Water Heater — 30 gal	1,200	89	107
— 40 gal	2,000	89	179
— 50 gal	3,000	89	286
Water Pump (1/2 hp)	460	44	20
Vacuum Cleaner	750	10	8

Note: Household consumption usually varies over a year. It is wise to study at least one complete year's bills.

For another way to estimate energy requirements, see Table 1, which lists the energy used by typical household appliances.

To estimate your monthly energy requirements, select all the appliances you use and add the corresponding numbers from the energy required column. To estimate peak power requirement, add the figures from the power column for all appliances that could conceivably be used simultaneously.

To determine if a micro-hydro facility can supply all the electrical power needed at a price you are willing to pay, you must measure both total energy consumption and peak power consumption because a situation may arise in which the system could meet one need but not the other.

In most micro-hydro systems — as with larger systems — the peak requirement is more likely to cause problems than the total requirement.

Any hydroelectric system — micro or macro — that has to serve an occasional peak load will be less efficient and more costly to install and run than a system serving a more uniform demand for energy.

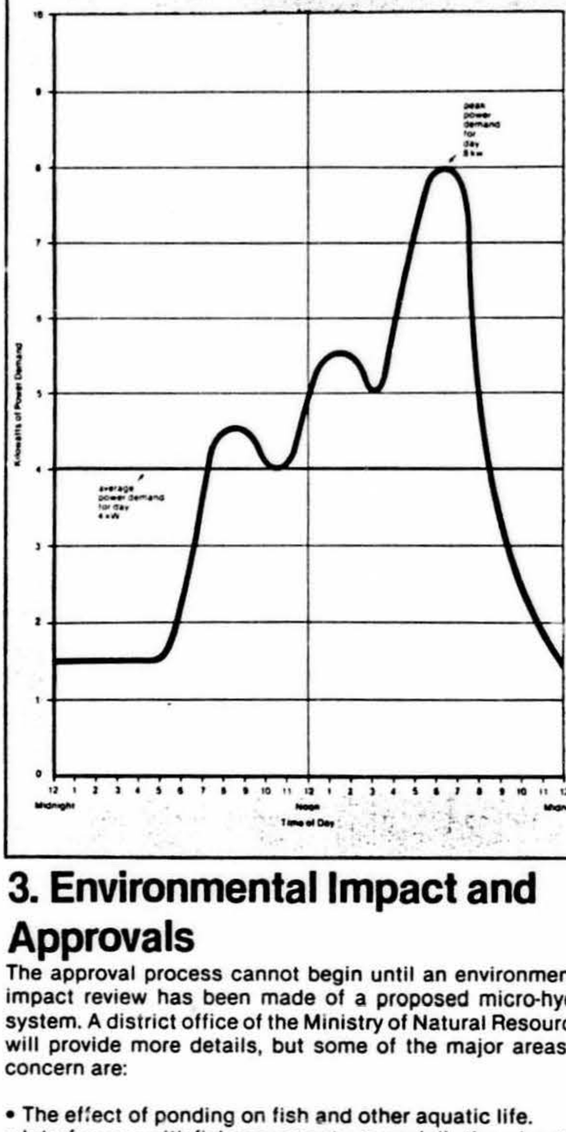
So, when analysing a proposed micro-hydro system, ask yourself if you can make some adjustments in the way, or pattern, in which you use electricity. Can you change your peak consumption to fit the system you are planning?

Explore this possibility before deciding that you must provide for a high peak consumption.

A typical pattern of peak load and the resulting average load is illustrated in Figure 4.

Figure 4

A generalized residential pattern of power demand illustrating the difference between peak and average power demand.



3. Environmental Impact and Approvals

The approval process cannot begin until an environmental impact review has been made of a proposed micro-hydro system. A district office of the Ministry of Natural Resources will provide more details, but some of the major areas of concern are:

- The effect of ponding on fish and other aquatic life.
- Interference with fish movements, especially the migration of pickerel, suckers, and salmonids.
- Making sure fish can't be sucked or dragged through the turbine.
- Physical and chemical consequences of changing the stream's flow.
- Maintenance of adequate stream flow downstream from the project.

If the initial assessment shows unacceptable environmental impact, it may be possible to change the plan or take steps that will allow the project to proceed. For example, if interruption of fish migration is a problem, provision could be made for a fishway.

A district office of the Ministry of Natural Resources, a regional office of the Ministry of the Environment, and officials of the local municipality and/or conservation authority should be contacted while the project is still in the design stage. They will be able to provide guidance for the assessment, and outline what is required for their approval.

For assistance in contacting the appropriate officials call the Ministry of Energy, Electric Power Section at (416) 965-9603, or write to them at 56 Wellesley St. W., Toronto, Ont. M7A 2B7.

Most of the approvals required can be handled by a district office of the Ministry of Natural Resources. The district manager and his staff can also help with a general assessment of the proposed site if called in at an early stage. For a complete list of district offices, turn to Appendix 6 on page 22.

The following outline of the required applications and approvals and of the federal and provincial legislation involved may appear daunting. But many of the approvals can be obtained concurrently. In this connection, a good working relationship with the staff at a district office of the Ministry of Natural Resources will be invaluable. With their assistance the approval process will be much smoother.

The first step is to determine whether the water course proposed for development is considered to be navigable. In general, a stream is considered to be navigable if, at average flow, it is suitable for commercial or recreational boating. In most cases, the district Ministry office will be able to advise whether a particular stream is considered navigable.

The following legislation may apply:

The Navigable Waters Protection Act

A hydroelectric project on a navigable watercourse requires the approval of the federal Department of Transport to ensure that the project will not interfere with navigation. An application form for approval under Section 5 of **The Navigable Waters Protection Act** (R.S.C. 1970) may be obtained from Transport Canada. It must be submitted to the Chief, Navigable Waters Protection Act, Programs Division, Canadian Coast Guard, Transport Canada, 6th floor, Tower A, Place de Ville, Ottawa, K1A 0N7.

The Beds of Navigable Waters Act and The Public Lands Act

If a stream is considered to be unnavigable, the bed belongs to the owner, or owners, of the banks of the stream. However, under **The Beds of Navigable Waters Act** (R.S.O. 1970 c. 41) the beds of all navigable waters belong to the Crown unless expressly granted in the original letters patent.

If the Crown owns the bed of the stream, the water may not be used to generate electricity without a Water Power Lease under Sections 4 and 4a of **The Public Lands Act** (R.S.O. 2970 c. 380). Such a lease currently costs approximately $2.50 per average kilowatt output per year ($3.50 per average horsepower per year) and may be obtained by applying to the local district office of the Ministry of Natural Resources. There is no charge for installations of less than 100 hp, or approximately 75 kilowatts.

CONSIDERATIONS FOR A MICRO-HYDRO SYSTEM

The Lakes and Rivers Improvement Act

If the project will require either construction of a new dam or material alteration of an existing dam — regardless of whether the landowner owns the bed of the stream or if it is navigable — the project must be approved by the Ministry of Natural Resources, under **The Lakes and Rivers Improvement Act** (R.S.O. 1970, c. 233).

If a new dam is required, the owner must first apply, under Section 10, to a district office of the Ministry of Natural Resources for approval of the location of the dam. If the location is approved, or if the only requirement is alteration of an existing dam, a separate application must then be made to the district office of the Ministry of Natural Resources for approval of the detailed plans and specifications for a new dam (Section 10) or for a modified dam (Section 12).

The Minister of Natural Resources may require that a fishway be installed around any new dam. If the Minister intends to refuse to approve a new dam, or modifications to an existing one, he must give the applicant 15 days' notice of his intention (Section 8a). At the applicant's request, the Minister will appoint someone to hold a public inquiry regarding the proposed undertaking and report back to him.

The Minister will then consider the report and decide whether to approve the application (Section 8b). The Minister's decision may be appealed to the Lieutenant Governor-in-Council (the Cabinet) (Section 8c).

The Canada Fisheries Act

New dams, modifications to existing dams and the intake to the turbine are also subject to **The Canada Fisheries Act** (R.S.C. 1970, Chapter F-14).

A project may be prohibited if it will result in harmful alteration, disruption, or destruction of fish habitat unless special permission is granted by the federal Minister of Fisheries (Section 31). If the project is permitted, that Minister may still require that a fishway be installed around any dam or other obstruction. He may also require that a minimum flow of water be maintained over the dam (Section 20). The Minister may also require a fishguard to be installed on the turbine's water intake or similar measures designed to protect the fish (Section 28).

In Ontario, this Act is administered by the Ministry of Natural Resources. The requirements of the Act may be considered in conjunction with the review under **The Lakes and Rivers Improvement Act.**

Conservation Authority Approval

If the proposed site lies within the boundaries of a conservation authority, its review of the project normally proceeds in parallel with the MNR's under **The Lakes and Rivers Improvement Act.**

Documents submitted to a district Ministry office will be forwarded to the conservation authority involved at your request. However, separate approval may be required, so the applicant should approach the authority early in the approval process. A conservation authority's main concern is usually the effect the project would have on flooding.

The Ontario Water Resources Act

Another Ministry of Environment approval is required if a project requires a dam and will use more than 1.1 cubic feet of water per minute (10,000 gallons per day). A water-taking permit under Section 37 of **The Ontario Water Resources Act** (R.S.O. 1970, Chapter 332) may be obtained by applying to the Chief, Water Resources Assessment, at a regional office of the Ministry of the Environment. The district office of the Ministry of Natural Resources will be able to direct you to the **proper Ministry of the Environment regional office.**

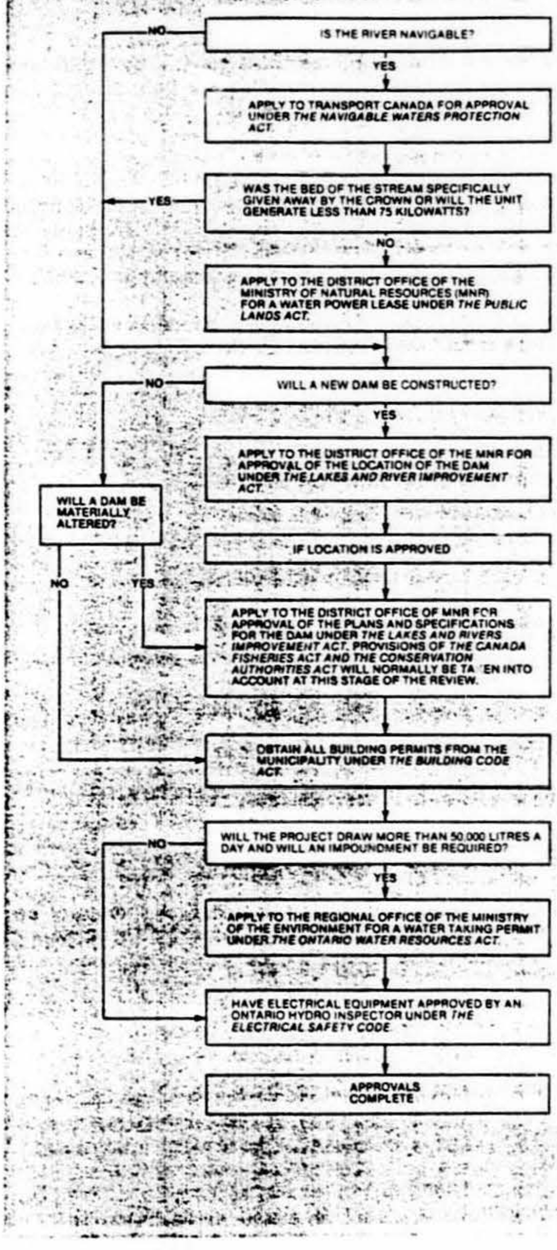
The Power Corporation Act — Electrical Safety Code

Installation of the generator, and other related equipment, is subject to the Electrical Safety Code, a set of regulations administered by Ontario Hydro under **The Power Corporation Act** (R.S.O. 1970, Chapter 354). As with most installations, the electrical part of a hydraulic generating station must be approved by an electrical inspector before use.

A municipality may require permit applications for parts of the project. Discuss your plans with the proper officials early in the approval process to determine the requirements.

Figure 5 illustrates steps to follow in the approval process.

Figure 5
Micro-Hydro Development — Summary of Approvals.



4. Equipment Options

Turbines

The mechanical energy developed by the turbine is converted into electricity by a generator similar to the one in your car. The electricity can be either direct current (DC) or alternating current (AC). Since most household appliances run on AC — 110 volts, 60 cycles — it will be the most practical type of electricity to produce.

If you intend to be completely independent of Ontario Hydro's power grid, a synchronous generator which produces a steady and dependable 60-cycle current should be used.

North American AC systems operate at a frequency of 60 cycles per second and any variation from that will affect the accuracy of clocks, turntables and other appliances. In order to generate power at this frequency, the speed of the synchronous generator must be constant. A governor is used to control water flow which determines the turbine's speed. A governor is a device that regulates turbine speed through water flow in synchronous generators. Available from numerous sources, they are reliable and accurate, but they do mean more cost and maintenance.

The classic approach to hydroelectric development may be described as follows:

Water is held back by a dam, which provides a steady water flow into the turbine as power is produced. As power is consumed, a governor-actuated feedback system regulates the amount of power produced in order to maintain a balance.

Because power demands must be met immediately by the generator, all components must be able to meet peak, rather than average, output. Such an installation requires a big enough turbine and generator, and enough water, to handle peak loads. This is a problem that plagues all utilities. Peak-load capacity, plus a little margin, costs far more than average-load capacity. In addition, the cost of sophisticated governing equipment and flow-rate control mechanisms may render a scheme unattractive.

One way of simplifying the problem is to provide constant generation at a level equivalent to the peak demand. Such generating units usually dump excess energy as heat which can sometimes be used for supplementary space heating or water heating.

When the demand begins to approach the system's capacity, such non-essential uses are automatically cut off so critical needs may be met. No throttling gates or valves are required to limit the flow of water during low-demand periods. This technique is attractive where there is an abundant supply of water, because of the relative simplicity of the control mechanisms required.

Another option is to generate direct current and either use it as is or convert it to AC, when needed, by means of an "inverter" of a "Gemini converter." Inverters are relatively expensive, power is lost in the conversion process, and some of them are limited in their ability to handle the kind of surge demand that occurs when many common appliances start up. The Gemini converter must be connected to an electrical utility.

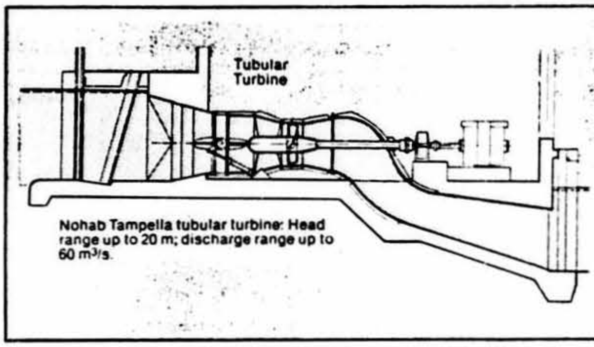
A DC-to-AC system has several advantages, especially in very small systems (less than 5 kW). The excess power generated by a DC system can be stored in batteries, thereby extending the system's peak capacity. DC generators are not speed-sensitive and no governor is needed. So, a small DC system will usually cost less and serve better than a comparable AC system because a small AC unit often cannot meet peak needs.

Battery storage systems generally work better in hydraulic units than in wind-power units because the hydraulic generator is nearly always putting some power back into the batteries. This means that a "deep-discharge" condition, common with wind systems, is very rare.

Deep discharge is a common cause of battery failure. The storage component limits the size of a DC system, as batteries become unwieldy and very costly in systems over 6 kW.

For those seeking the lowest cost, the conversion of DC to AC could be eliminated and 12-volt DC lights and appliances would be used.

There are two main types of hydraulic turbines: impulse and reaction.



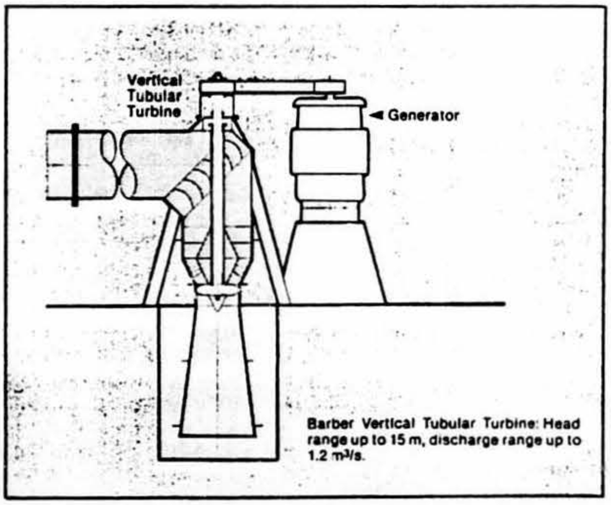
Nohab Tampella tubular turbine: Head range up to 20 m; discharge range up to 60 m³/s.

Impulse Turbines

Impulse turbines have generally been used for very high heads, although modern, efficient units exist for low-head applications, down to about 20 feet.

Advantages of impulse turbines include high reliability and low maintenance cost because of their mechanical simplicity and a minimum number of parts exposed to wear. Impulse turbine efficiencies often exceed 90 per cent.

Impulse turbines use a high-velocity stream of water that strikes buckets mounted around the rim of a rotating turbine wheel, or runner, which in turn is attached to and rotates an electric generator.



Barber Vertical Tubular Turbine: Head range up to 15 m, discharge range up to 1.2 m³/s.

The crossflow turbine is a modern adaptation of the impulse turbine and is used in the head range from 6 to 600 feet. This type of turbine utilizes a movable guide vane at its inlet and maintains turbine efficiencies of up to 85 per cent over a wide range of flows. The physical size of crossflow turbines is limited by design constraints. The largest runner has a 4-foot diameter.

Reaction Turbines

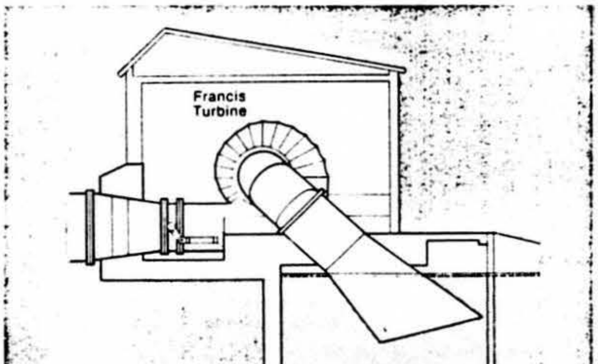
Reaction turbines, while doing the same job as impulse turbines, utilize a different principle. They use pressure as well as velocity to rotate the runner. The runner is submerged in water during operation and power is developed by water flowing over the blades, rather than striking individual buckets.

By using a gradually enlarging draft or discharge tube between the turbine's runner discharge and the tailwater, reaction turbines take advantage of the total head available to the tailwater level, whereas impulse turbines utilize only the head that is available to the centre of the runner. This enhances the value of reaction turbines in low-head installations where it may be critical to develop the total head.

Reaction turbines are widely used in large hydraulic plants, and several manufacturers produce small turbines of this type.

Two types of reaction turbines exist, commonly known as "Francis" and "propeller."

Francis turbines generally operate under higher heads than propeller turbines, although they are in use at some low-head installations. A Francis turbine has a runner with fixed blades. Water enters the turbine in a radial direction, with respect to the shaft, and is discharged axially.



Nohab Tampella Francis turbine: Head range up to 100 m; discharge range up to 30 m^3/s.

The principal components include the runner, a water-supply casing, wicket gates to control the quantity of water admitted, and a draft tube to return the water to the river. The wicket gate assembly, in conjunction with a governor, also regulates unit load and speed and shuts down the unit. The regulating system can be actuated either hydraulically or electrically.

Propeller turbines are generally used for lower heads than Francis turbines. The normal range is from 10 to 120 feet. Typically, a propeller turbine has a vertical shaft, a spiral casing and wicket gates to distribute flow, a draft tube, and fixed runner blades.

The blades of some propeller turbines are adjustable, and when this is the case the turbine is called a Kaplan turbine. The propeller turbine has good efficiency at an optimum flow point, but its efficiency drops off rapidly at higher and lower flows.

The Kaplan turbine has a relatively high efficiency over a wide range of flows.

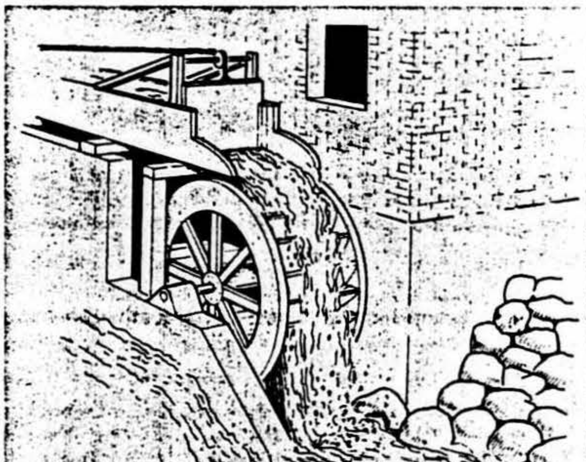
Another variation of the propeller turbine is the axial-flow turbine. Generally, these units have either a horizontal or a slightly inclined shaft, and they may have either fixed or adjustable runner blades.

Three types of turbines fall into this category. These are the rim-type, in which the generator is on the periphery of the turbine runner, the tube-type, in which the generator is located outside the water passage, and the bulb-type, in which the generator is housed in a bulb submerged within the water passage.

Water Wheels

Water wheels are the traditional means of obtaining useful energy from falling water. Their advantages over turbines include their relatively simple construction, their low cost if home-built, and their relative insensitivity to variations in flow.

In addition, debris in the water that can clog a turbine or its protective trashrack can normally pass by a water wheel. Unfortunately water wheels are much less efficient than turbines, and icing can be a problem. Details of water wheels are given in several of the references in the bibliography.



An overshot water wheel. This type of water wheel was extensively used throughout Ontario to provide mechanical power for mills. Conversion to produce electrical power is sometimes practical.

5. Alternative Layouts

As mentioned earlier, there are two basic types of development: concentrated-fall and divided-fall.

The first is more common with low-head development, but generally requires a dam to impound and divert water to the turbine. Since the dam required is often as high as the head being developed, it can be very expensive. However, the length of the pipeline required is generally short, resulting in small head losses and lower costs. Dams must also be designed and operated to pass flood flows, which can also add to the cost.

Environmental considerations may also be a significant factor in such developments. A dam may restrict fish movement and impoundment may alter natural flow patterns, particularly if a pond is used to store water for peak power generation.

With divided-fall installations, development may be practical with only minimal damming and impoundment, and in some instances with no dam at all. Therefore, dam cost may be greatly reduced, but pipeline costs will increase significantly.

Major civil structures must, of course, be properly founded and designed. Hiring an experienced engineering consultant is advisable.

Power lines are a further consideration and they too may add to the cost of the system. If the micro-hydro unit is some distance from the demand, power lines are obviously required.

Bear in mind that the greater the distance, the heavier the wires required if unreasonable power losses are to be avoided. This is especially true of DC systems which require very large conductors to avoid excessive losses. An increase in line voltage also reduces power loss. This should be considered if the line will be much more than 200 feet in length.

6. Estimating the Costs

The cost of a micro-hydro system depends on a number of factors. They include topography, availability of suitable equipment, and the ability of the individual as a do-it-yourselfer. In general, the cost will range upwards from $500 per kilowatt. This low cost implies a do-it-yourself job at a favourable site with an existing dam or one requiring no dam at all.

Here are some general guidelines:

Topography usually dictates the type and extent of work required. Also, at higher heads, lower flows are required to produce equivalent output; this influences the cost because lower flows permit the use of smaller water passages. So, for an installation with a relatively high head, equipment and all structures — including intake, pipeline, headrace, and tailrace — can be smaller than those needed for lower head installation of equal output.

Under even the best conditions, construction costs at a concentrated-fall site where a dam does not have to be built are usually about equal to generating equipment costs.

Additional construction such as a dam, a lengthy penstock, and headrace and tailrace excavations add to both initial and operating costs.

The distance from the powerhouse to the load can also add to the cost. The greater the distance, the larger the conductor must be, or the higher the transmission voltage required.

The price of new generating equipment ranges upwards from $500 per kilowatt. The cost per kilowatt is usually higher for smaller-capacity and for lower-head developments.

New equipment costs may range as high as $5,000 per kilowatt for low-head, small-capacity installations.

Used equipment is considerably less expensive than new equipment, but will usually require more maintenance and have a shorter useful life.

Finally, by doing it yourself, you can save a great deal in construction and equipment installation.

7. Evaluating the Economics

Before you can decide if a micro-hydro system will be economical, you have to determine **exactly** how much it will cost to install. The calculation must take into account the cost of the turbine and generator as well as any pipe, cable, buildings, dam, civil engineering work, permits, legal work — and so on — that will be required. You should also consider the other available sources of electric power and determine their costs.

An important consideration which might influence the economics of a micro-hydro system is the other uses that can be made of the water resource, such as fire suppression and irrigation. There is often very little extra cost involved in developing these uses along with a hydro system.

A characteristic of many renewable energy resources is that, while their "front-end" costs are high, their life-cycle costs may be competitive with conventional energy sources. Micro-hydro, for example, is fairly expensive to install, but, except for small maintenance costs, the system should provide "free" energy for 20 years or longer. The economic analysis of any micro-hydro project should take the life-cycle costs into account.

Proximity to existing Ontario Hydro power lines must also be taken into consideration.

Ontario Hydro currently extends its lines based on minimum density requirements to justify the capital expenditure and future operating expenses. For example, for a year-round residence, Ontario Hydro will extend its line approximately 1,200 feet along a township road at no charge.

If further line is needed, the customer can either contract with Ontario Hydro to supply up to an additional 1200 feet at an annual charge of approximately $0.40 per foot or pay the cost of the line, which might be approximately $4.50 per foot.

It would be advisable to contact the local Ontario Hydro office to find out the costs of extending the line to your site.

So, the further one is from an existing power line, the more attractive the micro-hydro option becomes, all else being equal.

If you have easy access to conventional power, and you have no other uses for the available water, it may be difficult to justify a micro-hydro unit on economic grounds. There are a great many factors that will affect your analysis. Here are some questions likely to arise:

- Should your calculations be based on the life of the micro-hydro unit, which may be 20 years or more, or on some shorter period?
- Are you faced with additional costs to obtain service from the power company?
- How long do you expect to remain at this residence and will the micro-hydro system have market value when you want to sell?
- How much of the installation and maintenance work can you do yourself?
- Is there a possibility of installing a unit larger than your needs and selling the surplus power back to the power company?

One realistic way to analyse your electric power options is to calculate the total cost of each option over a certain period of time and then compute the costs in current dollars. The following example shows how an installation might be evaluated.

The figures presented below reflect the cost of each option in current dollars, commonly known as present worth (PW). The actual numbers reflect the investment needed today, to cover the total cost over the 15-year period. It may be helpful to view the differences calculated as "profit" or "loss" resulting from having selected the micro-hydro option over the utility supply option.

The sample computations use a 10% interest rate (the rate at which you could alternatively invest your money). A summary only is presented below; details of the computation are given in Appendix 2.

Example

- The hydro system is a 6 kW, DC-to-AC battery storage unit.
- The total cost of the system is $10,000.
- The $10,000 is borrowed at 12%.
- Total maintenance is $1,358 ($50 per year with 8 per cent annual cost increases).
- The location where the power is to be used is near existing power lines and no charge will be made to connect the power lines.
- Utility power average cost for 1000 kilowatt hours per month is 4.6 cents per kilowatt hour and increases by 10 per cent per year.
- Average monthly consumption is 1000 kilowatt hours.
- The economic study period is 15 years.
- The value of the micro-hydro unit at the end of the 15-year period is $2,000 (20 per cent of purchase price).
- No tax deductions or credits are used.

To Determine the Present Worth of the Micro-Hydro Plant Option

Step 1 — The installation cost assumed is $10,000. The total cost of the loan at 12 per cent interest is $22,005.30, and, assuming a repayment plan with equal annual payments of $1,467,03, the present worth of the loan is $12,274.06.

Step 2 — To this must be added the present worth of the annual charges. In this example, only maintenance is involved; however, such charges could include taxes, the cost of back-up power, and so on. In this example, the present worth of annual maintenance costs is $661.66.

Step 3 — Deduct the present worth of the salvage value of the plant, which is $478.80.

Step 4 — The result is the present worth of the micro-hydro plant: $12,456.92.

To Determine the Present Worth of the Utility Line Supply Option

Step 1 — The cost of purchasing the required energy for each of the 15 years (1,000 kWh/month x 12 months x $0.046/kWh for the first year, and escalated by 10 per cent per year for subsequent years) is $17,561.00.

Step 2 — Total the present worth of each yearly value to determine the present worth of the utility line supply. This value is $8,280.00.

To Determine Which is the More Attractive Option

Subtract the present worth of the utility line supply from the present worth of the micro-hydro plant. A positive number indicates that the utility line supply is the more economically attractive option whereas a negative number indicates that the micro-hydro plant is more attractive.

In this example, the difference is $4,176.92, indicating that the micro-hydro option will be somewhat more expensive over the 15-year period, assuming all of the conditions mentioned above. A change in any of the assumptions can significantly affect the ecomonics one way or the other.

If, for instance, the household was one mile from the existing power line, and assuming the person was planning to pay for the cost of the line beyond the extension allowance given by Ontario Hydro, the cost of utility power would increase by approximately $18,500. Working through the above calculations, and adding this to the cost of line supply, the difference in present worth between the two options would be -$18,530.03, the micro-hydro system would be the more economically attractive option in this case.

The example shows that micro-hydro can be a viable option; the economics depend heavily on individual conditions. There is also the question of how highly you value independence from traditional energy sources. For some, economic conditions permitting, the option of unplugging from the power grid may be worth a small additional cost.

SAMPLE ECONOMIC ANALYSES

Purchase of Surplus Power by the Power Company

Ontario Hydro is in the process of determining what price it would pay to **buy** power from customers who generate electricity and would like to sell it to the utility. This would include not only micro-hydro sites but also large industrial cogeneration operations with potential loads of more than 5000 kW.

A set of interim rates has been developed reflecting the cost savings to Ontario Hydro of such purchases. The rates have been based on the assumption that the seller would contract to deliver a firm amount of capacity with a guaranteed minimum availability during system peak periods. However, "at will" energy can also be sold. Here, excess energy is sold on an "as available" basis. The maximum output and/or the timing of delivery may vary for "at will" energy sales. Accordingly, the purchase price would be lower than for the guaranteed availability option.

The cost of the special metering equipment necessary and the additional administrative expense indicates that it is not economic to provide for the guaranteed availability option unless 50-100 kW of power are produced. The "at-will" sale of energy is possible for even the smallest producers of power.

The appropriate rate could be obtained from the local Ontario Hydro office.

CONCLUSION

Micro-hydro is a long-term investment. It may be expensive to install and it will take patience to obtain approval for a project. However, once the micro-hydro unit is installed and running, the owner can expect to obtain many years of power at a steady, largely inflation-proof annual cost.

This electric power is valuable both to the individual and to the province.

The individual may eventually obtain electricity more cheaply than he could from a utility.

The province will benefit because less power will have to be generated from imported fossil fuels. The combustion of these fossil fuels creates both environmental and economic costs for the province.

Application of micro-hydro technology in a remote location where electricity currently is provided by diesel generators is a special case. Here, there is an opportunity for direct replacement of a premium fuel by a renewable power source.

Micro-hydro is practical and it can be economic. The Ministry of Energy encourages individuals across Ontario to take up the challenge of harnessing water power. Demonstration installations of micro-hydro units are planned for both northern and southern Ontario starting in 1981. These demonstrations should prove the technical feasibility of very small units under hard Ontario winter conditions. They also will provide an opportunity for interested members of the public to examine an operating micro-hydro unit.

If you still have unanswered questions about micro-hydro power, we would be happy to help. Please contact the Ontario Ministry of Energy, Electric Power Section, 56 Wellesley St. West, Toronto, Ontario M7A 2B7. Telephone (416) 965-9603.

BIBLIOGRAPHY

Some books listed here are currently out of print. Copies may be available at your library.

Cloudburst: A Handbook of Rural Skills and Technology (1973)
Edited by Vic Marks
Cloudburst Press, Ltd.
P.O. Box 90
Mayne Island,
British Columbia, V0N 2J0
$6.45

It contains a 30-page section on micro-hydro development; gives the standard techniques of measuring head and flow; describes do-it-yourself dam building; tells how to build an overshot water wheel and a crossflow turbine and also deals with water wheel design.

A Design Manual for Water Wheels
A VITA publication
VITA
3706 Rhode Island Avenue
Mt. Rainier, Md. 20822
$4.95 (U.S.)

A do-it-yourself booklet intended for developing countries, it has obvious application in North America as well. Deals with design and construction of an overshot water wheel for mechanical power.

Design of Small Dams (1973)
Prepared by the U.S. Department of the Interior
Available from:
Superintendent of Documents
U.S. Government Printing Office
Washington, D.C. 20402
Stock Number 024-003-0019-8
$15.00 (U.S.)

In 816 pages it describes medium-sized and large earth fill dams, site selection, soil sampling, design considerations, construction techniques and environment impacts.

A Handbook of Homemade Power (1974)
By the staff of "Mother Earth News"
Bantam Books
666 Fifth Ave.
N.Y., N.Y. 10019
$2.95 (U.S.)

Available in most book stores, it includes a brief section on hydro power and plans for a small water wheel.

Harnessing Water Power for Home Energy (1978)
By Dermot McGuigan
Garden Way Publishing Co.
Charlotte, Vermont 05445
$4.95 (U.S.)

Describes many aspects of small and micro-scale hydro, gives a number of examples of installations of various types of water wheels and turbines in the United Kingdom and the United States. Manufacturers are listed along with their products and outputs. Equipment costs are often included. It contains a good bibliography.

Harnessing the Turbulence:
Harrowsmith Magazine
(Issue No. 29)
Harrowsmith Magazine
Queen Victoria Rd.
Camden East, Ont. K0K 1J0
$1.75

There are several micro-hydro articles in this issue.

List of Water Powers of the Province of Ontario
Ontario Ministry of Natural Resources
Room 5620
Whitney Block
Queen's Park
Toronto, Ont. M7A 1W3

Under revision.

Low-Cost Development of Small Water-Power Sites (1967)
A VITA publication
VITA
3706 Rhode Island Avenue
Mt. Rainer, Maryland 20822
$2.95 (U.S.)

A 43-page booklet with information on every step in the process of developing small-scale hydro power sites. Descriptions cover water wheels, a small 12" diameter crossflow turbine and the Pelton Wheel. Small earth dam construction is also covered.

Micro-Hydro Power: Reviewing an Old Concept
Prepared by:
Technical Research Staff
The National Center for Appropriate Technology
P.O. Box 3838
Butte, Montana 59702
January 1979
Prepared for: U.S. Department of Energy
$1.30 (U.S.)

A comprehensive study, but geared to the U.S.

Other Homes and Garbage:
Designs for Self-Sufficient Living (1975)
By J. Leckie, G. Masters, H. Whitehouse, and L. Young
Sierra Club Books
530 Bush St.
San Francisco, Ca. 94108
$9.95 (U.S.)

Has a 12-page section on micro-hydro; describes techniques for measuring water flow, simple dam construction, and the basic types of water wheels and turbines.

Producing Your Own Power: How To Make Nature's Own Energy Sources Work For You
Edited by Carol H. Stoner
Rodale Press, Inc.
33 E. Minor St.
Emmaus, Pa. 18049
$3.95 (U.S.)

Deals with a variety of renewable energy systems. Includes sections on water power; measuring head and flow; calculating power available; a five-page piece on determining channel, pipe, and other head losses; small earth and rock dams; water wheels and turbines and the VITA hydraulic ram.

BIBLIOGRAPHY

Reference Index: Hydrometric Map Supplement
Inland Waters Directorate
Environment Canada
Water Resources Branch
Water Survey of Canada
Ottawa, Ont. K1A 0E7
FREE

Contains flows for rivers across Canada.

Site Owner's Manual for Small Scale Hydropower Development (1980)
Prepared by:
Polytechnic Institute of New York
Prepared for: New State Energy Research and Development Authority
Order from:
National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Rd.
Springfield, Va. 22161
Report No. 79-3
$11.00 (U.S.)

Small and Micro Hydroelectric Power Plants — Technology and Feasibility: (1980)
Edited by J. Paul
Noyes Data Corporation
118 Mill Road
Park Ridge, N.J. 07656
$42.00 (U.S.)

One of the most comprehensive reports available on the subject.

Use of Weirs and Flumes in Stream Gauging: Technical Note No. 117
Unipub
345 Park Ave. S.
New York, New York 10010
Order No. W93
$10 plus $1 shipping (U.S.)

Describes techniques for making an accurate assessment of stream water flow rates.

Water Measurement Manual
Prepared by:
Department of the Interior
Available from:
Superintendent of Documents
U.S. Govt. Printing Offices
Washington, D.C. 20402
Stock No. 024-003-00148-1

Under revision.

APPENDICES

NOTE: Appendices 2, 4 and 5 are adapted from:
National Center for Appropriate Technology, 1979.
"Micro-Hydro Power, Reviewing an Old Concept."
DOE/ET/01752-1:60 pp.

APPENDIX 1

Glossary

ASYNCHRONOUS GENERATOR:
Similar to the synchronous generator except that it must be hooked up to an independent power grid to produce usable power. (See **GENERATOR and SYNCHRONOUS GENERATOR**).

AXIAL FLOW TURBINE:
A reaction turbine through which the direction of flow is primarily parallel to the turbine shaft.

CAVITATION:
A phenomenon associated with liquids in motion past solid surfaces in which vapor bubbles form in areas of low pressure and then collapse suddenly in areas of higher pressure, resulting in shock waves which can damage solid surfaces.

CROSSFLOW TURBINE:
A drum-shaped hydroelectric turbine with vanes around its circumference which permit the water to enter from one side, cross through the hollow centre, and exit from the other side.

FOREBAY;
See **HEADRACE**

FRANCIS TURBINE:
A mechanical device used to convert revolving mechanical energy into electrical energy. (See **ASYNCHRONOUS GENERATORS** and **SYNCHRONOUS GENERATORS**).

GOVERNOR:
A mechanical or electronic device for automatically controlling the speed of the turbine by regulating the supply of water.

HEAD POND:
The pond immediately upstream from the hydro plant from which additional flow may be taken for peak generation and which can refill during periods of lower electrical demand.

HEADRACE:
A channel through which water passes to reach the hydro plant intake.

KAPLAN TURBINE:
A propeller turbine on which the pitch of the blades is adjustable to allow efficient use of the available water.

MICRO-HYDRO SYSTEM:
A hydroelectric installation with a capacity of 100 kW or less, including all components required to convert the potential energy in a stream or river to electrical energy at the user's location.

PELTON WHEEL:
A type of impulse turbine with buckets mounted on the rim of the wheel which are struck by a high-velocity jet of water to rotate the wheel.

PENSTOCK:
A pipeline used for carrying water to a water wheel.

RUNNER:
The rotating element of the turbine which converts hydraulic energy into mechanical energy.

SLUICE:
An artificial channel or passage for water with a gate or valve at its head to regulate flow.

SPILLWAY:
A passageway or channel to carry off excess water around a dam.

SYNCHRONOUS GENERATOR:
A machine which converts rotating mechanical energy into usable AC electrical power independent of a power grid. (See **GENERATOR** and **ASYNCHRONOUS GENERATOR**).

TAILRACE:
A channel through which the water flows out of a hydro plant.

TRASHRACKS:
A screen on the hydro plant intake that blocks debris from entering the turbine.

TURBINE:
A mechanical device used to convert the potential energy of falling water into electrical energy.

WICKET GATE:
Flow control gates located in a circle around a turbine and normally controlled by a governor.

APPENDIX 2

Flow and Head Efficiency

Flow is the quantity of water available, and it is rarely constant. Most rivers, even when they have large reservoirs, are subject to periods of drought as well as periods of heavy rain and resultant flood flows. These natural characteristics are a major consideration when selecting hydroelectric equipment, and are as important as is the available head. Heavy rain, which causes flood runoff, may result in the head at a site being reduced to almost nothing. Conversely, periods of drought may reduce the water supply to unacceptable levels.

Because of the great variability in natural stream-flows, a hydrologic record going back as far as possible is desirable as a basis for analysing the potential energy output of a site. However, if no stream-flow records exist for a particular site, then an estimate of flow can be made using one of the methods described.

Low flow is critical to power plant capacity. Measurements of stream-flow should be made during the summer when high rates of evaporation reduce stream-flow to a minimum. Storm runoff should be avoided by taking measurements seven days after a storm.

How to Determine Flow

Stream-flow records are maintained for many Ontario rivers. Although the actual stream gauging stations are operated by various agencies, a complete record is maintained by the Inland Waters Branch of Environment Canada in Guelph. Your local library may have their publication, "Reference Index: Hydrometric Map Supplement". Your district office of the Ministry of Natural Resources may be able to provide you with useful stream-flow figures.

Although records of 15 to 20 years are desirable, many existing gauging stations have not been operating that long. Nevertheless, any stream-flow record at or near a proposed hydroelectric development provides a more accurate estimate of flows than either of the techniques described below. However, if no stream-flow gauging data are available, then one of the following methods, applied during the low-flow period, should provide a reasonable estimate of the flow available for hydroelectric development.

Flow Measurement

In order to adequately assess the minimum continuous power output to be expected from your hydro unit, the minimum quantity of water that will pass through the system must be determined. So, it is important to know both the minimum flow rate of your stream and what portion of this flow can be used for power generation.

The percentage of the minimum flow that may be temporarily diverted for power generation is defined during the government approval process.

Measurement of Flow in a Stream

Area-Velocity Method

To estimate the flow in an ungauged stream the following procedure may be used. First, both the cross-sectional area of the stream and the velocity of flow in the stream must be determined.

To measure the cross-sectional area of the stream the following procedure (Figure A) may be used.

Step 1 — Select an easily measured section of the stream with fairly uniform depth and width.

Step 2 — Measure the width of the stream.

Step 3 — Measure and record the depth at equal intervals across the channel.

Step 4 — Compute the average depth by adding the measurements taken in Step 3 and dividing by the number of measurements taken.

Step 5 — Calculate the cross-sectional area by multiplying the average depth by width.

NOTE: If all measurements are in feet, the cross-sectional area will be in square feet.

To determine the stream-flow velocity, use the same uniform section of the stream and follow the steps outlined below:

Step 1 — Insert stakes at two points along the stream and measure the distance between them: 25 feet is a reasonable distance.

Step 2 — One person should drop a float (a bottle partially filled with stones or an orange, make good floats) in the centre of the stream opposite the upstream point and a second person should carefully time the seconds required for the float to pass the downstream point. Repeat several times to obtain an average time.

Step 3 — Compute the stream surface velocity by dividing the distance established in Step 1 by the period of time measured in Step 2.

Note: If the distance established is in feet and the time period is in minutes, the computed velocity will be in feet per minute.

Step 4 — The average velocity of flow throughout the stream section is less than the centre-line surface velocity because of friction losses due to channel roughness. To allow for this, the stream surface velocity computed in Step 3 should be multiplied by 0.8 to determine the average stream velocity.

Now, to calulate the stream-flow, multiply the average cross-sectional area, determined above, by the average velocity of flow. In mathematical terms this is:

$$Q = AV$$

where:

Q = flow
A = cross-sectional area of the stream
V = average velocity of flow

Once again, it should be noted that these measurements of flow are best taken during the dry season, since the flow during this season may limit the capacity of the proposed hydroelectric installation. Furthermore, government approval may not be possible for a power plant that utilizes the entire flow, even during low-flow periods, and this may further reduce the capacity that may be installed.

APPENDICES

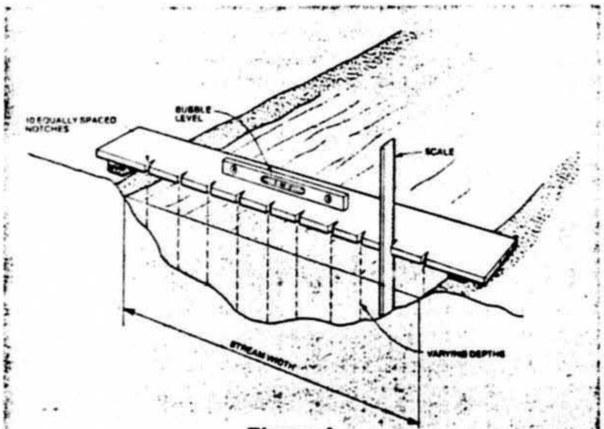


Figure A

Example

To determine the cross-sectional area (A) of a stream, multiply the channel width (w) — in this example, 10 feet — by the average depth (d) of the stream. The calculations to determine the average depth are below::

$$(d) = \frac{d1 + d2 + d3 + d4 + d5}{5}$$

$$= \frac{1.2 + 2.5 + 3.3 + 2.2 + 0.8}{5} = 2 \text{ feet}$$

$$A = w \times d$$

$$= 10 \times 2$$

$$= 20 \text{ square feet}$$

To determine the average velocity of stream-flow:

Say the distance marked off is 25 feet, and it takes 12 seconds, or 0.2 minutes, for the float to travel this distance, then:

With:

Surface velocity expressed as $\mathbf{V_S}$

Average velocity expressed as **V**

fpm = feet per minute

0.8 as a variable factor based on the resistance to the water's flow caused by the characteristics of the stream bed and shoreline. By multiplying it by the surface velocity (V_S) you will arrive at the average velocity (V).

$$(V_S) = \frac{\text{distance (feet)}}{\text{time (minutes)}} = \frac{25}{0.2} = 125 \text{ fpm} = V = 125 \times 0.8 = 100$$

To determine stream-flow:

$$Q = A \times V$$

$$= 20 \times 100$$

$$= 2{,}000 \text{ cubic feet per minute}$$

Weir Method

This is an alternative method for determining stream-flow. It is accurate and can be used to measure the flow rate of any stream. It is particularly advantageous for flow measurements in shallow streams where a weighted float would have difficulty floating freely. However, it is also a more complicated technique for measuring flow.

Essentially, a temporary dam structure is built across the stream perpendicular to the flow, with a rectangular notch or spillway of controlled proportions in the centre section. This notch has to be large enough to take the maximum flow of the stream during the period of measurement, so make some rough estimate of the stream-flow prior to building the weir. The notch width should be at least three times its height and the lower edge should be perfectly level. The lower edge and the vertical sides of the notch should be bevelled with the sharp edge upstream. The whole structure can best be built out of timber with all edges and the bottom sealed with clay, earth, and sandbags to prevent leakage. A typical weir is illustrated in Figure B.

In order to measure the flow of water over the weir, you have to set up a simple depth gauge. This is done by driving a stake in the stream bed at least five feet upstream from the weir, until a pre-set mark on the stake is precisely level with the bottom edge of the notch. The depth of water on this stake, above the pre-set mark, will indicate the flow rate of water over the weir. Refer to a "weir table" in order to determine this flow rate.

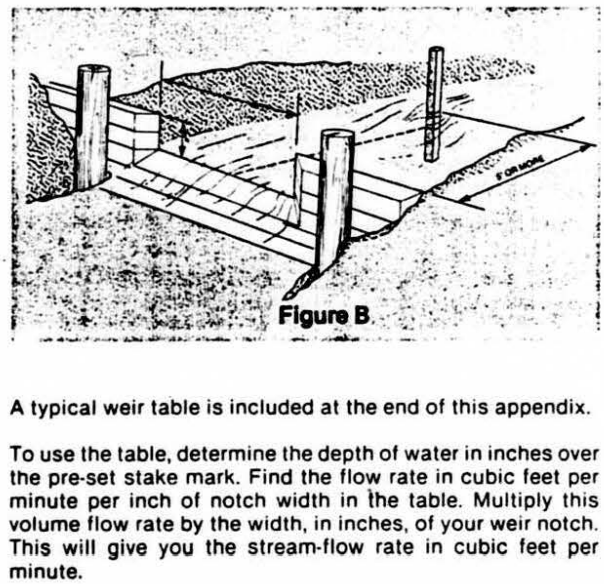


Figure B

A typical weir table is included at the end of this appendix.

To use the table, determine the depth of water in inches over the pre-set stake mark. Find the flow rate in cubic feet per minute per inch of notch width in the table. Multiply this volume flow rate by the width, in inches, of your weir notch. This will give you the stream-flow rate in cubic feet per minute.

For example:
Suppose your weir has a notch width of 30 inches. The depth of the water on the stake above the pre-set mark is 6.25 inches. On the weir table, read opposite 6.25 inches to the flow rate of 6.28 cubic feet per minute per inch of notch width. The flow rate of the total stream is then 6.28 cubic feet per minute x 30 inches **or** 188.4 cubic feet per minute.

When you have the weir in place, you can take readings at your convenience. If you are going to use the weir for an extended period of time, it is important to frequently check the watertightness of the sides and bottom.

Head

The head, once again, is the vertical distance the water falls at the site. The greater the distance, or head, the more potential power there is.

The gross head is the difference between the water levels both upstream and downstream, and is fairly easy to measure.

The net head of the power plant is equal to the gross head minus head losses due to friction and other disturbances in water passage to and from the turbine.

Keeping these head losses to a minimum will enhance potential power plant output. As a rule of thumb, if upstream and downstream water levels are relatively constant, net head should be assumed as equal to gross head minus 5 per cent for conduit head loss. If water levels vary a great deal, more detailed studies are required to determine the net or effective head.

How to Measure Head

Any good surveyor can be hired to determine the head. Ask him, or her, for the vertical distance between the water source, or proposed intake location, and the proposed location of the power plant.

If you know how to use standard surveying equipment such as a transit or a surveyor's level and levelling rod, borrow or rent what you need and get a friend to help do your own measurements.

Another do-it-yourself technique requires a carpenter's level, some sort of stand to raise the level a few feet off the ground, and a tape measure. The method is described below and illustrated in Figure C.

Step 1 — Set the level on the stand; make sure it is level and that its upper edge is either at the same elevation as the water source or a known vertical distance above the water surface.

Step 2 — Sight along the upper edge of the level to a spot on a nearby tree, rock or building that is farther down the hill and can be reached for measuring. Note this precise spot on the object and mark it (Point A in the diagram).

Step 3 — Move your level and stand down the hill slope and set it up again so the upper edge of the level is below Point A

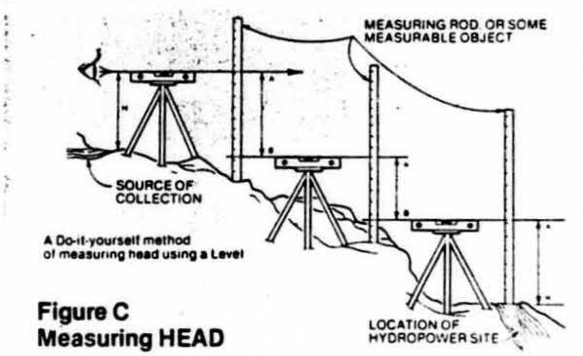


Figure C
Measuring HEAD

WEIR TABLE

Depth on stake (inches)	Cubic feet per minute per inch of notch width	Depth on stake (inches)	Cubic feet per minute per inch of notch width
1	0.40	12.25	17.26
1.25	0.55	12.5	17.78
1.5	0.74	12.75	18.32
1.75	0.93	13	18.87
2	1.14	13.25	19.42
2.25	1.36	13.5	19.97
2.5	1.59	13.75	20.52
2.75	1.83	14	21.09
3	2.09	14.25	21.65
3.25	2.36	14.5	22.22
3.5	2.63	14.75	22.70
3.75	2.92	15	23.38
4	3.22	15.25	23.97
4.25	3.52	15.5	24.56
4.5	3.83	15.75	25.16
4.75	4.16	16	25.76
5	4.50	16.25	26.36
5.25	4.84	16.5	26.97
5.5	5.18	16.75	27.58
5.75	5.54	17	28.20
6	5.90	17.25	28.82
6.25	6.28	17.5	29.45
6.5	6.65	17.75	30.08
6.75	7.05	18	30.70
7	7.44	18.25	31.34
7.25	7.84	18.5	31.98
7.5	8.25	18.75	32.68
7.75	8.66	19	33.29
8	9.10	19.25	33.94
8.25	9.52	19.5	34.60
8.5	9.96	19.75	35.27
8.75	10.40	20	35.94
9	10.86	20.25	36.60
9.25	11.31	20.5	38.28
9.5	11.77	20.75	37.96
9.75	12.33	21	38.65
10	12.71	21.25	39.34
10.25	13.19	21.5	40.04
10.5	13.67	21.75	40.73
10.75	14.16	22	41.43
11	14.67	22.25	42.13
11.25	15.18	22.5	42.84
11.5	15.67	22.75	43.56
11.75	16.20	23	44.28
12	17.26	23.25	45.00
		23.5	45.71
		23.75	46.42
		24	47.18

on the first object, as in the drawing. Mark this point B and measure and record the vertical distance from A to B. Now sight along the upper edge of the level in the opposite direction to another object that is farther down the hill.

Step 4 — Repeat this procedure until you end up at the same elevation as the proposed power plant site.

Step 5 — If more than one set-up was required, add all the vertical distances A-B. If your first set-up was above the water surface, subtract the vertical distance between the water surface and the upper edge of the level from the sum of the vertical distances.

You now have the total head.

Efficiency

Power-plant efficiency will vary according to the efficiency of the component parts. Typical efficiencies of major components in a micro-hydro system are listed below. More precise figures are generally available from the manufacturers. It is worth noting that turbines are much more efficient than water wheels.

By multiplying the efficiencies of the various components in the system, the overall efficiency can be estimated.

Typical Efficiency Ranges for Micro-Hydro Equipment

Component		Efficiency Range (%)
Water Wheels	— Undershot	25-45
	— Breast	35-65
	— Poncelet	40-60
	— Overshot	60-75
Turbines	— Reaction	80-90
	— Impulse	80-95
	— Crossflow	60-85
Generators	— Synchronous	90-95
	— Induction	95-98
	— Direct Current	90-98
Speed-Increasers	— Gear Box	90-95
	— Belt Drive	80-90
Inverters		85-95
Gemini Converters		85-95
Batteries		70-80

APPENDIX 3

Economic Analysis

Comparison of a 6 kW micro-hydro installation against utility power line supply.

Assumptions:

1. The micro-hydro system has a 6 kW capacity and is a DC-to-AC battery storage system.
2. The cost of the micro-hydro system is $10,000 (This cost is realistic for a system using new equipment but requiring little new construction).
3. The hydro plant is financed by a 15-year 12 per cent loan.
4. Total maintenance cost is $1,358 over 15 years ($50 per year with 8 per cent annual cost increase).
5. The location the power is to be used is near existing power lines and no additional costs are required to connect to the utility lines.
6. Average monthly consumption is 1000 kilowatt hours.
7. Utility power average costs start at 4.6 cents per kilowatt hour and increase by 10 per cent annually.
8. The hydro unit is worth $2,000 (20 per cent of the purchase price) at the end of the 15-year period.
9. No tax deduction or credits are used.

Step 1
Present Worth of the Micro-Hydro Plant

Installation cost is $10,000 and the loan at 12 per cent interest for 15 years requires equal annual payments of:

A = $10,000 x (F/P. 12%, 15) x (A/P, 12%, 15)

= $10,000 x 5.474 x 0.0268

= $1,467.03

F/P and A/P are taken from standard compound interest tables. To determine the annual payments for larger or smaller loans just substitute the actual loan for the $10,000 used in the example, providing that interest and period are 12% and 15 years respectively. Otherwise interest tables should be used to determine the correct factors.

1. Present worth (PW) of loan repayment at 10 per cent rate of return (rate at which funds could alternatively be invested) is as follows:

Year (n)	Payment (A) ($)	Factor (P/F: 10%, n) At 10 Per Cent	PW ($)
1	1,467.02	1	1,467.03
2	1,467.02	0.9091	1,333.68
3	1,467.02	0.8264	1,212.35
4	1,467.02	0.7513	1,102.18
5	1,467.02	0.6830	1,001.98
6	1,467.02	0.6209	910.88
7	1,467.02	0.5645	828.14
8	1,467.02	0.5132	752.88
9	1,467.02	0.4665	684.37
10	1,467.02	0.4241	622.17
11	1,467.02	0.3855	565.54
12	1,467.02	0.3505	514.19
13	1,467.02	0.3186	467.40
14	1,467.02	0.2897	425.00
15	1,467.02	0.2633	386.27
			12,274.06

Step 2

Present worth of annual maintenance charges is as follows:

Year (n)	Payment (A)	At 10 Per Cent Factor (P/F,* 10%, (n)	PW
	($)		($)
1	50.00	1	50.00
2	54.00	0.9091	49.09
3	58.32	0.8264	48.20
4	62.99	0.7513	47.32
5	68.02	0.6830	46.46
6	73.47	0.6209	45.62
7	79.34	0.5645	44.79
8	85.69	0.5132	43.98
9	92.54	0.4665	43.17
10	99.95	0.4241	42.39
11	107.95	0.3855	41.61
12	116.58	0.3505	40.86
13	125.91	0.3186	40.11
14	135.98	0.2897	39.39
15	146.86	0.2633	38.67
	1,357.60		661.66

*These factors are taken from standard compound interest tables.

Step 3

Present worth of salvage value of plant 2,000 X (P/F, 10%, 15) = 2,000 X 0.2394 = $478.80.

Step 4

Present worth of hydro plant = 12274.06 + 661.66 - 478.80 = $12,456.92.

Step 5

Present worth of purchase of energy from utility

Year (n)	Payment (A)	At 10 Per Cent Factor (P/F, 10%, n)	PW
	($)		($)
1	552.00	1.0	552.00
2	607.00	0.9091	552.00
3	668.00	.8264	552.00
4	735.00	.7513	552.00
5	809.00	.6830	552.00
6	890.00	.6209	552.00
7	979.00	.5645	552.00
8	1,077.00	.5132	552.00
9	1,185.00	.4665	552.00
10	1,304.00	.4241	552.00
11	1,434.00	.3855	552.00
12	1,577.00	.3505	552.00
13	1,735.00	.3186	552.00
14	1,909.00	.2897	552.00
15	2,100.00	.2633	552.00
	17,561.00		8,280.00

Step 6

Subtracting the present worth of line supply from micro-hydro supply = $12,456.92 - $8,280.00 = $4,176.92

In this example, the micro-hydro option would be somewhat more expensive over a 15-year period, assuming all of the conditions mentioned beforehand. A change in any of the assumptions can significantly affect the economics one way or the other. For example, if the location is one mile from an existing power line, an additional charge of approximately $18,500 (4,100 feet over extension allowance x $4.50 per foot) would be required before power could be obtained from the utility.

This added loan of $18,500 at 12% for 15 years requires equal annual payments of

A = 18,500 x 5.474 x .0268 = $2,714.00.

The present worth of the loan repayment is calculated in the same manner as shown previously and equals $22,706.95.

Adding this to the cost of line supply, above, the present worth of the utility supply would be:
$8,280.00 + $22,706.95 = $30,986.95.

Now, subtracting the present worth of line supply from micro-hydro supply:
$12,456.92 - $30,986.95 = – $18,530.03.

Clearly, with the inclusion of the additional charge in this example, the micro-hydro option is the more attractive one.

A similar technique can be used for an economic comparison of any two energy supply alternatives.

APPENDICES

APPENDIX 4

Cautions and Suggestions

The following list is presented to make installation easier and help you avoid future troubles.

In the final design stage, be sure to:

1. Consider your stream bed loading conditions. Silt and rocks coming down the stream, particularly during periods of high runoff, can cause intake clogging or even destruction of the intake pipes.

2. Size the pipe so that it is capable of handling the volume flow rate that you require. Any responsible pipe supplier can provide the correct size for the expected flow conditions.

3. Route the pipeline, from intake to the turbine, so that it contains the minimum number of bends. Do not use elbows of 45 degrees (or greater) in the pipeline. Otherwise, there will be too much strain on the pipe and excessive friction losses.

4. Keep a downhill slope in the pipe at all times (except for the initial siphon intake, if used) to avoid air locks and silt deposit.

5. Do not let the water velocity in PVC pipe get much above 5 feet per second. Above this line velocity other design considerations come into play that the do-it-yourselfer is not usually prepared to deal with.

6. Size the pipe in order to maintain about 5 feet per second line velocity to avoid excessive ice build-up in the pipe. If the line velocity is much less than this and the system is to be installed in an area where winters are severe, consider insulating or burying the pipe.

7. Consider installing a water by-pass above the turbine in case the water is needed for fire control.

8. Locate the DC turbine and generator adjacent to the point of use. This is important in order to keep electrical transmission lines as short as possible so that the line losses are kept to a minimum.

9. Plan to install the system in warmer weather, or at least not under freezing conditions if at all possible.

When obtaining your equipment, take these factors into account:

1. Deal with a reputable supplier. There is some poor equipment around. Buyer beware!

2. Expect delays in getting quotes and deliveries from equipment suppliers, since none of them is currently very big and are usually quite busy.

3. Obtain pipe with a suitable pressure rating; don't buy seconds.

4. Obtain a good trash control system for the intake. A screen mesh should be used that has openings smaller than the minimum nozzle diameter that leads into the turbine. This way, the only solid particles that can come down the pipe will be small enough to pass through the nozzle without clogging it.

During installation:

1. Be sure to follow the manufacturer's or supplier's instructions and suggestions.

2. Watch for rocks, and place them carefully when burying PVC pipe.

3. Use gate valves wherever valving is necessary. Other kinds of valving allow the water to be turned off too quickly, causing potentially dangerous water hammer or "banging pipes" effects.

4. Use standard house wiring procedures with the electrical hook-up. Go to your local bookstore and pick up an appropriate do-it-yourself book or hire a local electrician.

Once the system is operational, when you have to close valves, be sure to do so **slowly.** Closing a valve too quickly can cause a shock wave (a high pressure wave) that can damage the pipe.

APPENDIX 5

Manufacturers and Suppliers

Canadian
Robert Lee
Waterwheel Erectors Ltd.
P.O. Box 246
Welland, Ontario
L3B 5P4
(416) 735-5122

Claude Aleire
Dominion Bridge Sulzer Inc.
555 Notre Dame St.
Lachine, Quebec
H8S 2B1
(514) 634-3551

Mike Wilson
Barber Hydraulic Turbine
Barber Point, P.O. Box 340
Port Colborne, Ontario
L3K 5W1
(416) 834-9303

F. W. E. Stapenhorst
285 Labrosse Ave.
Point Claire, Quebec
H9R 1A3
(514) 695-8230

Alvin Beeler
L & S Power Company Ltd.
Box 90
Whitney, Ontario
K0J 2M0
(705) 637-5534

J. S. McAulay
Allis-Chalmers
3625 Dufferin Street
Downsview, Ontario
M3K 1Z2
(416) 789-5337

APPENDICES

Manufacturers and Suppliers

B. Tripp
Highlands Energy Systems Ltd.
R. R. #5
Orangeville, Ontario
L9W 2Z2
(519) 941-5041

Dependable Turbine Ltd.
1244 Boundary Road
Vancouver, B.C.
V5K 4T6
(604) 461-3121

Small Hydro Electrics Canada Ltd.
Box 54
Silverton, B.C.
V0G 2B0
(604) 358-2406

A. Nicholl
Solace Energy Centre Inc.
2425 Main Street
Vancouver, B.C.
V5T 3E1
(604) 879-5258

David Buchanan
Ingersoll-Rand Canada
255 Lesmill Road
Toronto, Ontario
M3B 2V1
(416) 445-4470

Leroy Somer Canada Ltd.
337 Deslauriers
Ville St. Laurent
Quebec
H4N 1W5
(514) 378-0151

Hayward Tyler
1 Vulcan Street
Rexdale, Ontario
M9W 1L3
(416) 243-1400

Dave de Montmorency
Galt Energy Systems Ltd.
57 Victoria Avenue
P.O. Box 1354
Cambridge, Ontario
N1R 3B0
(519) 653-2531

International

Independent Power Developers
Route 3, Box 285
Sandpoint, Idaho
83864

The James Leffel Company
Springfield, Ohio
45501

Gilbert, Gilkes & Gordon Ltd.
Westmoreland, England
LA9 7B7

Small Hydro Electric Systems
P.O. Box 124
Custer, Washington
98240

Ossberger Turbinenfabrik
Weissenberg
Pastfach 425
Bayern, West Germany

Barata Metal Works & Engineering PT
Mgagel (109)
Surabaya, Indonesia

Jyoti Ltd.
Industrial Area
P.O. Chemical Industries
R.C. Dutt Road
Baroda 390 003, India

Westward Mouldings Ltd.
Greenhill Works
Delaware Road
Gunnislake, Cornwall, England

Campbell Water Wheel Company
420 South 42nd Street
Philadelphia, Pennsylvania
19104

Maschinenfabrik Kossler GMBH
A-3151 St. Pölten
St. Georgen, Austria

Karlstads Mekaniska Weskstad
Fack S-681 01
Kristinehamn, Sweden

Elektro GMBH
St. Gallerstrasse 27
Winterthur, Switzerland 8400

Canyon Industries
5346 Mosquito Lake Road
Deming, Washington
98244

Briau S.A.
BP 43
37009 Tours Cedex, France

Northern Water Power Co.
P.O. Box 49
Harrisville, New Hampshire
03450

Land & Leisure Services
Priority Land
St. Thomas, Launceston
Cornwall, England

Alaska Wind and Power
P.O. Box G
Chigiak, Alaska
99567

Pumps, Pipes and Power
Kingston Village
Austin, Nevada
89310

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Manufacturers and Suppliers

Bell Hydroelectric
3 Leatherstocking Street
Cooperstown, New York
13326

Balaju Yuantra Shala (P) Ltd.
Balaju, Katmandu, Nepal

Maine Hydroelectric Development Groups
Goose River, Maine

Miscellaneous Equipment Suppliers

Tom Adair
Westburn Electric Supply
R. R. #1
Kearney, Ontario
P0A 1M0
Zenith 48240

Douglas Fleming
Reliance Electric Ltd.
678 Eric Street
Stratford, Ontario
(519) 271-3630

H. M. Barnett
Canadian General Electric
1900 Eglinton Avenue East
Scarborough, Ontario
M1L 2M1
(416) 751-3220

Westinghouse Canada Inc.
55 Goldthorne
Toronto, Ontario
(416) 445-0550

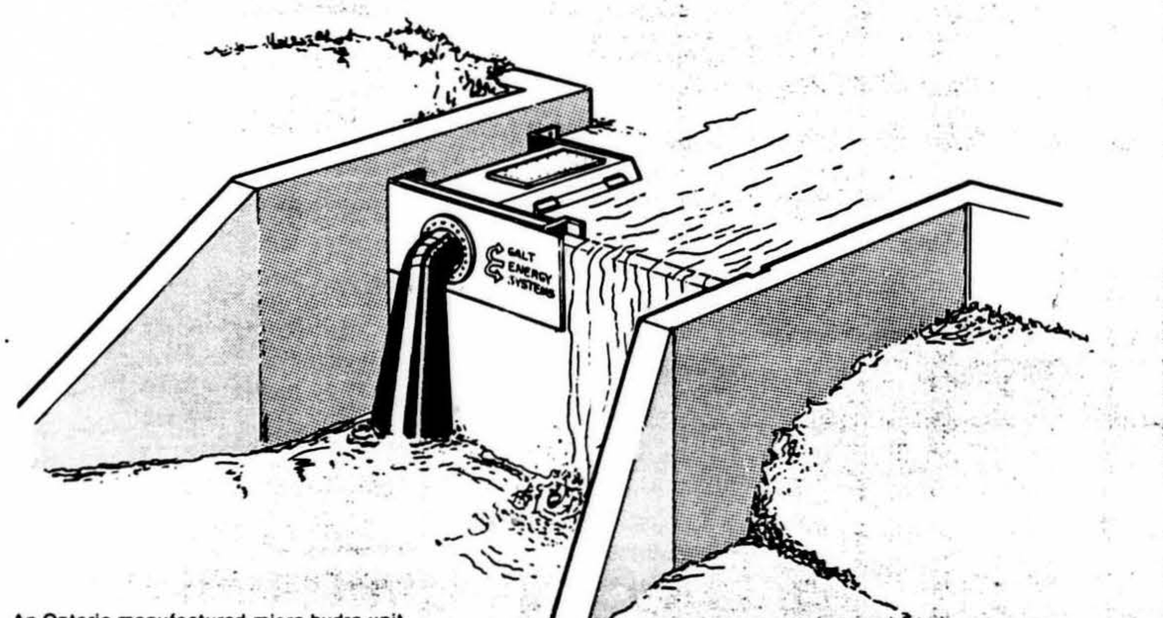
James Smith
Canbar Products Ltd.
Waterloo, Ontario
N2J 4A7
(519) 886-2880

Windworks
Box 329, Route 3
Mukwonago, Wisconsin
53149

Lima Electric Company Inc.
200 East Chapman Road
Box 918
Lima, Ohio
45802

Woodward Governor Company
5001 N. 2nd Street
Rockford, Illinois
61101

Natural Power, Inc.
New Boston, New Hampshire
03070



An Ontario manufactured micro-hydro unit.
Source:
Sketch Courtesy of Galt Energy Systems — Cambridge, Ont.

APPENDIX 6 Ministry of Natural Resources

District Office	Address	Telephone
Algonquin Park	Box 219 Whitney, Ontario K0J 2M0	(705) 637-2780
Atikokan	108 Saturn Avenue Atikokan, Ontario P0T 1C0	(807) 597-6971
Aylmer	353 Talbot Street West Aylmer, Ontario N5H 2S8	(519) 773-9241
Bancroft	Box 500 Bancroft, Ontario K0L 1C0	(613) 332-3940
Blind River	Box 190 62 Queen Street Blind River, Ontario P0R 1B0	(705) 356-2234
Bracebridge	Box 1138 Bracebridge, Ontario P0B 1C0	(705) 645-5244
Brockville	101 Water Street West Brockville, Ontario K6V 5Y8	(613) 342-8524
Cambridge	Box 2186 Cambridge, Ontario N3C 2W1	(519) 658-9355
Chatham	Box 1168 435 Grand Avenue West Chatham, Ontario N7M 5L8	(519) 354-7340
Chapleau	34 Birch Street Chapleau, Ontario P0M 1K0	(705) 864-1710
Cochrane	Box 730 2 Third Avenue Cochrane, Ontario P0L 1C0	(705) 272-4365
Cornwall	Box 1759 113 Amelia Street Cornwall, Ontario K6H 5V7	(613) 933-1774
Dryden	Ontario Government Bldg. Box 3000 Dryden, Ontario P8N 3B3	(807) 223-3341
Espanola	Box 1340 148 Fleming Street Espanola, Ontario P0P 1C0	(705) 869-1330
Fort Francis	922 Scott Street Fort Frances, Ontario P9A 1J4	(807) 274-5337
Geraldton	Box 640 Geraldton, Ontario P0T 1M0	(807) 854-1030

District Office	Address	Telephone
Gogama	Box 129 Gogama, Ontario P0M 1W0	(705) 894-2000
Hearst	Box 670 631 Front Street Hearst, Ontario P0L 1N0	(705) 362-4346
Huronia	Midhurst, Ontario L0L 1X0	(705) 728-2900
Ignace	Box 448 Ignace, Ontario P0T 1T0	(807) 934-2233
Kapuskasing	6 Government Road Kapuskasing, Ontario P5N 2W4	(705) 335-6191
Kirkland Lake	Box 129 Swastika, Ontario P0K 1T0	(705) 642-3222
Kenora	Box 5080 808 Robertson Street Kenora, Ontario P9N 3X9	(807) 468-9841
Lanark	Box 239 Lanark, Ontario K0G 1K0	(613) 259-2942
Lindsay	322 Kent Street West Lindsay, Ontario K9V 4T7	(705) 324-6121
Maple	Maple, Ontario L0J 1E0	(416) 832-2761
Minden	Minden, Ontario K0M 2K0	(705) 286-1521
Moosonee	Box 190 Moosonee, Ontario P0L 1Y0	(705) 336-2987
Napanee	1 Richmond Blvd. Napanee, Ontario K7R 3S3	(613) 354-2173
Niagara	Box 1070 Hwy. 20 Fonthill, Ontario L0S 1E0	(416) 892-2656
Nipigon	Box 970 Nipigon, Ontario P0T 2J0	(807) 887-2120
North Bay	Box 3070 North Bay, Ontario P1B 8K7	(705) 474-5550
Ottawa	Ramsayville, Ontario K0A 2Y0	(613) 822-2525

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District Office	Address	Telephone
Owen Sound	611 Ninth Avenue East Owen Sound, Ontario N4K 3E4	(519) 376-3860
Parry Sound	4 Miller Street Parry Sound, Ontario P2A 1S8	(705) 746-2401
Pembroke	Box 220 Riverside Drive Pembroke, Ontario K8A 6X4	(613) 732-3661
Red Lake	Box 323 Hwy. 105 Red Lake, Ontario P0V 2M0	(807) 727-2531
Sault Ste. Marie	Box 130 69 Church Street Sault Ste. Marie, Ontario P6A 5L5	(705) 949-1231
Simcoe	645 Norfolk Street North Simcoe, Ontario N3Y 3R2	(519) 426-7650
Sioux Lookout	Box 309 Sioux Lookout, Ontario P0V 2T0	(807) 737-1140

District Office	Address	Telephone
Sudbury	Box 3500, Stn. A. Sudbury, Ontario P3A 4S2	(705) 522-7823
Temagami	Box 38 Temagami, Ontario P0H 2H0	(705) 569-3622
Terrace Bay	Box 280 Terrace Bay, Ontario P0T 2W0	(807) 825-3205
Thunder Bay	Box 5000 Thunder Bay 'F', Ontario P7C 5G6	(807) 475-1501
Timmins	896 Riverside Drive Timmins, Ontario P4N 3W2	(705) 267-7951
Tweed	Metcalfe Street Tweed, Ontario K0K 3J0	(613) 478-2330
Wawa	Box 1160 Wawa, Ontario P0S 1K0	(705) 856-2396
Wingham	R.R. #5 Wingham, Ontario N0G 2W0	(519) 357-3131

Regional Office	Address	Telephone
Algonquin Regional Office	Brendale Square P.O. Box 9000 Huntsville, Ontario P0A 1K0	(705) 789-9611
Central Regional Office	10670 Yonge Street North Richmond Hill, Ontario L4C 3C9	(416) 884-9203
Eastern Regional Office	Provincial Government Building Concession Road Kemptville, Ontario K0G 1J0	(613) 258-3413
Northern Regional Office	140 Fourth Avenue P.O. Box 3000 Cochrane, Ontario P0L 1C0	(705) 272-4287
North Central Regional Office	Ontario Government Building 435 James Street South P.O. Box 5000 Thunder Bay 'F', Ontario P7C 5G1	(807) 475-1261

Regional Office	Address	Telephone
Northeastern Regional Office	174 Douglas Street West Sudbury, Ontario P3E 1G1	(705) 673-1111
Southwestern Regional Office	1106 Dearness Drive London, Ontario N6E 1N9	(519) 681-5350
Northwestern Regional Office	808 Robertson Street Box 5160 Kenora, Ontario P9N 3X7	(807) 468-3111
Leslie M. Frost Natural Resources Centre	Dorset, Ontario P0A 1E0	(705) 766-2451
Aviation and Fire Management Centre	P.O. Box 310 55 Church Street Sault Ste. Marie, Ontario P6A 5L8	(705) 942-1800

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