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I.A.H.R. SYMPOSIUM
ice and its action on hydraulic structures

REYKJAVIK, ICELAND

7-10 SEPTEMBER 1970

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Papers with discussions of the I.A.H.R. Ice Symposium in Reykjavik, Iceland, September 8 – 10, 1970.

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WELCOMING ADDRESS

by

His Excellency J. Hafstein
Prime Minister of Iceland

It gives me great pleasure to have the opportunity to address this meeting of distinctive scientists and engineers from the International Association of Hydraulic Research-people from east and west, who have gathered here in Iceland due to their common interest in studying and finding solutions to the various forms of ice problems.

All those living in cold countries are confronted with similar problems of survival, that constantly tax their ingenuity. One of the most important of these problems is how to provide heating for homes and energy for industry and transportation.

Here in Iceland we are blessed with the heat from geothermal sources and this heat we use extensively for the heating of our homes. The provision of electricity and secure transportation, however, presents the same problem here as in all the other colder countries. Ocean drift-ice occasionally blocks some of our harbours, and swelling of rivers and creeks due to ice can create annoying problems around bridges and culverts. Ice may accumulate on the bridge and mast of ships and on the wings of aeroplanes causing serious hazards and accidents. In this and many other ways does ice and ice problems influence our way of living.

The greatest ice problem we have encountered in Iceland is associated with the development of our hydroelectric energy. Ice problems present themselves at all our power plants. The most difficult of these are associated with the Búrfell Project on the Thjórsá River. Our engineers tell me that this river is one of the most efficient ice producing machines in the world. True or not, scientific methods were used to study and analyse the ice problems of the Thjórsá River, for which we have engaged engineers and scientists from various countries to find the best solution.

It gives me great pleasure to welcome all of you to Iceland and I wish this first Ice Symposium of the International Association of Hydraulic Research greatest success. I hope that it will help you come up with more and better ideas to solve the problems common to all of us living in the colder areas of the world.

OPENING ADDRESS

by

B.MICHEL. DR.ENG

President of the Committee on Ice Problems

I.A.H.R.

Your Excellency mr. Hafstein, Prime Minister; Mr. Sneesar of the Department of Industry, Chairman of the Organizing Committee, colleagues of the International Association for Hydraulic Research, Ladies and Gentlemen:

It is an honor and a pleasure to open this first International Symposium on Ice Problems sponsored by the IAHR. We are grateful, indeed, for the splendid opportunity that is given to us to hold this meeting in this wonderful country. Those of us who arrived early for the meeting have already enjoyed some of the wonders of this land of ice and fire and I am sure that everybody will have the pleasure to see them fully.

This is the first Symposium on Ice of the International Association for Hydraulic Research. Our Association was founded before the last war, in 1935, by a group of individual researchers with the objectives to stimulate and promote Hydraulic research, both basic and applied, in all its aspects. General congresses have been held ever since, every two years, and the XIVth Congress is coming in Paris during the fall of 1971.

During the last few years there has been a broadening of interest from the expanded membership and many groups have shown interest in a variety of fields of importance in the general area of hydraulics and fluid mechanics. To look after these particular fields Committees were created within the Association and have started to hold their own Symposia in their subjects.

Our Committee on Ice Problems was the first one created by the Association at the Montreal Congress of 1960 after a very successful seminar on Ice Problems. As a first baby it had a rough going for the first few years but the Committee was active in London in 1963 and afterwards seminars on Ice Problems were held at the General IAHR Congresses of Leningrad in 1965 and Colorado in 1967.

Now, our field of research has taken a new life within the very recent years with the increased development of difficult river sites for hydroelectric exploitation during the winter months. A special session of this Symposium will deal mainly with ice control at power plants. Much experience has been accumulated recently in northern countries on this question and one of the most outstanding power plant operating in the worst river ice conditions, anybody can imagine, is certainly the Burfell Power Station, right here in Iceland.

Another aspect of this Symposium is the beginning of a new era in research on interaction of floating ice with structures and ships. Much interest is presently being devoted to development of resources in the Arctic, where ice problems unheard of before, have to be dealt with in the transportation area. For the first time it is symptomatic to see that ice may not only be a nuisance as is always thought of, but may be a useful tool for development as suggested by some of the papers presented at this Symposium.

At this time I would like to express my thanks and those of the officers of the Directing Committee and members of the IAHR to the participants of the Symposium and to those who have worked so hard on the program and arrangements. Time does not permit thanking each of you induvidually now. Let me note that the names of the Committee members are listed on your programs. In addition to those people listed, I note that several others are helping with registration and some of the other tasks, I express my thanks to all of you. The success of the meeting is in your hands to a large extent.

We are particulary grateful to the chairman of the Organizing Committee, Sigmundur Freysteinsson from Reykjavík. I know that he has put a great deal of personal effort into this meeting and has had many difficult problems that he solved. His chief assistants were Dr. Gunnar Sigurdsson, Mr. Jonas Eliasson and Mr. Pall Theodorsson who helped in the preparation of excellent arrangements.

I now take great pleasure in declaring this First Symposium on Ice of the IAHR, in session.

BERNARD MICHEL

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Programme

Tuesday, September 8

09:00 - 10:00	**OPENING SESSION**
	Opening address B. Michel Welcoming address His Excellency J. Hafstein Prime Minister of Iceland Inauguration lecture S. Rist
10:00 - 12:00	**SESSION 1**
	ICE TERMINOLOGY AND MEASUREMENTS Guest lecture: River and lake ice terminology (H.R. Kivisild) Refreshment interval Presentation of papers Discussion
14:00 - 16:50	**SESSION 2**
	ICE FORMATION AND PROPERTIES Guest lecture: Heat exchanges and frazil formation (T. Carstens) Presentation of papers Discussion Refreshment interval Presentation of papers Discussion
17:30 - 19:00	ICE BREAKER
	Informal reception given by His Excellency Jóhann Hafstein, Prime Minister of Iceland, at Tjarnargata 32, Reykjavík

Wednesday, September 9

09:00 - 12:20

SESSION 3

FORMATION AND BREAKUP

Guest lecture: Ice formation and associated hydrodynamic effects (H.M. Oudshoorn)
Presentation of papers
Discussion
Refreshment interval
Guest lecture: Break up and control of river ice (G.P. Williams)
Presentation of papers
Discussion

14:00 - 17:20

SESSION 4

ICE CONTROL AT POWER PLANTS

Guest lecture: Ice control at the Búrfell Power Plant (G. Sigurdsson)
Presentation of papers
Discussion
Refreshment interval
Presentation of papers
Discussion

Thursday, September 10

09:00 - 12:00	SESSION 5	

09:00 - 12:00 SESSION 5

MECHANICAL PROPERTIES OF ICE AND INTERACTION WITH STRUCTURES

Guest lecture: Thermal ice pressure (S. Lindgren) presented by H.L. Rundgren

Presentation of papers

Discussion

Refreshment interval

Presentation of papers

Discussion

14:00 - 16:00 SESSION 6

FORCES EXERTED BY ICE ON STRUCTURES

Guest lecture: Forces exerted by ice on marine structures by A. Assur

Presentation of papers

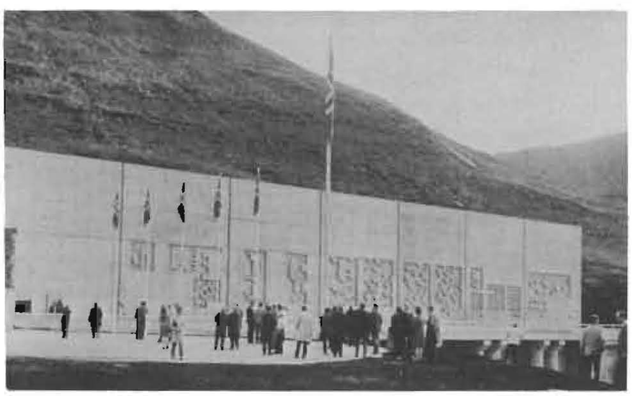
Discussion

Refreshment interval

16:00 - 17:00 CLOSING SESSION

19:30 BANQUET AT ÁTTHAGASALUR, HOTEL SAGA

Guest speaker: Mr. H.L. Rundgren



Friday, September 11

08:45 EXCURSION TO THE BURFELL POWER PLANT
Busses leave from Hótel Saga

LADIES PROGRAMME

Tuesday September 8

1400 hours CITY SIGHTSEEING (3 hours). Departure from Hótel Saga.

The tour passes through the centre of the town with the Parliament Building and the Square Austurvöllur, to the western sector, where a concert-hall, church, schools and an outdoor swimming pool have been built recently.
Visit to the National Museum and Art Gallery. Then the route continues past the University to the eastern part of the town with the statue of Leif Ericson (discoverer of America) and the Einar Jónsson sculpture museum. A stop is made on a hill where the storage tanks supplying the city's central-heating system with natural hot water are situated and from which there is a splendid view of Reykjavík and the surrounding area. The tour proceeds to the Asmundur Sveinsson sculpture garden, the sports stadium, a fish-processing plant and/or other places of interest.

Wednesday, September 9

1400 hours KRYSUVIK HOT SPRINGS (3 hours) Departure from Hótel Saga.
A drive to Alftanes with a brief visit to the residence of the President of Iceland, Bessastaoir, continue the picturesque port of Hafnarfjörour across the lava fields of Reykjanes peninsula to Krýsuvík with its steam jets and boiling sulphur pools.

Thursday, September 10

1930 hours Banquet

PARTICIPANTS AT THE I A H R ICE SYMPOSIUM

REYKJAVIK, ICELAND SEPTEMBER 8 - 10, 1970

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Ernst ZURFLUH Chef-Ingenieur	Elektro-Watt, Ingenieursunternehmung AG, Zürich Switzerland	X

X : Participant in excursion, September 11.

Brief minutes of the business meeting held during
the closing session on 10 September 1970

The chairman of the Ice committee opens the session and mentions that according to the by-laws of the IAHR all participants to the symposium are members of the FAE section and that all are free to intervene, put questions or discuss any subject.

The program of the business section is as follows:

1- Proposal on glossary

2- Bibliography

3- Report of the Nominating Committee

4- Report of the Organizing Committee on the Conference and its financial status

5- Publication of the proceedings

6- Announcement of the Second Ice Symposium

1 <u>Proposal on glossary</u>

Mr. Kivisild refers to his presentation of paper 1.o. and the discussions followed thereafter. All members are requested to study the proposal at ease and to send their remarks directly to Dr. Kivisild before October 1st 1970. The working group will then revise the draft taking into account the remarks received. The revised document will be sent to UNESCO for use within the scope of the "Glossary of Lake and River Ice terms".

Mr. Balanin of Russia will take care of the Russian translation of the glossary.

Mr. Michel will, in consultation with French and Swiss colleages, take care of the French translation.

The Islandic members offer to provide, in cooperation with Mr. Kanavin of Norway and Mr. Williams of Canada, illustrations for the glossary.

The motion of the Chairman to act as outlined above with the glossary is seconded by Mr. Ch.R. Neill and adopted by the meeting.

2 <u>Bibliography</u>

The secretary reports that after announcing the IAHR ice bibliography in the IAHR Journal only a few (less than 10) subscribers have reacted. It is obvious that the interest for a special IAHR bibliography is not existing and that the activities of other agencies fulfill the needs.

The Chairman motions that further activities on the bibliography should be omitted. The motion is seconded by Mr. Oudshoorn and adopted.

3 <u>Report of the Nominating Committee</u>

The nominating committee consisting of Mr. Rundgren, Mr. Assur and Mr. Korzhavin reports through Mr. Rundgren:

The team of the following members of the committee expires:

Mr. Jensen
Mr. Yamaoka
Mr. Balanin
Mr. Palosuo
Mr. Michel

Mr. Balanin., Mr. Michel and Mr. Yamaoka have agreed to be reelectable. As new members the nominating committee proposes Mr. Freysteinsson of Iceland and Mr. Kuuskoski of Finland.

The meeting adopts this proposal unanimously. Consequently the IAHR Committee on Ice problems consists now of:

dr. A. Assur	USA
mr. V.V. Balanin	USSR
dr. T. Carstens	Norway
mr. S. Freysteinsson	Iceland
dr. H.R. Kivisild	Canada
dr. K.N. Korzhavin	USSR
mr. M. Kuuskoski	Finland
dr. B. Michel	Canada
ir. H.M. Oudshoorn	Netherlands (secr.)
dr. I. Yamaoka	Japan

4 Report of the organizing committee

The organizing committee of the 1970 Reykjavik symposium reports through Mr. Freysteinsson.

The number of participants of the symposium was 117, including 48 Islandic colleagues.

14 Ladies were registered for the ladies programma.

5 Publication of the proceedings.

The secretary announces that the IAHR general secretariat will take care of the proceedings of this Reykjavik symposium. All authors will receive at the closing of the session their papers, the questions raised during the discussion and the remarks of the reviewing committee. They are requested to send the revised papers together with the discussions to the IAHR secretariat, P.O.Box 177, Delft before November 1st.

All efforts will be taken to publish within half a year after that the proceedings of the symposium.

6 Announcement of the second ice symposium

The chairman raises the point whether the interval between two symposia should be 2 or 4 years. The general feeling is that an interval of 2 years is most favourable.

Mr. Balanin of the USSR proposes according to a letter of Mr. Skladnev of the USSR committee for the USSR participation in international power conferences to have the second symposium at Leningrad during September 1972.

Mr. Balanins proposal is seconded by the chairman and accepted.

Closing remarks of the Bussiness meeting by the Chairman.

This first symposium on Ice Problems of the IAHR has been a great success from the statistics that were given to us by the Chairman of the Organizing Committee if we consider, furthermore, that this is our first experience in this field of basic and applied studies.

This is due in no small part to the talent and hard work of our Organizing Committee and on behalf of us all I would like to express an unanimous motion of thanks, gratitude and appreciation for all the fine efforts that our Icelandic hosts have gone to in providing this meeting. Let us have a good hand for them!

We have very much appreciated the warm and generous welcome of our hosts and the honor of being welcomed by the Honorable Prime Minister of this country, Mr. Johann Hafstein, to whom we want to express again our thanks.

I also want to express the appreciation of the Directing Committee to the General reporters and the Chairman of each session that made this Symposium possible as well as successful. I would not like to forget also the operators of projecting equipment which did a good job on keeping pace with the heavy schedule of presentations.

We will surely remember forever the souvenir of this wonderful city and its people. We are looking forward to the visit tomorrow of part of the country.

One thing is sure at the end of this meeting. It was the best place, half way between America and Europe on the Arctic Circle, with the contribution of wonderful weather, to hold a first International Symposium on Ice Problems and to all its contributors, our heartfelt thanks.

INAUGURATION LECTURE

Sigurjón Rist	Orkustofnun	Reykjavik
Chief Hydrologist	(The National Energy Authority)	Iceland

Participants and guests.

I hope this symposium will serve scientific progress.

I wish the foreign visitors a pleasant and profitable stay in Iceland. Although the weather is very fine this morning, you may find Iceland as a wide outdoor ice-laboratory:

1) 11% of the country is covered with glaciers.
2) Occasionally polar drift ice closes fishing-banks and harbours off north and east Iceland.
3) Winter ice effects all streams and lakes.

The laboratory itself is not always so cold. Each generation can see the interplay of the dynamic forces of ice and fire. During this act we could just as well change the name to Fireland. The history of the nation is a struggle against ice and fire.

Let us take a look at the winter-ice. If you walk along a riverbank you will find signs of the ice from last winter - scrapes in the soil, stones and heaps of sand. - Farther above the water you may find signs of older ice events.

If we were now in the highland we might see slush moving downwards, as it was clear sky last night, but we must wait until November to see extensive freezing-up. However, it is not certain, we cannot be sure. Ice can come in October and disappear in November, - so changable is the weather in Iceland.

Weather.

It is not only the ice and volcanic-fire which are fighting as mentioned, but warm and cold airmasses are still fighting for the domination of the weather in the country. Iceland is namely situated in the boundary zone between two very different air masses. One of them of polar origin and the other of tropical origin.

The weather is always changing. If you are not satisfied with the weather to-day you may well be so to morrow.

The climate is also influenced by confluence of two different ocean currents, the cold East Greenland Current moving southwest between Iceland and Greenland, and a branch of the warm North-Atlantic Drift reaching southwest Iceland and moving eastwards around the country. The amount of Atlantic water that flows eastward north of Iceland varies from year to year and is likely to reflect climatic trend.

River-ice-prosesses.

To gain clear view of river-ice-prosesses in Iceland we must first of all notice the weather instability, high precipitation and duration of strong wind. Average yearly run-off is 55 liters/sec pr. square kilometer. Storms are frequent especially in winter.

Now we must notice the nakedness of the country, vegetation covers only one quarter of the surface. Woods and shrubs make only one per cent, the rest, three quarters, is glaciers, lakes and deserts, so the country is exposed to great attack when cold spell sets in, with the result of enormous slush production in the rivers.

It is just because of the slush production that instruments have been constructed in Iceland to measure it and for warning system as will be discussed later during this symposium.

The instability of the weather is so big that the ice lay-time can hardly be determined in the coastal regions, but in mountain areas we have typical lay-time.

PROBLEMS.

It is certain that the winter-ice causes more damage at hydro-electric plants than any other man-made structure.

Problems related to river ice and hydro-electric power plants will be discussed at this symposium in articles concerning Burfell. And then we learn the characteristic of the three Icelandic river types: Glacial rivers, direct-run-off rivers and spring-fed rivers. We know a good deal about a river when we know what type it is

Glacial problems.

As I mentioned you may feel Iceland like an outdoor ice-laboratory. As a good ice-laboratory it has ice in stock, we have approximately 5000 cubic kilometers of ice.

The average thickness of Vatnajökull-glacier is 420 meters and maximum thickness is 1040 meters. Its ice mass is 3500 cubic kilometers.

The glacier-tongues in southern Vatnajökull creep by farms, so the people observe easily daily and yearly variations and movements of these glaciers. Therefore it cannot be regarded as anything extraordinary that these people visualised that the excess-snow-accumulation in the highlands was transported to lowlands by the glaciers and indeed this appears clearly in a writing of Rector Th. Vídalín:

> Abhandlung von den islandischen Eisbergen
> in Magazin aus der Naturforschung
> Hamburg and Leipzig 1754.

Inhabitation by glaciers leads to many problems. As mentioned in an article presented at this symposium road is still lacking in a 30 km wide alluvial sand-area south of Vatnajökull where jökulhlaup (glacier-bursts) transport ice blocks from the glacier.

Another glacial phenomenon the catastrophic advance of the Brúarjökull-glacier in the north east part of Vatnajökull, which occurs every 60 years reaches all the way down to a reservoir of a planned hydro-electric plant. I do not point this out to devaluate the project of gigantic hydro-electric-power-plant in eastern Iceland, but to make such project more secure and to encourage solution of all problems in the beginning.

Problems of Polar Drift ice.

Ice problems of inland-ice are in fact negligible compared with the problems of Polar Drift ice. This polar drift ice seriously influences the fate of the nation.

Polar drift ice did scarcely enter Icelandic waters during the period 1915-1965, but in that year this "old enemy" appeared and reached the coast and again in 1967 and 68. Last year a conference was held in Reykjavik about polar drift ice, but only among Icelandic scientists. The results of this conference have been published in a book of 550 pages. This conference did not deal with financial losses and damages caused by drift ice on harbours and other human constructions, nor did it attack the problems of how these damages could be reduced. I sincerely expect that the results of this symposium will direct us to these problems and it is to be hoped that the nation will benefit from us. Scientific experts in the field of agriculture provided indeed valuable advice to farmers when facing the problem of fluor poisoning from the ash of the Hekla eruption in May of this year. Instead of not knowing how to face the problems of ice and fire, the nation has now enjoyed expertise to solve them and so it must in the future.

Problems of winter ice in fjords.

Winter ice in fjords is insignificant, as I have mentioned in an article presented at this symposium. Its formation is most extensive in Breidafjördur. Few years ago such ice was drifted by strong southeast wind onto the coast and seriously damaged the new harbour by Reykir.

WHY ICELAND?

It has been believed that Iceland derived its name from polar drift ice and I conformed with this belief when I wrote the article on River and Coastal Ice Problems presented at this symposium. Upon critical reconsideration of this belief, I find it likely that winter-ice in fjords in Breidafjördur and western Iceland is responsible for the name. Indeed the folklore explaining the country´s name took place by Breidafjördur.

EDUCATION.

Although we Icelanders live in a kind of an ice-laboratory we know very little about the nature of ice. Only in isolated farming areas can we expect to find people who have observed freezing-up and break-up prosesses for years. However, catastrophic events tend to be on top in their mind, they believe that dams and sluices in Iceland are much to weakly constructed. On the contrary most Icelanders do not even know basic process of freezing-up. Indeed the study of icing is hardly mentioned in schools. This is bad for engineers, who need to work on icing after completing their general university courses. This is, however, not only the situation in Iceland as is clearly stated by the International Hydrological Decade, for example at the First International Seminar for Hydrology Professors at the University of Illinois last year (1969), where it was pointed out that students studying hydrology or any Water resources development get curruculum too late in the hydrology.

The University of Iceland has adopted the direction to emphasize subjects which are of importance for the national economy. A symposium such as this one is encouraging for such a direction.

A PHILOSOPHY ON TACKLING ICE PROBLEMS

Banquet speech by L. Rundgren.

Ladies and Gentlemen,

Pressure has been brought to bear on me to say something about ice problems on this solemn occasion. In fact I haven't had any serious problems with ice for many years because since 1963 I have mainly been working in countries under the equator, where the main ice problem has been to find enough ice to cool the drinks.

Nevertheless I have found it a good thing to think a little about a general philosophy on tackling ice problems. As a result of some more thought on the subject I have come to the conclusion that two fundamental laws for dealing with ice can be established.

These two Ice Tackling Laws, I have found, are very similar to those two rules, which have been governing my relations with the organizers of this symposium. The first one is:

"Do not fight against people who ask you to speak on ice problems - try to avoid them".

The second rule to which I had to adhere when the first one did not work is:

"Do not fight against the opportunity to speak on ice problems - make use of it".

Therefore, ladies and gentlemen, there is no longer a pressure on me but rather a great pleasure to me to present to you on this special occasion the two Fundamental Laws on Tackling Ice Problems. I cannot think of any better place than the capital of Iceland nor any better occasion than that of the first separate Ice Symposium for this presentation.

First of all I wish to draw your attention to the fact that the formation of ice is a natural phenomenon which has only been simulated by man during the last two centuries when trying to make ice cream.

Nature is always or almost always stronger than man. Therefore, I would like to recommend any engineer facing ice problems to behave like a field marshal.
First he shall count the numbers of the enemy, then he shall count the numbers of his own men and then he may decide to fight a battle or not. No clever ice field marshal should fight against the ice if there is no chance to him to win the battle.
This is the meaning of "Ice Law No 1", which may be formulated as follows:

"Do not fight against the ice - avoid it".

I would like to give some examples on the application of this basic law. If I were a German engineer I would of course have started by saying "Buy Volkswagen and you won't run into the problems of getting ice in your cooling system". Some Volkswagenowners have told me that this is true. The air-cooled engine in a Volkswagen never gets frozen. Only the passengers do.

Turning to the field of more powerful engines, I wish to say something about the general layout of hydroelectric power plants. A very common layout is that the river is cut off by a dam, from which a longer or shorter headrace channel brings the water to a power station, where the potential of the water is conversed into electrical energy. In cool countries such as Iceland the reservoir upstream of the dam as well as the water surface of the headrace channel will be covered by ice during the winter. This is fair enough and doesn't cause any harm to anybody - except to ice-jumping schoolboys.
The trouble arises first when the reservoir is being used for short term regulation of the river flow. In this case the water level will continuously rise and fall during the day both in the reservoir and in the headrace channel.

In the reservoir the ice cover will normally be intact and follow the rising and falling of the waterlevel without any difficulties. Only some ice cracks along the shore-line of the reservoir will show up. In a narrow waterway like a headrace channel, however, the ice cover very often breaks down into pieces by the water level variations. Only the schoolboys may be happy about this. The ice sheets float with the water down to the ice racks of the power station, where they may completely block the intake. Very often ice chutes have been arranged at the side of the intake to take care of the floating ice but still a lot of trouble is caused by the ice. In addition a lot of power is lost because the ice and water passing through the ice chutes do not produce any power.

What then, is the solution?
Well, the trouble is caused by broken ice sheets. Thus when applying the first ice law we recall:
"Do not fight against ice sheets --- avoid them".

How do we then avoid broken ice sheets? The answer must be: Simply by avoiding headrace channels at power plants where short term regulation is envisaged and where the climate is such that ice problems may arise.

By making a proper layout of man made structures it is possible to avoid another serious problem, i.e. the formation of frazil ice in the rivers. Frazil ice consists of a great number of ice crystals formed in supercooled turbulent water. On some occasions the whole body of water may consist of frazil ice forming a real ice jam which may block any kind of waterway including intakes for water supply and power plants.

The existence of frazil ice in a river will also promote the formation of anchor ice on the river bottom or ice bridges at the surface. These ice dams will have a considerable backwater effect, thus causing flooding of land beside the normal river channel.

The generation of frazil ice is the result of supercooling of the water on river reaches where the velocity is sufficiently high to prevent formation of a continuous ice cover. Once a continuous ice cover is formed the water flows under this cover without direct contact with the air and there is no risk of supercooling the water.

Once again we may apply the first ice law by saying:
"Don't fight against frazil ice --- avoid it".

How then, do we avoid the generation of frazil ice? The answer is: Simply by bringing down the velocity of the water to such a low level that a continuous ice cover will be formed.
This is archieved by complete regulation of the river profile by constructing a sufficient number of dams leaving no rapids left, where supercooling may take place. Another prerequisite is that the cross sectional area of any open waterway is made large enough to keep the velocity low. As a rule of thumb in Sweden we used to say that the velocity should not exceed 0.5 m/sec in order to allow for the formation of a protective ice cover.

These two examples have shown how major ice problems may be avoided simply by arranging a proper layout. On a somewhat smaller scale another method of avoiding ice problems has been proved useful.

You may have been on a long skiing tour on a cool but pleasant day. After some 50 miles or so you may feel a little thirsty. You make a break, remove your rucksack from your back and after some fumbling you find what you are looking for -- the coca cola you have brought for this special occasion and about which you have been thinking for the last half an hour. But - what a surprise! It is completely deep-frozen. Now you face a real ice problem when trying to absorb the coke. The only good thing is that you don't have to worry about the bottle-opener you left at home. There is no bottle any longer, just a heap of glass pieces.
All your trouble is the result of the fact that ypu didn't consider the first ice law when you started on your skiing trip. Simply by putting the bottle into your pocket instead of the rucksack you would have avoided all this trouble and you would have been able to enjoy a blood-warm coke whenever you wanted.

I wouldn't dare to say that you may solve all ice problems by putting things into your pocket. I just want to indicate the possibility of avoiding some ice problems by a continuous adding of heat to a fluid which is subject to heat losses. In order to reduce the heat to be added it is often economically justified to improve the heat insulation of the fluid and thereby reduce the heat losses.

As examples of the method described the following may be mentioned.

1. Slippery sidewalks may be avoided by putting heat slings under the pavement.
2. The blocking of spouts by ice is avoided by putting the spouts inside

instead of outside the house walls.

3. The freezing of waterpipes is avoided by putting an electrically heated wire in the pipe.

4. The blocking of gates at power station intakes and in spillway dams is avoided by heating of gates and gate sealings.

5. Waterways and pools are locally kept free of ice bringing warm bottom water up to the surface by means of air bubbles.

At this symposium we also have heard of the Russian method of pumping warm ground water into the head race channels of power stations in order to prevent ice problems.

The last method has been used extensively in Sweden where the waterways to several cities are kept open for the entire winter by means of air bubbles.

In spite of the fact that the two methods described which we may term "the layout method" and "the layin method" enable us to eliminate several severe ice problems, there are in fact some cases where we cannot avoid the formation of ice. For this particular case I have formulated the Second Ice Law, which summes:
"Do not fight against the ice -- make use of it".

In fact we have already had an example on the application of this law. That was when speaking of avoiding the generation of frazil ice by arranging for the formation of a continuous ice cover. Obviously both the first and the Second Ice Law are applied in this case which requires a special formulation such as: "Avoid frazil ice by making use of solid ice".
An example on pure application of the Second Ice Law was given in the early Forties when the winters in Scandinavia were extremely cold. Also in winters the ice-breakers do not manage to keep the waterways open for the harbours on the northern part of the Baltic Sea but further south there is normally no break in the shipping between Sweden and Finland. During the extremely cold winters in the beginning of the Forties, however, the Baltic was completely blocked by thick ice and no icebreakers were capable of opening any navigation channel between the two countries. As a matter of fact Finland needed vast imports from Sweden at the time and the situation was harassing. What was to be done? Obviously the First Ice Law could not be applied because the ice was already there. Then the Second

Ice Law had to be applied to the effect that:
"If you cannot break the ice -- make use of it"

This was done. One of the longest floating bridges ever used by automobiles was brought into service across the Baltic Sea at a place where the shortest distance between Sweden and Finland is some 80 km. The thickness of the ice was very variable and had to be checked from time to time. Weak parts were strengthened by pooring water on the surface. The ice road had also to be kept free of snow by ploughing. On a smaller scale this method of utilizing the ice cover on lakes and rivers for transportation has a long tradition in Sweden, where the rivers are many but the number of bridges was very limited in earlier days. Nobody would have dreamed fighting against the ice in Swedish rivers, just to make use of it!

Also in these days the ice cover on lakes and rivers is extensively used. Very often it serves as a platform for surveying river sections and for drilling into the bottom as a preparation for underwater blasting and winter fishing from the ice should not be forgotten.

Also in earlier days the ice cover was sometimes utilized for transportation on a larger scale. One for the more well known historical events took place when King Charles X brought the whole Swedish army across the Belts between the Danish islands in 1658. King Charles X was on his return journey home after some fighting on the continent when he came to the shore of the Great Belt with his army. He found the Great Belt covered by ice which completely hampered his plans of "hiring" some ships to bring the army across, He also experienced that the icebreaker service in Denmark was not very well developed at that time so he decided to apply The Second Ice Law i.e not to fight against the ice but to make use of it.

Thus he sent some men to judge the strenght of the ice cover and he probably also consulted some people clever in weather forecasting and afterwards went across with the whole army, a total of some 5 000 men. In spite of the fact that the ice was fairly thin and in places covered by half a metre of water King Charles X was very successful in his task and managed to bring over the major part of his forces to Shaelland.

Thus this exciting adventure in the past came to a happy end and so I think this little speech must do the same before we are getting to far into the future. As a matter of fact ice can be utilized in many other

Ways than for walking and motoring. For instance you may play ice-hockey on it. You can make artificial islands of it as they do in Alaska and you can utilize the ice for temporary stabilization of soil. All these uses come under the second ice law. Thinking of ice-hockey, ice-racing, ice-skating, ice-cobble and ice-cream I feel inclined to introduce a variant of the Second Ice Law wording: "Don't worry about ice -- make fun of it"

I am quite aware of the fact that you cannot always avoid ice problems, nor will you be able to use the ice. In some cases you will have to fight against the ice and we learned yesterday from our Frankenstein how this can be made in a very nice way by using explosives. So, why shouldn't we introduce a third law for the tackling of ice problems wording:

"If you have to fight against the ice -- do it by means of explosives -- and you will have a lot of fun"

Thank you!

LRn/EEt
4.9.1970

ICE SYMPOSIUM 1970
REYKJAVIK

RIVER AND COASTAL ICE PROBLEMS IN ICELAND

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Synopsis

A general outline is given of the problems of ice in relation to human activities and constructions including winter ice; polar drift ice and ice blocks derived from glaciers. The major ice problems rise from ever-changing conditions as a result of sudden and intense weather fluctuations. Formations of sludge jams accompanied by winter storms do threaten inhabitated areas. Run-off river power plants are influenced by operational disturbances especially by chocking by sludge ice. During the next few years there will be a good opportunity to study what operational disturbances will occur at such power plants in the Thjorsa at Burfell where the intake is especially designed to minimize ice disturbances.

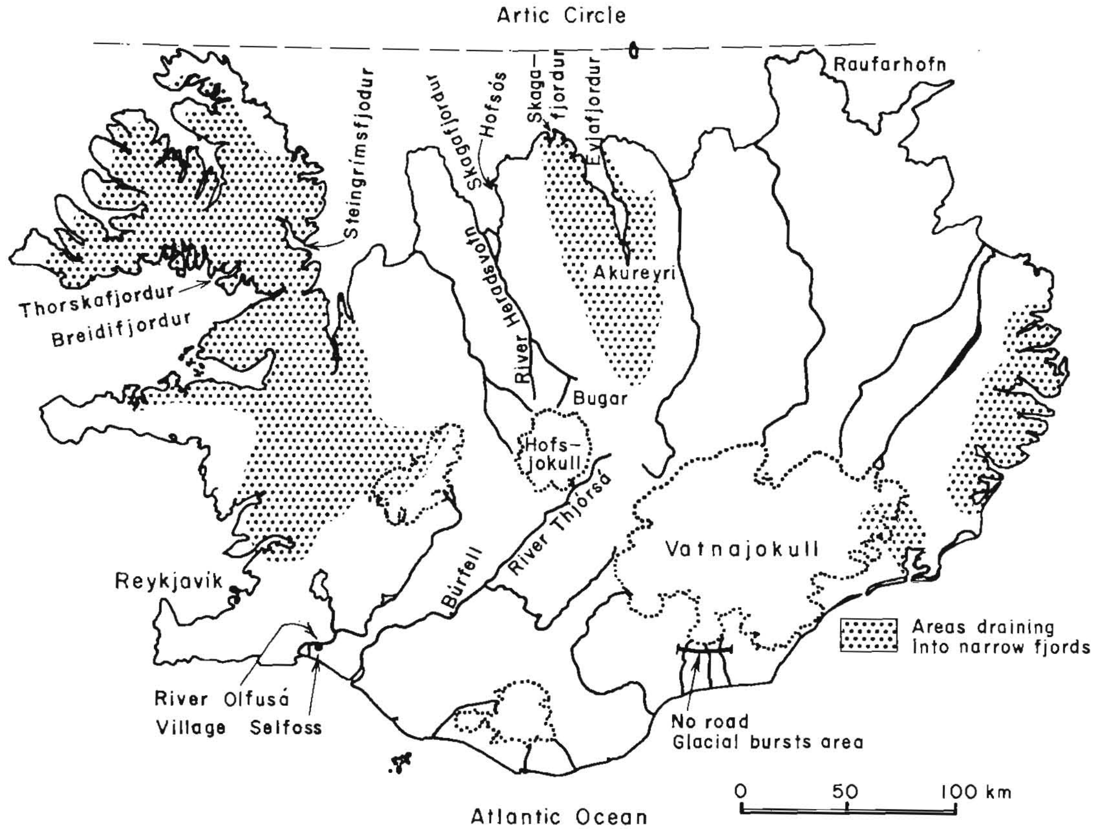
Movement and collision of glacier-derived ice blocks in areas of jökulhlaup (glacial bursts) in connection with road building is an unsolved problem. Geographical engineering ice problems related to construction of reservoirs in the highlands are pointed out.

Introduction.

In Iceland ice may have one of three different origins:

1) Accumulation of ice through precipitation. Glaciers cover 11% or 11300 km^2 of the country. They are very prominent on maps of Iceland.

Jökulhlaups (glacial bursts) bring huge blocks of ice from glaciers onto the braided river courses in southeastern Iceland. Jökulhlaups take place at semi-irregular intervals but they raise serious problems in connection with construction of roads in southern Iceland. Because of these jökulhlaups a continuous road has not been built around Iceland through the coastal regions. Road is still lacking in a strip of 30 km south of Vatnajökull in the main area of jökulhlaups.



Locations mentioned in text Fig. I

0.8

2) Polar drift ice. This type of ice drifts by wind and ocean current from the East Greenland current especially to the north and east coast of Iceland. This happens very occasionally. The drift ice appears in late winter and during the spring. In Iceland it is customary to speak of "ice years" when drift ice remains at the coast and in fjords for a few months and covers fishing banks and blocks harbours on the north and east coasts. The name of the country, Iceland, was derived from this drift ice.

3) Winter ice in rivers, lakes, and fjords. As can be expected the face of winter ice is obvious in a country like Iceland, which is located between 63.5 and 66.5^{o} northern latitude. The Artic Ocean is not far to the north and Greenland is only 300 km away but these parts of the earth are well known for cold climate. However, Iceland is crossed by many lows that move northeast across the Atlantic Ocean in the same direction as the warm Gulf Stream. These lows bring with them warm and moist airmasses that move across the country. The persistance of the winter ice depends therefore on which dominate the warm southern or the cool northern wind. Rapid and intense formation of ice may take place for a few days, but thaw may set in before the stage of ordinary lay-time has been reached, that is breaking-up of the ice may occur unexpectedly and the river swell into heavy flood during the middle of winter. Such happens every winter. If freezing-up, lay-time and breaking-up is termed an "ice-cycle", one can speak of many "ice-cycles" during the winter but these ice cycles are not all equally complete. Winter floods are particularily confined to rivers near the coast. The number of ice cycles are fewer in the highlands and at the same time the cycles are more complete. Usually there is one ice cycle with stable ice cover and an ordinary lay-time of a few months every winter.

Rivers in Iceland have been classified into three types:

1) glacial rivers "J", 2) run-off streams "D", and 3) spring-fed streams "L". The difference in ice formation between these three types of rivers has been discussed elsewhere (1).

ICE PROBLEMS IN RIVERS AND LAKES.

Instability: The major problem of winter ice on rivers and lakes results from its instability or low persistance. As a result of this instability, the same ice disturbances take place repeatedly during the same winter at hydroelectric power plants.

Because of the instability of the ice, travelling across rivers and lakes becomes more difficult than otherwise. However, travelling on ice and the crossing of frozen rivers are presently very insignificant since bridges have now been built across most rivers.

Ice and inhabitated areas: When rivers freeze they often flood their banks. However, the population of Iceland is very scanty and has been adopted to these circumstances so flooding rarely causes damage in inhabitated areas. It is known that occasionally tens of square kilometres of land are covered by water and ice in winter time as a result of ice-jams. Ice jams are known to have formed in some rivers that raised the water level by more than 10 meters, the maximum is 18 metres (2).

There have been no major constructions by rivers and lakes in Iceland until during the last decades.

Recently inhabitation has become more dense by some rivers than it was before such as the village Selfoss, in southern Iceland, located on the banks of Ölfusá river. Damage by flooding can be expected on the average roughly every 20 years. Danger of flooding is most serious when the following sequence of events take place:

1) Heavy ice cover has been formed on the Ölfusá for a few kilometres downstream from Selfoss and this ice has not been weakened by solar radiation.

2) Formation of an ice-jam that fills the riverbed by Selfoss has taken place and ice blocks in the jam have frozen together.

3) A sudden storm with heavy rainfall and melting of snow sets in causing a flood in the Ölfusa and transport of ice blocks downstream from the icy banks of the river.

Under these circumstances observations have shown that the ice cover and the sludge jam will not be removed and that the flow through channels under the ice on the bend by the village become blocked and the water floods the banks.

In Skagafjördur in northern Iceland the river Héradsvötn becomes frequently blocked when the lower stretches of the river are freezing with the result that the lowlands of the valley of Skagafjördur come under water. Such winter ice damming may last for up to two months.

Ice and Roads.

It is uncommon that bridges become damaged by ice transported by streams and rivers. This results partly from the fact that bridges are frequently built across rivers where damage by ice is unlikely, for example where the rivers flow in gorges. This selection of bridge sites, which may make the roads longer is possible because of scanty population and light traffic. In other circumstances for example on gravel fans the bridges are built so they stand high above the riverbed but the road on either side is lower. Therefore when the water level is raised as a result of freezing the water flows over the road and disturbs the traffic for a day or so, but does not damage the bridge or the road.

As mentioned before a road has not been built in the main area of jökulhlaups (glacial bursts) - alluvial sands - in Iceland south of Vatnajökull. During a jökulhlaup the largest ice blocks are not transported far from the edge of the glacier but smaller ice blocks can be carried much farther by the glacial water. These ice blocks which float up by the margin of the glacier and brake from it have a cubic shape and therefore look very different from flat ice sheets that form by freezing of river water. It appears logical to build roads as far from the glaciers as possible because of the transport of these ice blocks. However, it should be realised that the sediment of river beds and alluvial plains become finer downstream. By the glacier it is boulder clay but gradually changes into fine clay. Therefore it is not certain where a bridge would be most favourably located.

Ice and Hydroelectric Power Plants.

Operational Problems. It is certain that the winter ice causes more damage at hydroelectric plants than at any other man-made structure. Experience has shown that operational disturbances are more severe the smaller and shallower the intake pond is in relation to the flow of the river. This is clearly shown in a table in an article in the Journal of the Engineering Society of Iceland from 1959 (3). In fact most hydroelectric power plants suffer from disturances which occur repeatedly during the whole winter because of the ever changing weather conditions. These disturbances are particularily severe in the case of run-of-river power plants.

The disturbances occur first and foremost during the period when the rivers are freezing up and cause the well known blockage of intake channels by sludge ice and also sudden reduction in flow because much of the water in the river channel upstream accumulates above ice-jams. A few power plants are always threatened by surge of pack ice from "step-bursts" (the Icelandic ferm "þrepahlaup").

Disintergration of an ice cover presents rather serious problem if there is a thick ice sheet on the intake pond, because ice floes accumulate in this pond. This happens particularily during winter thaws and is most severe in the case of run-off rivers. In order to avoid this problem it is essential to brake up the ice-sheet on the pond so the ice floes will float away with the spillwater. The breaking up of the ice-sheet must be carried out before ice-jams form in the riverbed upstream to facilitate the flow of ice through the pond. In order to brake up the ice-sheet on the pond it is necessary to be able to change the water level suddenly; particularily to lower it.

The problems related to ice and hydroelectric power plants will not be discussed further but the interested reader is referred to articles at this symposium concerning Burfell power plant.

Geographical Engineering Problems:

In connection with plans of exploitation of hydro-energy inIceland preliminary plans have appeared which involve construction of large reservoirs in the highlands close to the glaciers, such as one reservoir of 240 km^2 south of the glacier Hofsjökull at an elevation of 600 m and another of some 60 km^2 north of the same glacier at an elevation of 750 m in the so-called Bugar. The water from that reservoir would be directed southwards. However, Bugar are located north of the water-divide of central Iceland where Hofsjökull is situated. There are also preliminary plans of constructing reservoirs north of the glacier, Vatnajökull, and to direct the water from these reservoirs to a proposed power plant east of the glacier. In this connection it should be pointed out that the reservoirs such as that in Bugar would be located in an area where the climate is close to that of permafrost. The so-called mounds (Icelandic "rústir") which are found there are the fossil remnants of a tundra climate (4). It is clear that ice problems of geographical engineering will rise in connection with the construction of reservoirs under such climatic conditions. The white ice and snow cover on the reservoirs would reflect solar radiation effectively and it is known how that could influence ablation in spring and early summer. Will it have the effect of reducing thawing and turning the area into one of permafrost? Could it be that ice would accumulate in the area from one year to the next? And if there would be climatic trend to colder conditions, would large changes take place in these areas that would turn them into permafrost and would accumulation of snow and ice take place even if minor changes to colder conditions occurred?

COASTAL ICE PROBLEMS.

Ice near the coast of Iceland may have one of two different origins:
1) Winter ice in the bottom of fjords, which appear every year. It is, however, insignificant. 2) Polar drift ice, which causes very serious difficulties when it covers fishing grounds and reaches the coast.

Winter Ice in Fjords.

Every year at least some ice forms at the bottom of fjords in western, northern, and eastern Iceland. It generally forms at the end of January and may persist until April.

Fishermen on small boats must observe the ice because it may drift. A few harbours may become blocked to these small boats due to ice every now and then for a day or a few days during the winter. However, this ice does not interfere with larger boats. Breidafjordur in western Iceland is famous for fast moving long ice strips, and patches. The ice forms on the smaller fjords penetrating into the country from the main fjord of Breidafjordur. The ice cover persists longest in Thorskafjordur where it may remain for many months. There is often a strong ice foot by the shore. When the ice brakes up in these fjords, which generally happens during the spring tide, sheets of ice drift to Breidafjordur itself. The ice drifts by wind to the islands of Breidafjordur or to the coast, erodes and may damage small harbours and places where ships are drawn ashore. The ice may block sailing routes between islands for a day or so but never for long periods.

Drift ice is generally troublesome but the sheet ice itself may be so also, for example during the "frost winter" of 1918 when all of Siglufjördur became covered with land ice and it also formed between pillars of piers in the harbour. During spring tide the ice lifted the piers and broke them into pieces.

In northwestern Iceland ice ledges form in many places. They are most prominent when the wind is blowing from the sea during spring tide and may disturb fishing and grazing of sheep in the tide zone.

There are four main reasons for the fact that formation of winter ice in fjords is small in Iceland:

1) Warm and salty sea currents warm the coast of the country.
2) Heavy tide currents. The difference between tide and ebb is 1.5 - 4 m.
3) Continuous mixing of sea water by strong ocean waves that move into all fjords at all times.
4) Most rivers do not flow into the sea in fjords but into the open ocean as can be clearly seen on a map of Iceland.

It has not been observed nor do studies indicate that construction of hydroelectric power plants increase formation of shore ice in fjord bottoms with the exception of a proposed plant in Eyjafjördur which would be accompanied by directing rivers into the valley of Eyjafjördur that presently flow into Skagafjördur. The utilization of water in this way would increase shore ice in the harbour of Akureyri which presently may be troublesome.

Polar Drift Ice Problems.

The polar drift ice causes many severe problems when it drifts to the coasts of Iceland. Indeed it is called the country's "old enemy". Basic writings about polar drift ice near the coast of Iceland appeared in the Journal of the Icelandic Glaciological Society last year (5). The interested reader is referred to that Journal. However, damage of harbours by the drift ice are not dealt with in this Journal nor are ways of minimizing damage discussed. This will be briefly discussed here.

The most severe damage occurred in the harbour of Siglufjördur on January 7th, 1968, when the ice drifted by easterly storms into the harbour and destroyed many piers. The damage amounted to tens of millions in Icelandic kronur and reconstructions have not yet been completed. In April 1967, drift ice moved out of Skagafjördur with the result that it broke 20 m off the concrete harbour wall in the seaside village Hofsós. Reconstructions cost 1.5 million Icelandic kronur. On May 27th, 1965, drift ice bruised electric cable in Steingrimsfjördur where it came to the shore on the south side of the fjord. On March 6th, 1969, the same damage occurred again, this time on the north side of Steingrimsfjördur. There are many more examples of minor damages of harbours and landing places caused by polar drift ice. During the latest "ice years" a steel cable has been placed across the entrance of harbours to prevent drift ice from moving into the harbours. This has proved very successful. It was first done in Raufarhöfn in March, 1965.

ACKNOWLEDGEMENTS.

The author is indebted to observators of water level recorders in the various parts of Iceland for good cooperation and important information and advice. The general director of the National Energy Authority, Jakob Gíslason, and members of his staff are acknowledged for their assistance.

Regarding drift ice and ice in fjords special thanks are offered to those laymen from almost every corner of Iceland who have submitted letters and reports including description of these phenomena. It has not been possible to include but a small part of the information in these letters and reports.

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ICE SYMPOSIUM 1970
REYKJAVIK

RIVER AND LAKE ICE TERMINOLOGY

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River and lake ice terms have been evolved gradually in the last 40 years as the processes of ice formation and breakup have become better understood. The IAHR has considered it appropriate to contribute to a systematic river and lake ice nomenclature. The preparation of a terminology was given to a subcommittee headed by the author and a set of terms is presented herewith for discussion and approval. The set is presented in English only, since a multilingual glossary should be based on an agreed set of terms. The terminology has been developed to be in harmony with the work by UNESCO on sea ice, since a submission from us to UN on lake and river ice terms has been requested.

RIVER AND LAKE ICE TERMINOLOGY

Following a proposal by the Committee on Ice Problems, the International Association for Hydraulio Research decided at the Congress in Fort Collins on 13 September 1967 to prepare a descriptive terminology of river and lake ice. I was elected chairman of a sub-committee to carry out this work. Since that time we have consulted various agencies of the United Nations, the members of our committee on ice problems, the National Research Council of Canada and various educational institutions. As a result of these consultations a glossary was prepared reflecting the opinions of the members of our Ice Committee.

The presentation will be made in English and the terms will be arranged within the logic of ice processes so that the necessity of each term and their proper context can be established prior to translation to other languages.

The shown listing of terms by subjects is arbitrary by necessity because of the conflicts of inter-related phenomena in ice formation on lakes and rivers. In our earlier joint review with the National Research Council of Canada, we followed as closely as possible, the glossary of sea ice developed by the United Nations Educational, Scientific and Cultural Organization. In the present presentation the classification has been slightly modified to permit the listing of terms under headings which more definitely describe the development of ice covers and ice accumulations on lakes and rivers. It should be considered, however, only as a working document for the purposes of gathering illustrations and to permit a proper translation into other languages such as the other working languages for the International Association for Hydraulic Research, French and Russian. It might well be advisable to issue our final recommendations in the same way as the submission of the National Research Council of Canada following very closely the sea ice glossary which has now won international recognition.

HISTORY

It may be of interest to review the development on ice terminology in the last four decades.

In 1934 Dr. M.W. Laszloffy described ice formations in rivers, in particular the Danube, and gave definitions of various phenomena and of ice formations. In some aspects this was built on earlier work by Dr. Devik (1930). In its scope and many of the practical aspects, Dr. Laszloffy's work was the first systematic presentation of ice terminology.

Dr. G.D. Ransford and Professor Giroud presented in 1951, in La Houille Blanche, a series of articles establishing and discussing technical terms related to frost, snow and ice.

Further work on this line was done by the author in a series of reports in 1952 and 1954, giving a systematic classification of ice phenomena on rivers. A major part of the study was presented to the IAHR in a paper on Ice Floods in 1959.

Wilson et al.(1954) have given a good account of lake ice classification from which some of the terms have been adopted or modified. A more recent classification for various ice types based on texture has been prepared by Peschanskii (1967).

The U.S. Navy (1966) have included ice terms in their oceanography glossary.

UNESCO included ice terminology in their glossary on hydrology. This was, however, an incomplete effort and many members were disappointed in the description of ice terms. The inadequacy of this glossary was one of the reasons for the IAHR to start the development of an ice terminology. In the meantime, the World Meteorological Organization organized an attempt to improve the ice portion of the UNESCO Hydrological Glossary. In 1968, a glossary of lake and river ice terms was prepared by WMO consultant, Mr. S. Fremling, of the Swedish Meteorological and Hydrological Institute, Stockholm. This was originally compiled in Swedish and subsequently translated into English. It is a very ambitious document, and we consider that the coverage of the whole system presented in this document is impractical at present until a more concise system is adopted first.

Dr. Michel prepared in 1969, a classification based on ice formation processes. This paper was expanded in a joint article by Dr. Michel and Mr. Ramseier, in the report on Classification of River and Lake Ice Based on its Genesis, Structure and Texture.

A guide to field description of lake and river ice was prepared in 1969 by Mr. G. Williams of the National Research Council of Canada.

A set of ice terms for field description has been established by the Meteorological Division of the Department of Transport of Canada.

Messrs. B. Michel, D. Carter, M. Drouin, R. Ramseier and R. Boisvert prepared in 1969, a Glossary of Lake and River Ice Terms with added French terms.

Parallel with the above described efforts on lake and river ice terminology, sea ice was classified and terms were agreed by the Working Group on Sea Ice for Maritime Meteorology of the World Meteorological Organization of the United Nations. This group had some tentative sea ice terms developed for 1951. An internationally agreed "Abridged Sea Ice Nomenclature" was accepted in 1954. A draft nomenclature was presented in 1967 and approved in 1968. The Working Group states that the terms are equally valid for lake and river ice. This is contrary to the statement made by UNESCO in their glossary on hydrology, and we assumed that amendments should be introduced where advisable.

The associate committee of the N.R.C., Canada established a working group to review the ice portion of the UNESCO Hydrological Glossary. This working group included Prof. Michel, Mr. Ramseier and the author from the IAHR subcommittee. The glossary prepared by the N.R.C. working group with an input and submitted by Mr. L.W. Gold to UNESCO provided the basis for the glossary now put before the membership of the IAHR for their consideration.

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TERMS RELATED TO ICE FORMATION, ACCUMULATION AND DECAY

A minimum number of terms has been chosen which would describe the processes of ice formation and breakup without ambiguity.

FRESH WATER ICE

In fresh water bodies there are two quite distinct forms of ice formation. LAKE ICE and RIVER ICE form by a static process on calm lakes and sections of rivers with relatively still water. In the final open water stage there is a supercooling of the water surface and ice crystals start to form and primary ice would cover the surface. Winds and currents may disturb this formation and cause a change to dynamic processes.

DEVELOPMENT FORMS OF ICE

In the static process, nucleation will lead to DENDRITES, ICE NEEDLES and TABULAR ICE and a SKIM ICE is developed on the surface. Main continuing growth will be of COLUMNAR ICE, finally building covers of BLACK ICE. Under disturbed conditions also GRANULAR ICE will form part of the growth.

Under a strong wind and in rivers with a substantial current, ice is formed in ACTIVE ZONE in a dynamic manner. FRAZIL formed on supercooled surface is carried by the current to depth or on the surface and combines into floating masses of FRAZIL SLUSH. The addition of snow can create a similar condition with SNOW SLUSH carried by water.

Water and slush appearing on top of ice covers may freeze, creating FROZEN FRAZIL SLUSH, FROZEN SNOW SLUSH or AUFEIS.

DYNAMIC ICE COVER

Frazil slush can float to the surface to form circular discs of PANCAKE ICE. Slush can also be compacted into SHUGA (Sludge) and by turbulence to SLUSH BALLS which also include other elements of ice. Broken parts of static ice cover forms such as SHALE ICE and ICE FLOES combine with original dynamic forms into BRASH ICE.

MOTION OF ICE

FLOATING ICE and ice forms suspended by current may form ICE RUN downstream. This could include SLUSH ICE RUNS and ANCHOR ICE RUN. ICE TWITCH may cause a part of sheet ice to join the UNCONSOLIDATED ICE COVER in an ice run. Depending on hydraulic conditions the ice cover may be COMPACTING or DIVERGING. Freezing may form an AGGLOMERATE. CRITICAL VELOCITY governs transition of the unconsolidated ice cover to other forms.

STATIC ICE COVER

Static formations could appear in landfast form as BORDER ICE and ICE FOOT. They could form a limited ICE BRIDGE or have an overall coverage as an ICE SHEET. Dynamic ice forms could first be reformed by compaction and freezing, finally covering the surface in a CONSOLIDATED ICE COVER.

ICE ACCUMULATION

Depending on flow conditions at the ICE EDGE upstream of the ice cover, a BIGHT may form or a COMPACTED ICE EDGE may develop. The cover may grow and cause an ICE PROGRESSION upstream. Floating ice or suspended ice may be carried under by turbulent currents on the ice edge and will deposit on the underside of ice covers to form HANGING DAM. Flow or wind conditions may cause onshore ICE SHOVES. These formations are indicated on surface ICE MOUNDS. Frazil may also be carried by river to underwater obstructions and form ANCHOR ICE. Anchor ice, growing from the river bed could block the river substantially in the form of ANCHOR ICE DAMS.

ICE JAMMING

DRIFTING ICE may be driven to shallow water or block a water course as GROUNDED ICE. Such happening may cause ICE JAMS which may also result by choking from floating formations.

DEFORMATION PROCESSES

An ICE PUSH or hydrodynamic conditions may also put ICE UNDER PRESSURE to cause DEFORMED ICE or FRACTURING. Depending on the thickness and rheological properties of ice could be deformed by RAFTING or RIDGING. Depending on ice thickness and elasticity RAFTED ICE may take the characteristic form of FINGER RAFTED ICE. These forms and RIDGED ICE with RIDGES cause the appearance of HUMMOCKS and HUMMOCKED ICE through HUMMOCKING.

FRACTURES are often concentrated in FRAVTURE ZONES, often accompanied by SHEARING motion.

OPENINGS IN ICE

Varying water levels, hydrodynamic forces and temperature variations may cause THERMAL CRACKS, TIDAL CRACKS and SHEAR CRACKS. These CRACKS could be SURFACE CRACKS and DRYCRACKS or further developed into THROUGH CRACKS or HINCE CRACKS. Motion of ioe may create LEADS and hydraulic conditions may keep open a POLYNYA.

MELTING AND BREAKUP

In the BREAKUP PERIOD sheet ice usually disintegrates by internal MELTING, resulting in ROTTEN ICE. Depending on crystal boundaries columnar ice becomes CANDLE ICE and rotten granular ice becomes CORN SNOW ICE. Melting creates PUDDLES which later develop into full THAW HOLES. ICE POT HOLES appear and disintegration causes FLOODED ICE.

A BREAKUP with intensive melting in place, without excessive wind or current results in INSITU BREAKUP. The weakening of ice might also result in DETACHED ICE which will move during ICE CLEARING leaving STRANDED ICE often in the form of ICE LEDGE or massive accumulation in the form of ICE GORGE usually formed by breaking ICE JAMS.

During all described phases, ICE CONCENTRATION varies, until the body of water becomes ICE FREE.

TERMS ARRANGED BY SUBJECT

In the above description of ice formation, motion, accumulation and decay processes, the terms were made to appear more or less in the sequence of ice processes during a season. This is not necessarily the best way to present an encyclopedic system of terms. For this purpose the terms should be arranged under subjects which refer to obvious features and are clearly definable. The same term could then be used in two genetic contexts.

The following presentation is suggested, which has the advantage that it follows the already accepted sea ice nomenclature as much as it is considered feasible.

1. Floating ice

floating ice	river ice
lake ice	sea ice

2. Development of forms of ice

black ice	granular ice grain
columnar ice grain	ice needle
dendrites	ice rind
frazil	new ice
floc	skim ice

3. Forms of static ice

anchor ice	hanging (ice) dam
anchor ice dam	ice bridge
aufeis	ice cover
border ice (shore ice)	ice crossing
brackish ice	ice foot
columnar ice	ice gorge
drained frozen frazil slush	ice jam
drained snow ice	ice sheet
frozen frazil slush	snow ice
granular ice	stranded ice
grounded ice	tabular ice

4. Forms of dynamic ice

agglomerate
bight
brash ice
compacted ice edge
concentration
concentration boundary
consolidated ice cover
diffuse ice edge
frazil slush
ice edge
ice floe
pancake ice
shale ice
shuga (sludge)
slush ball
snow slush

5. Motion of ice

anchor ice run
compacting
diverging
drifting ice
ice jamming
ice progression
ice run
ice shove
ice twitch
shearing
slush ice run

6. Deformation processes

fracturing
hummocking
ice push
ice under pressure
rafting
ridging

7. Openings in ice

channel lead
crack
dry crack
fracture
fracture zone
hinge crack
lead
polynya
shear crack
shore lead
surface crack
thermal crack
through crack
tide crack

8. Surface features

deformed ice
finger rafted ice
glare ice
hummocked ice
hummock
ice pot hole
ice mound
ice wrinkle
new ridge
rafted ice
ridge
ridged ice
ridged ice zone
rough ice
sastrugi
shore depression

9. Stages of melting

breakup
candle ice
corn snow ice
detached ice
flooded ice
ice free
insitu breakup
puddle
rotten ice
thaw holes

10. Terms relating to lake and river ice

active zone	dynamic ice pressure
beginning of breakup (date)	freeze up period
beginning of freeze up (date)	frost smoke
breakup date	ice boom
breakup period	static ice pressure
critical velocity	
duration of ice cover	

The definition of all terms are presented in the attached list of terms in alphabetical order.

The close relationship for the above arrangements to the sea ice glossary is shown by the following comparison of subjects

Lake and river ice	Sea ice
Floating ice	Floating ice
Development forms of ice	Development
Forms of static ice	Forms of fast ice
Forms of dynamic ice	Pack ice
Motion of ice	Pack ice motion processes
Deformation processes	Deformation processes
Openings in ice	Openings in ice
Surface features	Ice surface features
Stages of melting	Stages of melting
Special terms relating to lake and river ice	

It should be noted, however, that there is no complete correspondence of terms. Certain deviations were indicated by traditional usage, and other deviations by different processes in fresh water.

RECOMMENDATIONS

When our Committee on Ice Nomenclature was formed it was thought that there would be no previously established nomenclature. Subsequent investigation showed that many institutions, notably the Office of Hydrology of the United Nations Education, Scientific and Cultural Organization and the Working Group on Sea Ice for Maritime Meteorology of the World Meteorological Organization had done substantial studies on ice nomenclature.

We established contact with these organizations. After a review of their work we decided to work through the United Nations and instead of publishing our own nomenclature, to prepare a submission to the United Nations as a contribution towards the establishment of a representative ice terminology.

In 1968 we wrote to UNESCO with a request to have a chance for an official review and submission on the glossary of river and lake ice terms. On December 18, 1968 we were requested by the Chief, Office of Hydrology, Department of Advancement of Science, Mr. J.A. da Costa, to give our comments on river and lake ice terms.

It is hoped, following today's presentation, that a terminology can be accepted by IAHR which could be submitted to UNESCO expressing our views.

Terms Arranged in Alphabetical Order

Active Zone	-	Stretch of river where frazil ice is forming.
Agglomerate	-	An ice cover or floe formed by the freezing together of various forms of ice.
Anchor Ice	-	Submerged ice attached or anchored to the bottom, irrespective of the nature of its formation.
Anchor Ice Dam	-	An accumulation of anchor ice which acts as a dam and raises the water level.
Anchor Ice Run	-	Ice run of mainly anchor ice. The lumps are often of darkish colour from sand or gravel.
Aufeis	-	Ice formed when brook water or underground water freezes on previously formed ice.
Beginning of Break-up (Date)	-	Rivers - Date of definite breaking or movement of ice due to melting, current or rise of water level. Lakes - Date of visual evidence of initial deterioration along shore-line - appearance of shore leads.
Beginning of Freeze-up (Date)	-	Date on which ice forming stable winter ice cover first observed on the water surface.
Bight	-	An extensive crescent-shaped indentation in the ice edge formed either by wind or current.
Border Ice	-	An ice sheet in the form of a long border attached to the shore.
Black Ice	-	Transparent ice formed in rivers and lakes.
Brackish Ice	-	Ice formed from brackish water.
Brash Ice	-	Accumulations of floating ice made up of fragments not more than 2 m across; the wreckage of other forms of ice.
Break-up	-	Disintegration of ice cover.
Break-up Date	-	The date on which a body of water is first observed to be entirely clear of ice, and remains clear thereafter.
Break-up Period	-	Period of disintegration of an ice cover.
Candle Ice	-	Rotten columnar-grained ice.
Channel Lead	-	Elongated opening in the ice cover caused by a water current.
Columnar Ice	-	Ice consisting of columnar shaped grain. The ordinary black ice is usually columnar-grained.
Columnar Ice Grain	-	Vertical ice column forming columnar ice.
Compacted Ice Edge	-	Close, clear-cut ice edge compacted by wind or current, usually on the windward side of an ice cover.

Compacting	-	Pieces of floating ice are said to be compacting when they are subjected to a converging motion, which increases ice concentration and/or produces stresses which may result in ice deformation.
Concentration	-	The ratio in eighths or tenths of the water surface actually covered by ice to the total area of surface, both ice covered and ice free, at a specific location or over a defined area.
Concentration Boundary	-	A line approximating the transition between two areas of floating ice with distinctly different concentrations.
Consolidated Ice Cover	-	Ice cover formed by the packing and freezing toether of floes, brash ice and other forms of floating ice.
Corn Snow Ice	-	Rotten granular ice.
Crack	-	A separation formed in an ice cover or floe that does not divide it into two or more pieces.
Critical Velocity	-	Velocity above which a stable ice cover will not form on a river except by progression of ice.
Deformed Ice	-	A general term for ice which has been squeezed together and forced upwards in places (and downwards). Subdivisions are rated ice, ridged ice, hummocked ice and other similar deformations.
Dendrites	-	Thin branch-like growth of ice on the water surface.
Detached Ice	-	Floating ice free from shore.
Diffuse Ice Edge	-	Poorly defined ice edge limiting an area of dispersed ice; usually on the leeward side of an area of floating ice.
Diverging	-	Floes in an area subject to diverging or dispersive motion, thus reducing ice concentration and/or relieving stresses in the ice.
Drained Frozen Frazil Slush	-	An accumulation of frazil slush that has partially or completely drained before freezing forming a porous white ice.
Drained Snow Ice	-	Snow ice from which the water has been wholly or partially drained prior to freezing.
Drifting Ice	-	Pieces of floating ice moving under the action of wind and/or currents.
Dry Crack	-	Crack visible at the surface but not going right through the ice cover, and therefore dry.
Duration of Ice Cover	-	The time from freeze-up to break-up of an ice cover.
Dynamic Ice Pressure	-	Pressure due to a moving ice cover or drifting ice. Pressure occurring at movement of first contact termed Ice Impact Pressure.

Finger Rafted Ice	-	Type of rafted ice in which floes thrust "fingers" alternately over and under each other.
Floating Ice	-	Any form of ice floating in water.
Flooded Ice	-	Ice which has been flooded by melt water or river water and is heavily loaded by water and wet snow.
Fracture	-	Any break or rupture formed in an ice cover or floe due to deformation.
Fracture Zone	-	An area which has a great number of fractures.
Fracturing	-	Deformation process whereby ice is permanently deformed, and fracture occurs.
Frazil	-	Fine spicules, plates or discoids of ice suspended in water. In rivers and lakes it is formed in supercooled turbulent waters.
Floc	-	A cluster of frazil particles.
Frazil Slush	-	An agglomerate of loosely packed frazil which floats or accumulates under the ice cover.
Freeze-up Date	-	The date on which the water body was first observed to be completely frozen over.
Freeze-up Period	-	Period of initial formation of an ice cover.
Frost Smoke	-	Fog-like clouds due to contact of cold air with relatively warm water, which can appear over openings in the ice or leeward of the ice edge and may persist while ice is forming.
Frozen Frazil Slush	-	Accumulation of slush that has completely frozen.
Granular Ice	-	Ice made of granular ice grains.
Granular Ice Grain	-	Small ice crystal of irregular form but somewhat rounded like a sand particle.
Grounded Ice	-	Ice which has run aground.
Glare Ice	-	Ice cover with a highly reflective surface.
Hanging (ice) Dam	-	A mass of ice composed mainly of slush or broken ice deposited under an ice cover in a region of low flow velocity.
Hinge Crack	-	Crack caused by significant changes in water level.
Hummocked Ice	-	Ice piled haphazardly one piece over another to form an uneven surface.
Hummock	-	A hillock of broken ice which has been forced upward by pressure.
Hummocking	-	The pressure process by which ice is forced into hummocks.

Ice Boom	-	Floating structure designed to retain ice.
Ice Bridge	-	A continuous ice cover of limited size extending from shore to shore like a bridge.
Ice Clearing	-	Break-up prior to full melting.
Ice Cover	-	A significant expanse of ice of any possible form on the surface of a body of water.
Ice Crossing	-	Man-made ice bridge.
Ice Edge	-	The demarcation at any given time between the open water and ice of any kind, whether static or dynamic. It may be termed compacted or diffuse.
Ice Floe	-	Free floating piece of ice greater than 1 meter in extent.
Ice Foot	-	A narrow fringe of thickened ice attached to the shore unmoved by changes in water level.
Ice-Free	-	No floating ice present.
Ice Gorge	-	The gorge or opening left in a jam after it has broken.
Ice Impact Pressure	-	(see dynamic pressure).
Ice Jam	-	An accumulation of ice at a given location which, in a river, restricts the flow of water.
Ice Jamming	-	The process of accumulation of ice to form an ice jam.
Ice Ledge	-	Narrow fringe of ice that remains along the shores of river after break-up.
Ice Needle	-	A small needle-like ice crystal formed under certain nucleation conditions.
Ice Pot Hole	-	A roundish hole formed in the ice by water motion in a narrow crack or small hole or by the effect of radiation. It may or may not extend through the ice cover.
Ice Progression	-	Upstream progression of ice cover when drifting ice is stopped by a boom, ice barrier or ice jam.
Ice Mound	-	A hump in an ice cover resulting from frazil ice accumulations beneath the cover.
Ice Push	-	Compression of an ice cover particularly at the front of a moving section of ice cover.
Ice Rind	-	Sea ice term, similar to skim ice.
Ice Run	-	Flow of ice in a river. An ice run may be light or heavy, and may consist of frazil, anchor, slush or sheet ice.
Ice Sheet	-	A smooth continuous ice cover.

Ice Shove	-	On-shore ice push caused by wind, and currents, changes in temperature, etc.
Ice Twitch	-	Downstream movement of a small section of an ice cover. Ice twitches occur suddenly and often appear successively.
Ice Under Pressure	-	Ice in which deformation processes are actively occurring.
Ice Wrinkle	-	An unevenness appearing in the surface of an ice cover due to folding by horizontal pressure.
Insitu Break-up	-	Melting in place.
Lake Ice	-	Ice formed on a lake, regardless of observed location.
Lead	-	Long, narrow opening in the ice.
New Ice	-	A general term for recently formed ice which includes frazil ice, slush, shuga (sludge) and other types of ice.
New Ridge	-	Ridge newly formed with sharp peaks; slope of sides usually about 40°.
Pancake Ice	-	Circular flat pieces of ice with a raised rim; the shape and rim are due to repeated collisions.
Polynya	-	Any non-linear shaped opening enclosed by ice. Polynyas may contain brash ice and/or be covered with new ice.
Puddle	-	An accumulation of melt water on ice, mainly due to melting snow but in the more advanced stages also to the melting of ice. Initial stage consists of patches of melted snow.
Rafted Ice	-	Type of deformed ice formed by one piece of ice overriding another.
Rafting	-	Pressure processes whereby one piece of ice overrides another. Most common in new ice.
Ridge	-	A line or wall of broken ice forced up by pressure. May be fresh or weathered.
Ridged Ice	-	Ice piled haphazardly one piece over another in the form of ridges or walls.
Ridged Ice Zone	-	An area in which much ridged ice with similar characteristics has formed.
Ridging	-	The pressure process by which ice is forced into ridges.
River Ice	-	Ice formed on a river, regardless of observed location.
Rotten Ice	-	Ice in an advanced stage of disintegration.
Rough Ice	-	General term for ice covers with rough surfaces.

Sastrugi - Sharp, irregular ridges formed on a snow surface by wind erosion and deposition.

Sea Ice - Any form of ice originating from the freezing of sea water.

Shale Ice - An accumulation of thin broken plates of ice formed when skim ice breaks up.

Shear Crack - Crack formed by movement parallel to the surface of the crack.

Shearing - Motion of an ice cover due to horizontal shear stresses.

Shore Lead - A water opening along the shore.

Shore Depression - Depression in the ice cover along the shore often caused by change in water level.

Shuga (sludge) - An accumulation of spongy ice lumps formed from compressed frazil slush, snow slush, or anchor ice.

Skim Ice - Initial thin layer of ice on a water surface.

Slush Ball - The result of extremely compact accretion of snow, frazil and ice particles. This is produced by either wind and wave action along the shore of lakes or in long stretches of turbulent flow in rivers.

Slush Ice Run - Ice run composed mainly of slush ice.

Snow Ice - Ice that forms when snow slush on an ice cover freezes. It has a while appearance due to presence of air bubbles.

Snow Slush - Snow which is saturated with water on ice surfaces, or as a viscous mass floating in water after a heavy snowfall.

Stranded Ice - Ice that has been floating and has been deposited on the shore by a lowering of the water level.

Static Ice Pressure - Pressure developed by a static ice cover.

Surface Crack - Crack visible at the surface.

Tabular Ice - A particular type of ice whose grains have large horizontal dimensions.

Thaw Holes - Vertical holes in ice formed when surface puddles melt through to the underlying water.

Thermal Crack - Crack caused by contraction of ice due to change in temperature.

Through Crack - Crack extending through the ice cover.

Tide Crack - Crack caused by rise and fall of tides.

Unconsolidated Ice Cover - loose mass of floating ice.

ICE SYMPOSIUM 1970
REYKJAVIK

ICE MONITORING EQUIPMENT

Björn Kristinsson, Dipl.Ing., Managing Director. Rafagnatækni, electronics. Reykjavik, Iceland.

Synopsis.

Description of monitoring methods for ice phenomena in rivers. In som Icelandic rivers ice problems exist in connection with operation of Hydroelectric Power Plants and three types of monitoring equipment were developed, i.e. one for frazil ice discharge, another for ice thickness and a third one for step-bursts. The monitor for frazil ice is described in some detail and a short mention is made of the other two types.

The frazil ice detector and the ice thickness detectors are based on measurements of electrical conductivity changes. The step bursts detector is a pressure sensitive detector.

Résumé.

Description de méthodes de surveillance de phénomènes ayant trait à la glace dans les rivières. Dans certaines rivières islandaises le fonctionnement de centrales hydroélectriques pose des problèmes concernant la glace; pour faire face à la situation trois types d'équipement de surveillance ont été conçus: un pour la décharge du fraisil de glace, un autre pour l'épaisseur de la glace, un troisième enfin pour la rupture avalanche par écelons.

L'équipement de surveillance du fraisil de glace se trouve décrit d'une manière assez détaillée, et une brève mention est faite des deux autres types.

Le détecteur du fraisil de glace et les détecteurs de l'épaisseur de la glace sont basés sur les mesurages des changements de la conductivité électrique. Le détecteur des ruptures avalanches par échelons est un détecteur sensible à la pression.

INTRODUCTION.

At the Thjórsá River in the Southwest part of Iceland the new 105MW Búrfell Hydro Electric Power Plant is located, which, when completed, will be 210MW. In winter one of the points which has to be considered in connection with operation of the Power Plant is ice formation. In order to watch and measure ice movements in the river a special measuring technique was developed which will be further explained in this report. The phenomena which are intended to be observed are frazil ice movements, ice thickness and step-bursts. These will now be explained in a few words.

Frazil ice constitutes the major part of the ice produced in the Thjórsá River and its tributaries. The frazil ice appears as accumulations of loose crystals, about 4mm in diameter, sticking together in flocks. Sludge formed from blowing snow, similar to the before named particles but containing smaller crystals, adds to the ice discharge occasionally. Single ice floes and blocks also occur and besides this sediments and weeds but the last named may disturb measurements temporarily if drifting onto the measuring rod. Frazil ice tends to reduce turbulence at the river surface; it floats on the surface and seldom goes deep down but forms ice clusters with clear water between them. Because of rapids, waterfalls and swift currents, the clusters do not freeze together. The site chosen for measurement of the frazil ice is at Sandafell about 9 km above the Diversion Structure of the Power Plant. At this place the flow velocity is about 2 m/s. By measurement at this place warnings are obtained with 1 to 1 1/2 hours notice so that ample time is given to change position of the gates at the dam. We now turn to what is called here ice thickness.

Ice thickness is in fact only the thickness of the frazil ice which lies close to or moves along the walls of the Diversion Structure and should pass over into the ice channel but not underneath it into the inlet pool of the Plant. Here it is of most importance to measure how deep the frazil ice reaches down and whether it is moving; but by steering of the gates, the flow can be directed. This brings one to the last point which is step-bursts.

Step-bursts is a floodwave of frazil ice and ice floes which builds up by breaking one ice dam or obstruction after another and releasing water and anchor ice which they have accumulated. The main characteristics of a step-bursts is a sudden increase in water level and ice discharge. Instruments for sensing this increase were positioned at a few places up to Sandafell but it is expected that devices can be lost during step-bursts.

For each of the above mentioned points a special method of measuring was developed and we call the instruments Ice Discharge Gauge, Ice Thickness Gauge and Step-Bursts Indicator.

Principles of function. The Ice Discharge Gauge consists of a float sailing on the river with a measuring rod fastened to the bottom of it pointing downwards and cuts the frazil slush as it passes by. The ice is sensed as a change in electrical conductivity.

The Ice Thickness Gauge is a vertical bar inserted in structural part with electrodes evenly spaced along it in order to sens the depth of ice. The ice is sensed in a similar way as before as a change in electrical conductivity.

Step-bursts are usually accompanied by change in water level and this gauge consists of pressure sensitive cells which are placed on the bottom of the river at a few spots upstreams from the Diversion Structure.

ICE DISCHARGE GAUGE.

The ice discharge gauge is made of a few main parts of which particularly to be mentioned is the measuring rod itself, a float together with protecting gear for the rod, mooring cable and heating transformer, electonic part and a recorder.
The measuring rod rod is about 60 cm long and about 12mm in diameter and around the whole of it a double spiral of stainless steel wire 0,6mm in diameter is wound and forms two measuring electodes. The rod itself is a steel pipe with a heating wire within and insulated on the outside by epoxy fiberglass. Spiral grooves are ground in the rod keeping the steel wire in place. The rod is fastened to the bottom side of a float so that it goes down through the frazil ice and cuts it when passing by. The rod is too weak to withstand ice floes and, therefore, the float is equipped to protect the rod against too much load. As already mentioned, ice floes are especially common in step-bursts and usually the frazil ice is loose and not frozen together on the surface.
The float, which carries the measuring rod is shaped like a boat and is equipped with heating ribbons through which a low voltage current is sent. The heating power is about 800W. The float is maintained sailing in the river with a mooring cable which is again attached to a cableway across the river. Icing tends to form on the float during frost and begins where the water splaches against the boat but the ice melts and becomes loose from the heated surface when a certain thickness is obtained, but this depends on the heating inside and the cooling conditions. Hot lines are placed at a few strategic parts on the float in order to make breaking off of the ice easier. The gear which protects the rod against too much weight when ice blocks go under the float and press against the rod allows it to turn backwards when the force passes a certain limit. The rod is steered by a spring which is compressed to the limit that the torque is first raised but then falls considerably. In order to reduce shocks when the rod springs forward again, a shock absorber is also placed at the fastening point of the rod.

Mooring cable on the float includes, besides electrocal conductors, a steel wire which has 700 kp ultimate stress. The electrical wires are for heating themselves, the float and the measuring rod. Then there are measuring leads from the rod. In the bow of the boat is a short pipe which prevents the wires to twist apart when the float is thrown around or rolls.

At the top of the mooring cable at the cableway there is a somewhat weaker breaking wire and it is presumed that the steel wire will break at that place if the stress on the float is too heavy. From this weak spot on the same side as the float, is an accessory wire connected to the land, loosely fastened to the cableway across the river are transformers which change 220V line voltage into low voltage for heating and deicing on the float.

The measuring electodes are fed with A.C. current to reduce the effect of polarization on the measurements. A transformer is in the input ciruit with two primary coils, one for the measuring ciruit and the other for a compensating circuit which makes it possible to have an adjustable but reverse current sent through the transformer to zero compensate the current which passes through the electrodes in iceless water.

Electrodes in water always polarize somewhat and as a result they behave to some extent as a condenser.

To be able to perceive the resistive part of the signal but exclude the condenser effects, the signal from the secondary coil of the compensating transformer is sent through a phase detector and then a signal has been obtained which can be put on a recorder. The zero reading of the signal, on the other hand, did not prove stable because of increased conductivity of the river water during frost, probably because the salts remain in the water but the ice that forms is pure. Besides this, mixing of the water in the river is very limited so that the conductivity is not uniform and, therefore, depending on the respective cross section of the river. Furthermore, meltwater in the spring has also its effects as it contains limited amount of salts. In order to always have a known zero reading two different time constants are used for the records. That is, a very short time constant to find the zero reading which always comes between the ice clusters, and a long time constant for obtaining a fairly good average record. The short time constant is inserted at 2 hours intervals and lasts for about 3 minutes.

Theory for ice discharge gauge. The fundamental principle for the discharge gauge is the fact that, compared with river water, ice is an absolute insulator for electricity.

The change in conductivity which takes place between the two spiral electrodes of the measuring rod is established when ice crystals replace water. This may be so described that the ice thins out the water from the point of view of the conductivity.

Measurements were later based on the hypothesis that relative change in conductivity of river water containing frazil ice is equal to the ratio between ice volume and total volume of the ice water mixture or

$$\eta = \frac{\Delta\sigma}{\sigma} = \frac{\Delta V}{V} \qquad \text{(Density Equation)} \qquad (1.1)$$

$\Delta\sigma$: change in conductivity owing to ice (mho), η : ice density (m^3 ice/m^3 total volume), σ: conductivity of water (mho), ΔV: ice volume (m^3), V: total volume (m^3).

This hypothesis is supported by laboratory experiments (1) and appears sufficiently proven to be of practical use. This linear relationship between conductivity change and amount of ice makes it possible, instead of measuring at many points at different depths and finding the average, to use one pair of electrodes only, which sense the average change in conductivity down to a depth which is more than frazil ice usually reaches.

Other experiments were made which involved measuring the change of conductivity as a function of depth in frazil ice clusters and this proved to give readings (2) which were almost linear so that on a diagram it formed a triangle with the axis. With this in mind, it was not considered necessary to use a long rod. Although on a few occasions the measurements are cut off at the lowest level the discrepancies become insignificant.
These conditions are fulfilled where the water velocity in the river is low and consequently limited suspension of ice in the water. As an extreme example, it can be mentioned that if the rod only reaches half of the full depth of the ice clusters, only 25% of the ice has escaped measurement. If the rod reaches three fourths of the full depth only 6.7% escape measurement. Then it may be mentioned that the size of electrodes and space between them is wide as compared to the size of the ice crystals but influence of size of electrodes and space between them has not been further examined.
The quantity which is in fact measured and recorded is $\eta(w)$, the relative average density of the frazil ice in a surface layer defined by the length of the rod, which is 0 (or 0%) in iceless river water and 1 (or 100%) if the rod has frozen fast. By multiplying the density ratio with the velocity of the clusters on the spot and intregating straight across the river the total ice discharge is found.

$$Q_{ice} = rd \int^{W} \eta(w)v(w)\, dw \qquad \text{(Discharge Equation)} \qquad (1.2)$$

Q_{ice}: ice discharge (ton/sec), d: specific density of ice (ton/m^3), r: rod length (m), $\eta(w)$: ice density (m^3 ice/m^3 total volume), v (w): surface velocity (m/sec), w: distance from river bank (m), W: river width (m).

This calculation is normally not carried out and only the density is observed and as a monitor the gauge gives us a semiquantitative measuring method for the discharge.

ICE THICKNESS GAUGE.

Main parts of this instrument are sensing bars with electrodes, electronic part and a mimic board. The ice sensing bars are U-formed profiles 1.5 m in lenght and with 8 electrodes and the respenctive electrodes can distinguish whether or not ice is positioned against them. These bars are inserted in grooves at a few places in the walls at the Diversion Structure and the purpose is to sense depth and movements of the flow along the walls of the construction. In each groove there are one, two or three instruments according to conditions. The theory behind this instrument is similar as for frazil ice i.e. ice is an insulator as compared with water and the change in conductivity is noted. The conductivity is indicated on a mimic board with light bulbs which form a row corresponding to the electrodes and the brightness depends on the conductivity of the electrodes and, therefore, the quantity of ice in front of them.

STEP-BURSTS INDICATOR.

The instrument is a sensitive indicator for pressure which changes pressure into electric signals. In the electronic part of the instrument this signal is differentiated and then fast changes are noted but not slow changes on the water surface or air pressure.
It is intended to place a few indicatore along the river to note the flood wave of step bursts. By having a number of instruments some will be left although some are lost, when boarder ice breaks up.

READ-OUT DEVICES.

At Sandafell it is possible to observe frazil ice and three indicators are planned there; here also are the required electronic parts for the frazil ice sensors. Each gauge must be calibrated and set at 0% in icefree water and 100% for a fast frozen rod but the equivalent is obtained by disconnecting the rod. Then the information is sent by a telephone cable down to the Diversion Structure.
Diversion Structure. In a watch tower it is possible to make a continuous record of frazil ice at Sandafell. At this place there are a few fast connected mimic light rows for the ice thickness at the inlet construction and selector knob in order to choose one particular mimic row of hte ice thicknees indicators. Then there are warning lights for ice step-bursts.

In the Power House there are planned two panels mainly for ice monitoring. These incorporate meters which show frazil ice at Sandafell and a recorder which records density of frazil ice. A mimic board shows ice thickness at the Diversion Structure and a record can be made of what equals one measuring row at ones descretion and there are meters for ice step-bursts and their warning lights. On the same panels there are also water level recordere for Sandafell and a temperature-recorder for river water at the Diversion Structure.

OPERATIONAL EXPERIENCE.

The frazil ice gauges have been in use during the winter 1969/1970 with recorder in a watch tower at the Diversion Structure and the measuring method proved as hoped for.
The ice thickness instruments were tested at Sandafell during the winter 68/69 and they operated as hoped for.
At the inlet construction the eletronic part had not been started during the 69/70 but by measuring at individual eletroded on a few gauges at the middle of winter, it was possible to note the frazil ice, its movements and depth. During an examination in the spring 1970 damages on construction and indicators were observed where huge ice blocks, up to 100 tons, had hit the structure.

ACKNOWLEDGEMENTS.

Development, laboratory and field tests, of the Frazil Ice Gauge was financed by Orkustofnun, The National Energy Authority, NEA, Reykjavik; and Landsvirkjun, The National Power Company, NPC, Reykjavik.
Other comprehensive experiments with the ice measuring instruments and tests in the field have been financed by the National Power Company and employees of that Institution have assisted in various ways with these tests.
Of individuals who have contributed to the work a special mention should be made of: Dr. Gunnar Sigurdsson, Head of Engineering, NPC, general basic theories, Elías Elíasson, Civil Engineer, NPC, mechanical arrangements for mooring and protecting the float, Jónas Elíasson, Civil Engineer, NEA, matters relating to hydraulics in connection with the floats, Jakob Björnsson, Electrical Engineer, NEA, calculations on conductivity of electrodes, Sigurjón Rist, Chief Hydrologist, NEA, cableway, Sigmundur Freysteinsson, Civil Engineer, Thoroddsen & Partners, Reykjavik, semiempirical calculations of frazil ice discharge for comparision and processing of measured data, Halldór Eyjólfsson, Operator, NPC, operation of instruments and field study of their behavior.

We extend our best thanks to these persons for their cooperation.

REFERENCES.

(1) and (2) Rafagnatækni, Short reports to the State Electricity Authority, Reykjavik, on the feasibility of applying eletrical methods to measure frazil ice, March and September, 1965. In Icelandic.

Fig. 1
Sandafell site showing frazil ice in the river, double cableway, cableway ferry, and a float on the river.

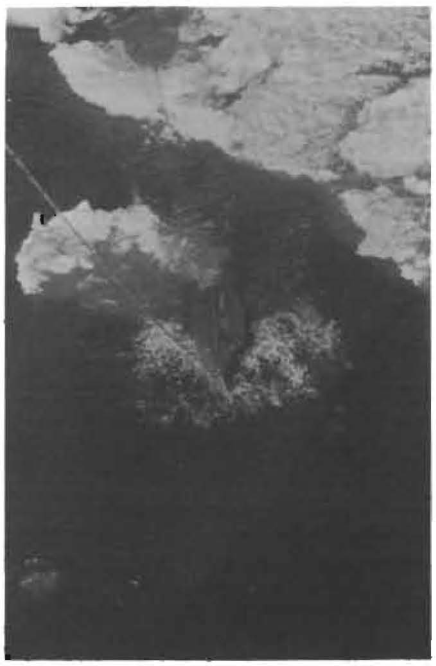


Fig. 2
The float enters free water after a frazil ice cluster has sailed by.

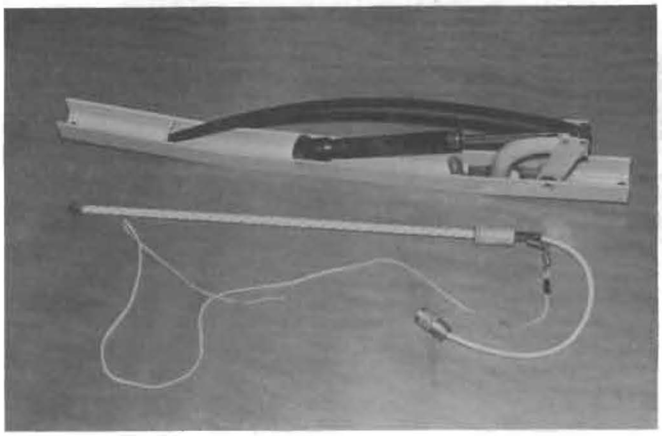


Fig. 3
Internal mechanism of the float and the measuring rod.

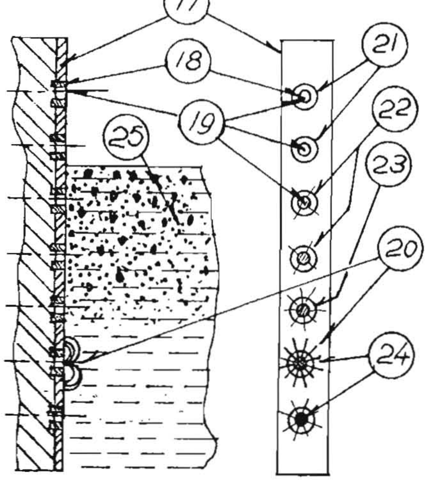


Fig. 4
Schematics of ice thickness indicator, sensor immersed in water and frazil slush and corresponding mimic row with lamps.

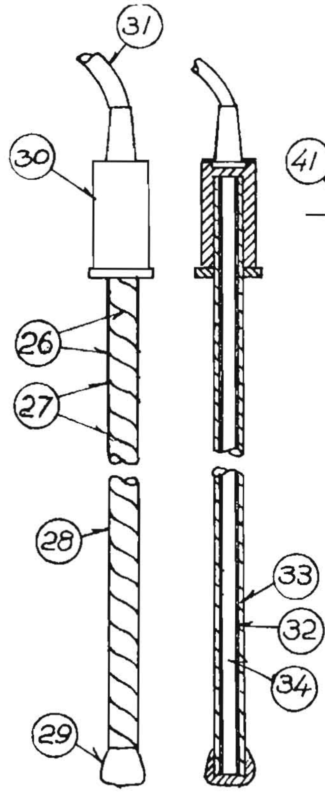


Fig. 5a

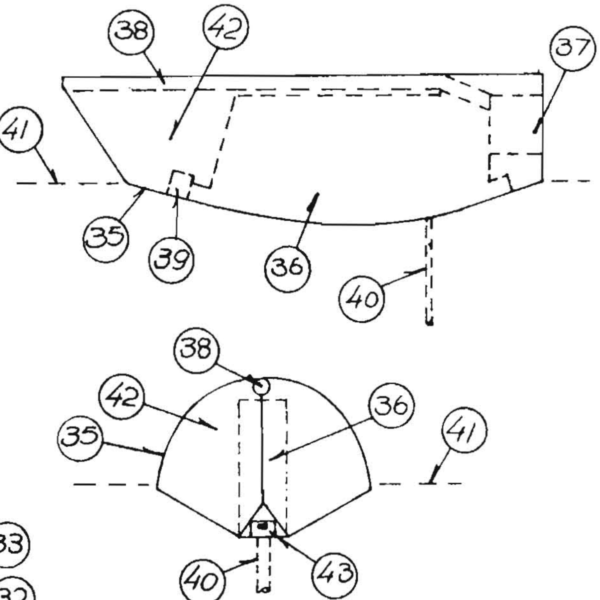


Fig. 5b

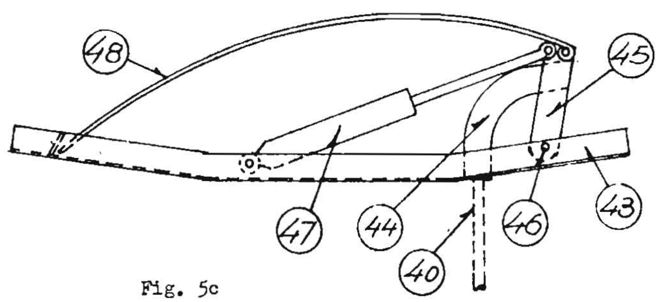


Fig. 5c

Schematics of measuring rod, float and internal mechanism of the float.

Fig. 6

Diversion Structure as seen from an upstream posision before the dam was filled. River water to the intake pond passes under the balcony and ice is skimmed off above it. The watch tower is on the left.

Fig. 7

Ice thickness sensor bars inserted in the wall. Individual electrodes can be distinguished.

Fig. 8

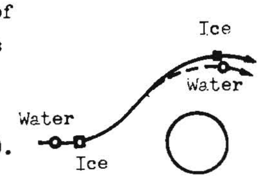
Frazil ice on its way down the ice sluice (balcony) towards the ice canal as seen from the watch tower. The ice enters on upper right of the picture.

Fig. 9

Step-bursts cell and cable.

DISCUSSION BY G.TSANG.

The probe measuring the concentration of frazil ice is a slender cylindrical rod. Due to the curvature of the rod, a centrifugal force will be produced to a particle of a continiuum passing by. For a mixture of water and frazil ice, because the density of water is higher than that of the ice, water particles will deflect less and thus come closer to the surface of the rod (similar to the separation of milk and cream). The result of the above is that the concentration of ice crystals at the neighbourhood of the probe is less than in the undisturbed flow. As this intertial force is proportional to v^2/r where v = velocity, r = radius of curvature, the error of the probe caused by this offset can be quite pronounced for fast flows. The above consideration seems here not been taken into account in the design and examination of the performanc of the probe by Mr. Kristinsson. It is my opinion that an evaluation of this effect should be made.



Authors reply: Calculation of the possible separation during the time of passing by, which is very short indeed, reveales that this effect will be negligible at normal flow velocities in rivers. Closer inspection also reveals that the lines of flow around the rod show a negative curvature in front of the rod and a positive curvature at the sides, resulting in effects, that help to counteract a net separation of ice and water.

More pronounced effect owing to the water velocity, if the rod has access to free surface, is, however, suction of air bubbles down along the rod at velocities higher than say 3m/s especially if it is titled with the upper end against the flow. This results in a velocity component directed down in the suction region at the rear of the rod. The air bubbles will be sensed in the same way as ice. Tilting with the lower end against the flow helps to avoid this phenomenum.
The value of the gauge can, regardless of effects like that mentioned by Mr. Tsang, be further established by direct calibration under different conditions.

Calibration of the gauge is possible by sampling and determination of the exact amount of ice in the ice-water mixture by use of a calorimeter.
We used a sampler made of a sack of soft fabric with a rigid opening of known dimensions. In order not to disturb the flow during the period of sampling the sack is put in the opening of the frame before the instrument is pushed rapidly into the stream behind the measuring rod. The sack fills quickly and the time is taken. This applies for horizontal sampling. Vertical sampling can also be made. After draining most of the water out of the sample the ice is poured into a calorimeter half filled with warm water and allowed to melt.
Using a calorimeter of low heat capacity, polythene covered styrofoam, and

neglecting its effects we get.

Heat lost from water initially kept in calorimeter

$$Q = c_w (T_i - T_f) m \tag{1}$$

Heat gained by the ice-water mixture

$$Q = c_w (T_f - T_o) M + \eta M C_m \tag{2}$$

Equating (1) and (2) we get the mass of ice

$$M_{ice} = \eta M = \frac{c_w m (T_i - T_f) - c_w M (T_f - T_o)}{C_m} \tag{3}$$

Q: heat (kcal), c_m: specific heat of water (kcal/kg^{o}C),
C_m: melting heat of ice (kcal/kg), m: mass of water initially in calorimeter (kg),
M: Mass of ice-water mixture (kg), T_i: initial temperature (^{o}C),
T_f: final temperature (^{o}C), T_o: Temperature of the mixture, (^{o}C), η: ice ratio in sampled mixture (kg ice/kg mixture).

DISCUSSION BY OTHERS.

A mention should be made of a special contribution by Dr. B. Michel suggesting the possibility of extending the use of this frazil measuring technique to sens the onset of frazil formation in rivers at the intake to hydroelectric power plants in order to determine when to make special precaution such as to switch on electrical heating on the intake grill.
Very small changes in conductivity can be detected or less than 1% if the rod does not have access to free surface and suction of air bubbles is prevented.

In a personal discussion with Mr. R.S. Arden it appeared worth while testing the possibility of quantitative measuring of anchor ice by use of the frazil ice detector after mounting it on a special sledge and towing it across a river.

Dr. J.F. Kennedy's suggestion of changing length, diameter and orientation to suit special laboratory condition is an open possibility.

IAHR AIRH
ICE SYMPOSIUM 1970
REYKJAVIK

THE IOWA LOW TEMPERATURE FLOW FACILITY

John F. Kennedy
Director

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SYNOPSIS

The Iowa Institute of Hydraulic Research has recently designed and constructed a low temperature flow facility for conduct of research on various aspects of the formation, characteristics, and melting and breakup of river ice. The facility consists of a 40-foot long, 2-foot wide, 1-foot deep rectangular cross section recirculating flume mounted on a tiltable truss. The flume is housed in a temperature controlled room which can be chilled down to -20°F. The floor and walls of the flume were constructed from specially fabricated heat transfer plates through which a temperature controlled coolant is circulated. A water flow may also be heated, either directly or through the boundary panels. Experience to date with the flow facility is briefly recounted.

RESUME

L'Institut de Recherches Hydrauliques d'Iowa a récemment conçu et réalisé une chambre experimentale á basse température pour la conduite de recherches sur les nombreux aspects de la formation, des caracteristiques, de la fonte et de la rupture de la glace recouvrant les riviéres. Le dispositif experimental consiste en un canal á recirculation, de 40 pieds de long, de section rectangulaire 2 pieds en largeur X 1 pied en profondeur et monté sur une structure inclinable. Le canal est installé dans une piéce dont la température est controllée et peut être descendue jusqu'á -20°F. Le fond et les parois laterales du canal ont été réalisés avec des plaques conductrices de la chaleur specialement conçues a l'interieur desquelles circule un fluide réfrigérant dont la température est controllée. Un courant d'eau peut aussi être chauffé, soit directement, soit á travers les parois du canal. Les expériences effectuées jusqu'á ce jour sont brievement décrites.

INTRODUCTORY REMARKS

The idea for the Iowa Low Temperature Flow Facility traces its origin, indirectly at least, to a short paper published by Carey (1) in 1966 on the configuration of the underside of river ice. The several similarities he reported between the occurrence and behavior of "ice ripples" and of sediment ripples and dunes greatly intrigued the writer, who had for some years been active in research on the mechanics of sediment bed forms, and he undertook to pursue his newly found interest by reading more about river ice. It soon became evident that very few experiments on ice processes in free surface flows had been conducted under controlled, laboratory conditions, apparently because of the paucity of facilities that had been constructed for such research. This deficiency prompted consideration of design and construction of a small, simple apparatus for conduct of experiments on the stability of the interface between turbulent flows and ice, and on the related problem of ice ripples. As design of the proposed apparatus progressed and those who had become involved in the project became more familiar with the state of knowledge about flows past ice boundaries, the need and potential for research on other aspects of river ice became ever more apparent. Accordingly, the equipment being planned became progressively more sophisticated and versatile. The low temperature flow facility that finally emerged from the planning and design effort initiated by ideas springing from Carey's (1) paper is described in the succeeding sections, and the experience gained to date in operating the facility is briefly summarized.

DESCRIPTION OF THE FACILITY

The low temperature flow facility is depicted schematically in figure 1, which shows the flume, pumping systems and related piping, refrigeration and heating systems, and cold room. Additional details of the facility are shown in figures 2 and 3.

The working section of the facility consists of a rectangular cross section flume, 40 ft. long, 2 ft. wide, and 1 ft. deep. The flume is mounted on a tilting truss supported on a pivot near the downstream end and on a motorized jack near the upstream end; the slope can be varied from zero to 2.2 percent. Flow enters the flume through a vaned inlet section and moves along the working section and into a stationary outlet sump which is attached to the tilting flume by means of a rubber connection. From the sump the flow passes to the intake of an eight-inch axial flow pump (specific speed: 12,400) which is driven by a variable speed (continuous from 352 rpm to 1760 rpm) five horsepower motor, and then through the eight-inch diameter return line, calibrated Venturi meter, flexible hose connecting the return line to the inlet section, and thence through the inlet and back into the flume. The maximum discharge attainable is 3.1 cfs. The discharge can be reduced to any desired value by means of the variable speed motor and removable baffles in the sump outlet. The speed of the pump motor is remotely controlled

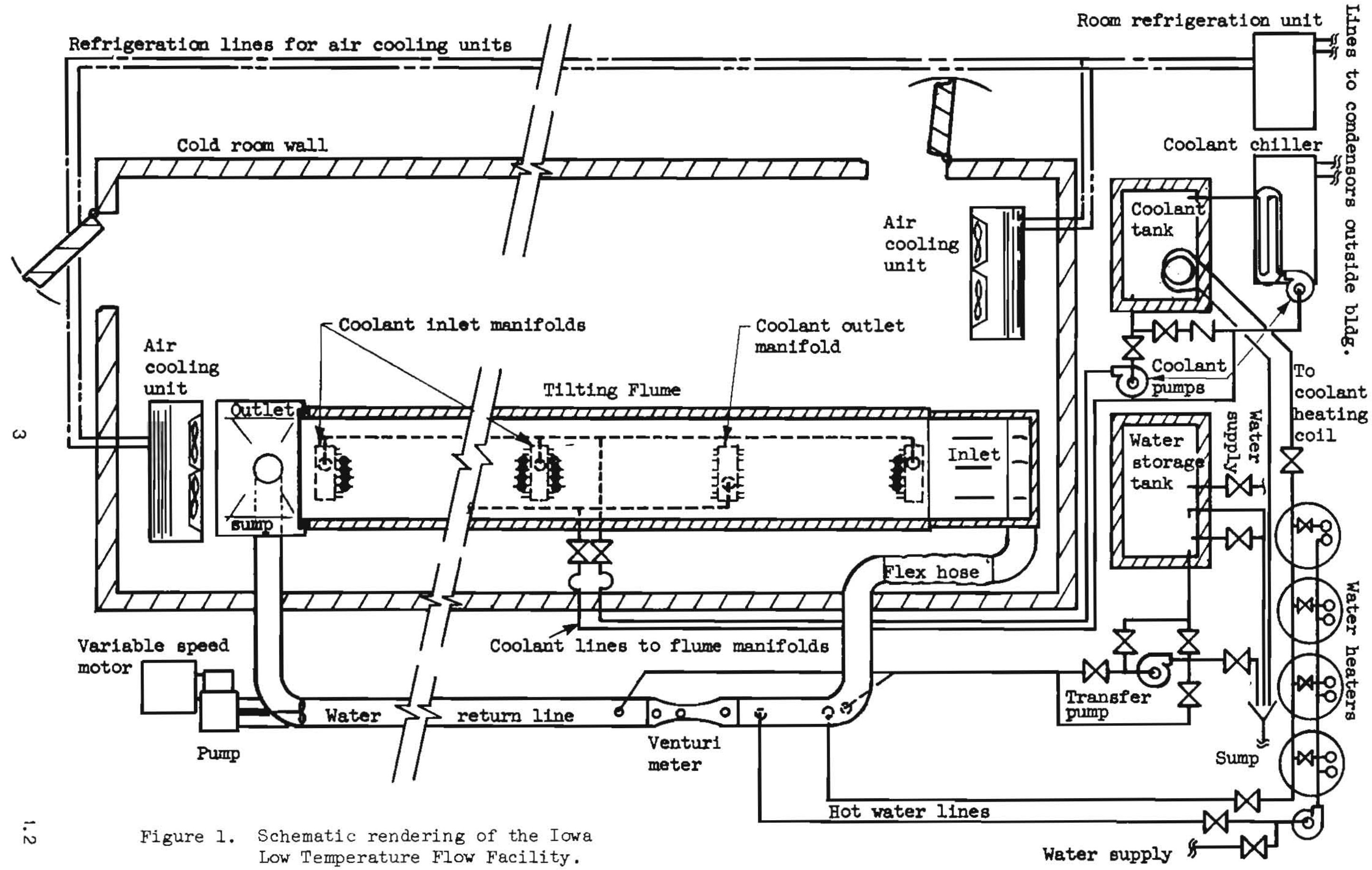

Figure 1. Schematic rendering of the Iowa Low Temperature Flow Facility.

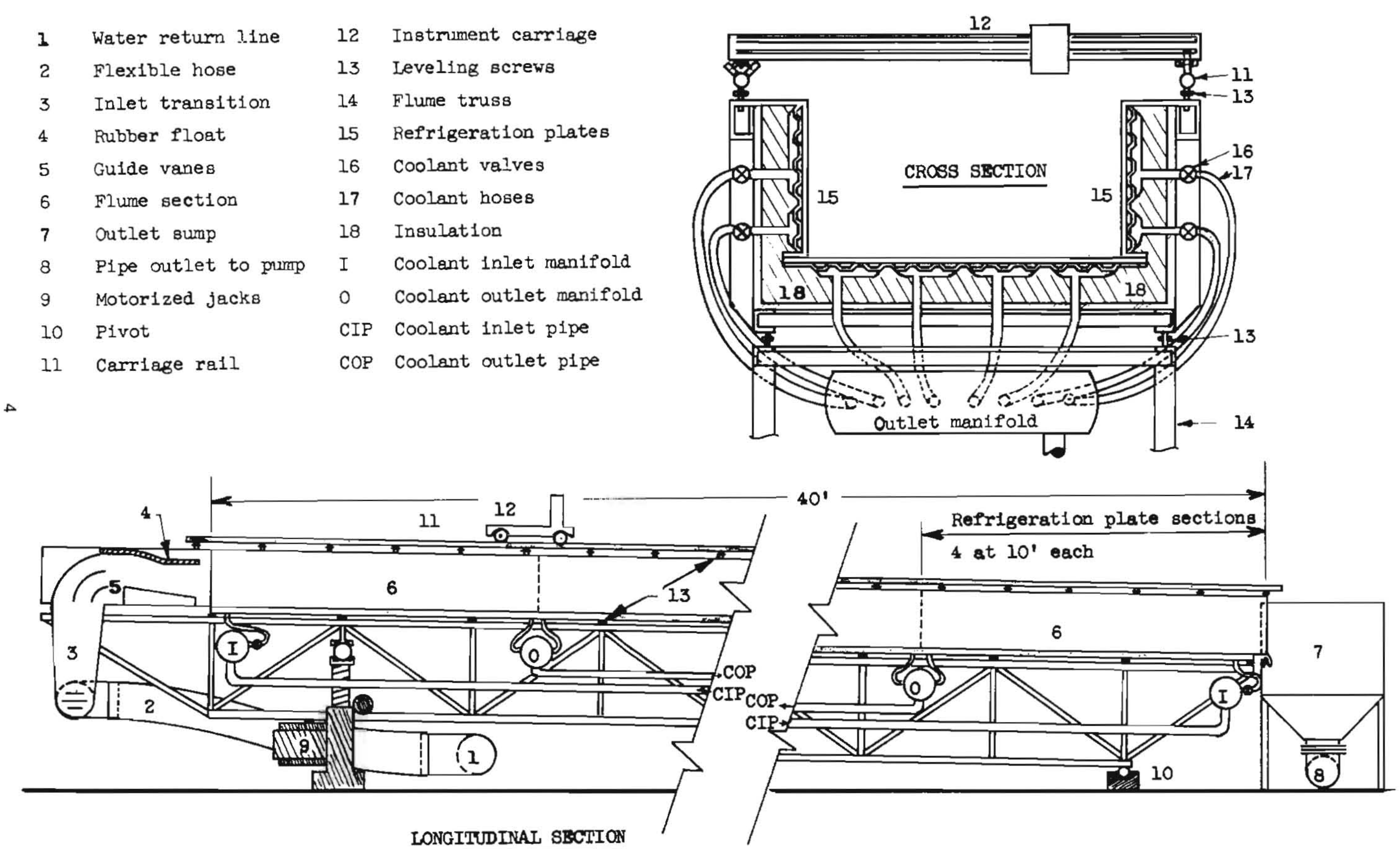

Figure 2. Sectional views of the flume.

Figure 3. Photograph of the flume and insulated room, viewed from the downstream end of the flume. Note surface ice cover on the flow.

from either inside or outside the cold room. The Venturi meter is equipped with three rings of piezometer taps, which are used in various combinations to permit use of the meter with a six-foot air-water manometer at the larger discharges while still producing adequate resolution at smaller discharges.

The flume walls and floor were constructed from specially fabricated steel heat transfer plates (see cross section in figure 2) obtained from Dean Products, Inc. (Brooklyn, New York). The wall and floor sections were received in separate, ten-foot long panels, which were welded together to form a monolithic trough. The dimensional tolerance of the plates as received from the manufacturer was so great that considerable effort had to be expended in straightening and aligning them. In addition, the surfaces of the panels were so rough, as a consequence of the welding process used in their fabrication, that extensive filling with auto-body lead and grinding were required to achieve a hydraulically smooth surface in the flume. Both the large dimensional tolerances and the surface roughness were apparently unavoidable consequences of the manufacturing techniques used, and pose no problems for the normal applications of these plates. After considerable effort, wholly acceptable dimensional tolerance, alignment, and boundary smoothness were achieved. In retrospect, however, it would have been better to have had the plates made in the Institute shops, where better quality control could have been maintained from the outset.

Each ten-foot long wall and floor panel is connected separately to the coolant inlet and outlet manifolds. The six coolant passages in each wall cross section consist of a pair of parallel passages which makes three traverses of the panel length. Each wall panel is connected to its intake and outlet manifolds by two inlet and two outlet hoses; these are valved such that either the upper, middle, or lower pair of passages, or any combination of these, may be turned off. The twelve coolant passages in each ten-foot long floor panel are connected in parallel. These are connected to the same manifolds that supply coolant to the wall panels, and are valved as shown in figure 2.

The outside of the plates are insulated with two inches of polyurethane insulation board, which is covered with 3/4-inch thick plywood. One-inch diameter rails mounted on the flume walls support the motorized instrument carriage. Both the flume and the rails are supported on leveling bolts, so that the vertical alignment of each can be adjusted.

The inside, working surface of the flume is painted with Z.R.C., a 95 percent zinc coating manufactured by the Sealube Company (Quincy, Mass.). An ordinary paint or an epoxy surfacing was not used because of its adverse effect on the heat transfer rate between the flume boundaries and the flow. Hot dip galvanizing was rejected because of the warping it produces. The only difficulty encountered to date with this coating is that it has been pulled away by ice from some small spots over lead-filled areas. However, the underlying lead is also non-corroding and is similar in appearance and texture to the zinc coating, and hence this minor peeling has posed no problem.

The coolant circulated through the flume boundaries is a 50 percent solution of WINTER-FLO, an ethylene glycol base, rust inhibiting anti-freeze manufactured by Union Carbide Corporation (Consumer Products Division, Tarrytown, New York). The coolant is distributed to the flume panels by means of three inlet manifolds, one near the center and one near each end of the flume. After passing through the panels the coolant is collected through the two outlet manifolds, one located at ten feet from each end of the flume. The piping connecting the wall and floor panels, manifolds, coolant tank, and coolant chiller is shown in figures 1 and 2.

The coolant chiller is a ten horsepower packaged liquid cooling unit manufactured by Dunham-Bush, Inc. (West Hartford, Conn.). The outlet temperature is thermostatically controlled and can be maintained to within about 0.5°F at any desired level from 10°F to 30°F; the corresponding heat transfer capacities of the cooler are 52,000 BTU/hr to 96,000 BTU/hr, respectively. The liquid cooling unit is equipped with a centrifugal pump which circulates the coolant through the chiller and the 500 gal. coolant storage tank. A second centrifugal pump circulates the coolant through the flume heat transfer panels. The pumps may be operated independently. The design discharge for coolant was 60 gpm, with either both pumps running and coolant circulating through the plates, chiller, and tank, or with just the chiller unit pump circulating coolant through the chiller and tank.

The four gas-fired water heaters shown in figure 1 have a combined heat transfer capacity of 300,000 BTU/hr. Their function is to heat either the flume water or the coolant, to accelerate the melting or ablation during experiments concerned with these aspects of ice behavior.

An insulated tank located outside the cold room (see figure 1) provides storage for chilled water. This tank makes it possible to drain the flume, in order to measure ice accumulation or configuration, or when the flow is stopped overnight, and later refill it with water whose temperature is only slightly above the freezing point.

The insulated room in which the flume is installed is 54 ft. long, 12 ft. wide, and 8.5 ft. high. The walls and roof of the room are insulated with eight inches of polystyrene insulation board, and the floor with four inches of polyurethane board covered with a two inch concrete wearing surface. The refrigeration system for the room consists of a 7.5 horsepower compressor (Copland, Inc., Sydney, Ohio), two 12,500 BTU/hr (at a temperature differential of 10°F) dual fan evaporators (Krack Corp., Chicago, Ill.), and a condensor (Larkin, Inc., Atlanta, Ga.). Room temperature is thermostatically controlled; temperature variations do not exceed ±0.5°F from the thermostat setting. The two-stage thermostat activates either one or both evaporator units, depending on the difference between the room temperature and thermostat setting. The fans of one evaporator unit operate continuously (except during defrost) to maintain uniform temperature throughout the room. The design minimum temperature for the room was -20°F, and during a test run with no water present in the flume a temperature of -22°F was attained. The minimum temperature obtained for a given heat transfer rate is, of course, heavily influenced by the amount of water present in the flume, its temperature and state of freezing, and the rate of evaporation from the flume water and consequent frosting of the evaporator coils. Room temperatures down to about -10°F can be obtained under all operating conditions. Because of the high humidity in the room the evaporator coils rapidly become ice covered. A defrost period (during which the coils are electrically heated) of at least twenty minutes every six hours is required to keep the coils acceptably ice free.

The water return line, coolant lines, refrigerant lines, drain-fill lines, etc., are all insulated as required to prevent condensation and frosting.

OPERATING EXPERIENCE WITH THE FACILITY

The Low Temperature Flow Facility was placed in operation in mid-February 1970. During the following several weeks a series of tests was conducted to verify the performance of the various components and to evaluate the overall performance of the facility. The individual systems all met or exceeded the design criteria, and were judged fully satisfactory. The integrated system has, with one or two minor exceptions, been found to function equally as well. With the system filled with water, the water temperature can be reduced from about 60°F to the freezing point in about eight hours with coolant circulating at 32°F and a room temperature of about 0°F. No measurable temperature gradients have been found in the room. The coolant undergoes a temperature rise of 0° to 0.5°F, depending on the state of boundary icing and the coolant-water temperature difference, in passing through the heat transfer panels, and the water temperature difference between the two ends of the flume has never been found to exceed 0.02°F. No problems have arisen from icing of the pump, Venturi meter, etc.

When the flume water is being circulated and cooled, by either the coolant or lowered room temperature, or both, the water temperature decreases at an ever diminishing rate until a small amount of supercooling occurs. In general supercooling does not exceed 0.05°F. A suspension of frazil ice then forms and circulates through the system, and the water temperature rapidly recovers to the ice point. The frazil ice significantly diminishes the discharge. The frazil ice concentration decreases as frazil is replaced by surface ice, which forms outward from the flume walls, or by boundary-fast ice, or by both, depending on how the water is being cooled. During the early stages of surface ice formation it accumulates slightly faster just downstream from the inlet sections, a consequence no doubt of the increased heat transfer rate across the surface accompanying the higher turbulence intensity of the flow as it emerges from the return pipe and inlet section. The ice thickness also tends to be slightly greater near the flume walls, because of the heat transfer from the water through the metal plates to the air. As the surface ice cover thickens, it tends to become quite uniform along the length of the flume, and the increased thickness near the flume walls extends no more than one or two inches into the flow.

When boundary-fast ice is being formed, by circulating low temperature coolant through the wall and floor panels, the ice forms at a uniform rate along the flume except for reaches extending two or three inches upstream and downstream from the joints between adjacent ten-foot long heat transfer panels. The coolant passages do not extend over these joints, and hence the heat transfer rate between the water and the coolant is locally diminished. This local suppression of boundary ice formation is greatest during the initial stages of formation, and decreases as the ice thickens. For example, when the ice thickness is approximately 0.5 inches at a point some distance away from a joint, the ice accumulation over the joint may be only 0.3 inches. But when the general ice thickness has increased to 3 inches, the thickness over the boundary will be 2.8 inches or more. When surface ice is being formed by means of reduced air temperature, without coolant circulating through the pipe, a small amount of boundary-fast ice may form in the vicinity of the joints because of the heat leak between the air and the flume boundaries through the coolant pipes (see cross section in figure 2). The amount of boundary ice so formed is generally inconsequential, and has posed no problem in the experiments conducted thus far. This effect can be substantially reduced by placing insulated covers over the valves and ends of the pipe nipples attached to the boundary panels.

A slight hydraulic difficulty arose from minor lateral surging in the downstream sump. The sump is ten inches wider than the flume; this provision was made to accommodate the rubber coupling which attaches the stationary sump to the tiltable flume. The rather large, vertical axis, separation eddies generated at the abrupt expansion where the flow passes from the flume into the sump tended to be unstable, and to concentrate first on one side and then the other of the sump. This surging produced a small, periodic fluctuation in water surface elevation which propagated upstream. The objectionable surging has been eliminated by means of two partitions, one aligned with each flume wall, placed in the sump. The partitions are attached to the sump and extend approximately two inches below the level of the flume floor at the downstream end. This arrangement permits the slope of the flume to be changed even when the surface is covered with ice (the ice in the vicinity of the rubber connection being relatively thin and easily broken by the motion of the flume relative to the sump), and also provides a smooth flow transition from the flume into the sump.

Several types of experiments have been conducted to date. Briefly, these are as follows:

1. The formation of ice ripples. A layer of ice has been frozen on the flume floor, and a turbulent flow then passed over it to investigate the conditions leading to instability of the ice-water interface.

2. The stability of ice jams. A layer of surface ice was formed over still water. The surface ice was then broken into pieces of relatively uniform size but random shape. The critical velocity at which the cover of broken ice becomes unstable has been determined for different flow depths, ice thicknesses, etc.

3. The re-formation of ice after passage of an ice breaker. A layer of surface ice is formed on still water. A uniform channel four to six inches wide is then cut in the ice with a saber saw, and the ice cut from the channel is either removed or fragmented and allowed to remain in place. The rate and characteristics of re-formation of the surface ice, and the rate of thickening of the adjacent ice are measured.

4. Characteristics of ice during formation. Detailed observations have been made on the patterns and other characteristics of surface ice as it forms outward from the flume boundaries.

5. Development of analytical frameworks. The first three of the foregoing research areas are being accompanied by efforts to develop analytical models to explain the observed phenomena.

CONCLUSIONS

The Iowa Low Temperature Flow Facility, designed and constructed over the past two years and recently put into operation, has proven to be a useful and versatile apparatus for conduct of research on a variety of problems related to river ice. The only difficulties encountered to date with the unit are the slight nonuniformity of boundary-fast ice formation over the joints between the heat transfer panels from which the flume boundaries were constructed, and minor peeling of the zinc-base coating from some of the lead filled areas of the flume.

If the flume were being constructed again, the heat transfer plates would be fabricated from stainless-clad steel, and the coolant passages would be made continuous over the whole length of the flume, alternate passages carrying flow in opposite directions to minimize temperature nonuniformity along the length of the flume. It is believed that these modifications would overcome the only difficulties that have arisen.

ACKNOWLEDGMENTS

The cost of the Iowa Low Temperature Flow Facility was slightly in excess of $60,000. Funds for its construction were provided by the National Science Foundation under Grant GK-2873, the Graduate College of The University of Iowa, and the Institute of Hydraulic Research.

In the design of the facility the author was ably assisted by Messrs. George Ashton, Dale Harris, Sheng-Tien Hsu, An-Ching Lin, Jung-Tai Lin, and David W. McDougall, all of the Institute staff. Design and installation of the refrigeration systems was accomplished under the direction of Mr. Lloyd Kohl, of the University Physical Plant Office. Mr. H. J. Hannigan, Technical Manager for antifreeze products, Union Carbide Corporation, provided invaluable information in the selection of a coolant. In addition, Union Carbide donated 330 gallons of WINTER-FLO Antifreeze for use in the facility.

The facility was constructed by the Institute's shop staff, under the supervision of Mr. Dale Harris. The experiments briefly described were conducted by Mr. George D. Ashton.

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1. Carey, K. L., "Observed Configuration and Computed Roughness of the Underside of River Ice, St. Croix River, Wisconsin," U.S. Geological Survey Professional Paper 550-B, 1966.

Discussion of

THE IOWA LOW TEMPERATURE FLOW FACILITY

Q. (T. Carstens, Norway). The Iowa Ice Flume is a very sophisticated piece of equipment, and a major step forward in experimental lab ice research. I am speculating on the future of experimental ice facilities. Will it duplicate the trend in open channel flow, especially with wave problems, towards wider and larger flow areas?

A. (Kennedy). Future facilities of this type will, no doubt, be increasingly larger. I suppose this reflects human nature; each investigator wants his facility to be in some way better, or at least larger, than those that went before it. In the present case the size was selected in a very scientific manner: we built the largest facility we could with the available funds! This was really not consistent with my general dislike of large experimental facilities; I greatly prefer cleverly conceived, small, "table-top" facilities. However, in the present case we foresaw many experiments, including work on ice breaking, formation of surface ice at moderately high Froude and Reynolds numbers, etc., which necessitate a large facility.

Q. (B. Michel, Canada). Could you describe more precisely the hydraulic characteristics of the flume and the tests you are doing now on formation of ice covers?

A. (Kennedy). The hydraulic characteristics of the facility are described rather completely in the printed paper. As noted there, it has a maximum discharge capacity of 3.1 cfs and slope variable from zero to 2.2 percent. The tests presently underway are also enumerated in the printed paper. In addition, we plan soon to undertake tests on the forces exerted on a cylinder moving through a continuous ice cover.

Q. (P. Tryde, Denmark). Have you used the IBM 1800 Computer both as a data logging system and process controller?

A. (Kennedy). To date we have used the computer system primarily as a data logging and/or analysis system. We are just at this moment installing process interrupt features and a digital-to-analog converter which will enable us to use the facility as a process controller, which will feed back information to experiments and alter the controllable variables as data are received from the experiment and interpreted.

ICE SYMPOSIUM 1970
REYKJAVIK

INSTRUMENTATION FOR ICE INVESTIGATIONS IN THE NIAGARA RIVER

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Ontario Hydro is investigating, as an International Hydrologic Decade project, the formation and action of ice in the upper Niagara River with special reference to anchor ice and other underwater ice forms. Such ice at times causes a considerable reduction in the flow of the river and a corresponding reduction in hydro-electric power production. The Meteorological Branch (Department of Transport, Canada) is also investigating the energy balance of the same reach of river and its relation to the production of surface ice in particular. Instrumentation for both projects are essentially similar and the two entities were able to co-ordinate their requirements. Only a note mentioning the location of meteorological instruments provided by the Meteorological Branch is given here since a description of these has been given elsewhere. This paper chiefly concerns itself with a survey of the instruments used to measure and record water temperature and equipment to facilitate the observation of frazil and anchor ice in the natural water environment.

Dans le cadre de la Décennie Hydrologique Internationale l'Hydro-Ontario étudie la formation et l'effet de la glace dans les eaux du Niagara supérieur. Cette glace cause à certains moments une réduction considérable du débit de la rivière, réduisant par conséquent la production d'énergie hydroélectrique. La Direction de la Météorologie canadienne étudie également le bilan énergetique du Niagara ainsi que son rapport à la formation de la glace superficielle en particulier. L'appareillage pour ces deux projets étant du même genre, les deux organismes ont pu coordonner leurs commandes. Les appareils météorologiques étaient fournis par la Direction de la Météorologie. L'exposé traite principalement les instruments de mesure et d'enregistrement de la température de l'eau et l'apparsillage utilisé pour l'observation du frazil et de la glace de fond dans l'eau qui constitue leur milieu naturel.

1. INTRODUCTION

Coinciding with the establishment of the International Hydrologic Decade (I.H.D.) sponsored by UNESCO, Ontario Hydro had an interest in a study to investigate the formation and action of anchor and other underwater ice forms and their influence on the hydraulic regime of the river. The Upper Niagara River covering the reach from the outlet of Lake Erie to the International Control Structure upstream of Niagara Falls was selected as the area for investigation. Figure I is a map of the study area showing the main geographic features important to the investigation.

The Meteorological Branch (Department of Transport, Canada) was also considering a study of the energy balance on a river during the winter period as an I.H.D. project and the Niagara River satisfied their requirements. Accordingly, a basis for co-ordination was agreed upon such that the Meteorological Branch would supply and install instrumentation dealing with the atmospheric environment and Ontario Hydro would provide and install instrumentation dealing with the water environment and carry out the routine instrument maintenance and observational functions as well as the ice investigation and experimental programs.

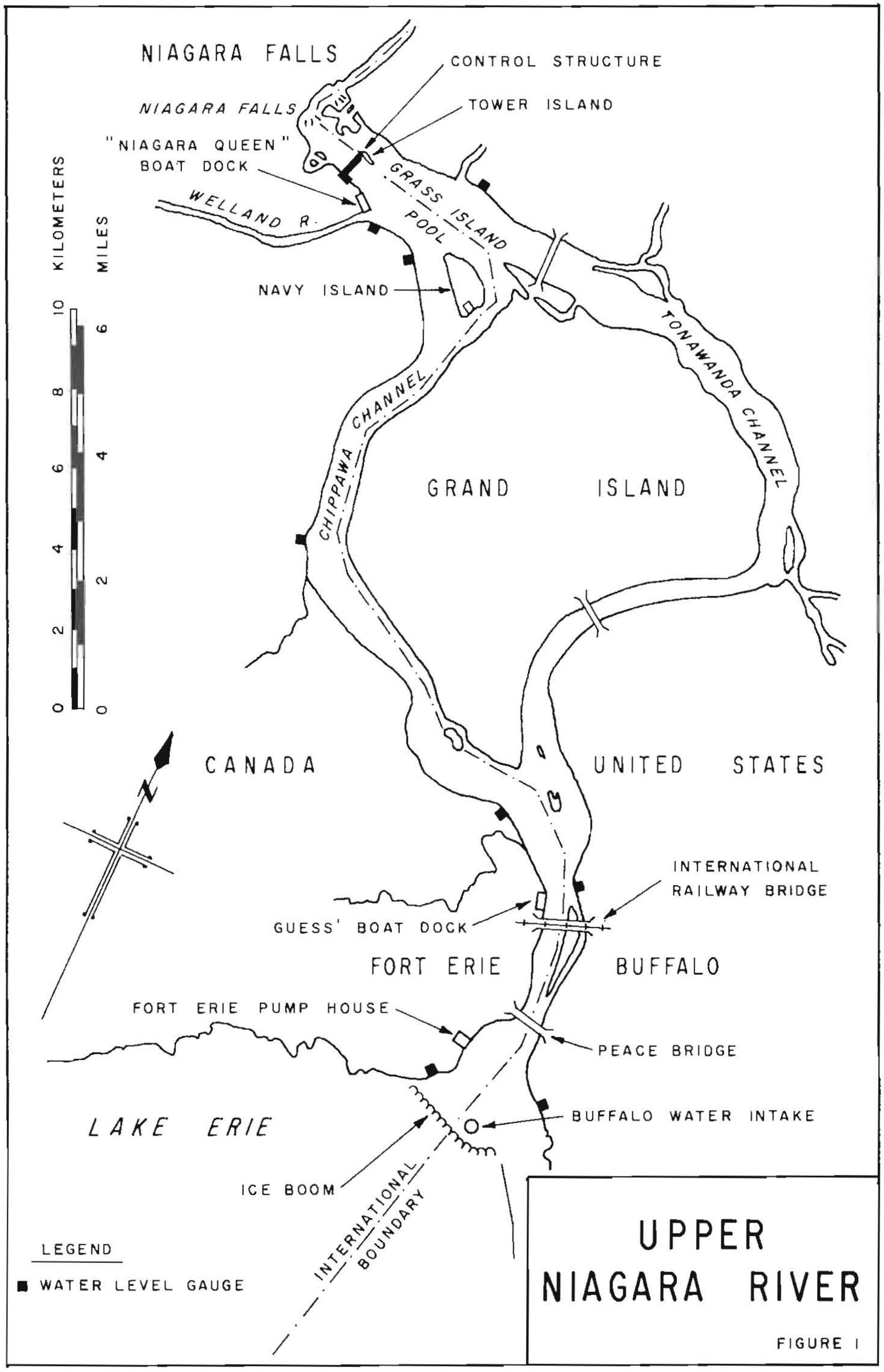
Observations on frazil and bottom ice formation and programs involving water temperature cross-section surveys, water velocities and many other investigations were carried out from the ice breaker "Niagara Queen", (Photograph 1). The vessel's normal function is to clear ice above the Control Dam, however, numerous opportunities were afforded when the "Queen" could be used for ice investigation purposes.

2. METEOROLOGICAL INSTRUMENTATION

A description of the meteorological instruments, the type of installation and their location in 1966 is given in an I.H.D. report (Cork and Chapil - 1966), (1) by the Meteorological Branch. In subsequent years, further installations were made to expand the network. A table (Figure 2) used in conjunction with the map (Figure 1) indicates the location of instruments employed during the 1970 ice season.

3. HYDROMETRIC INSTRUMENTATION

An important part of the study is the quantitative determination of the retardation effects on river flow caused by the formation of ice on the bed of the river channels. For this purpose, data on water levels and flows are necessary to make these assessments. A discussion of the instrumentation involved is beyond the scope of this paper except to point out that the location of water level gauges in the Upper Niagara are shown in Figure 1 and the Buffalo and Fort Erie gauges are used to determine the outflow of Lake Erie.



NIAGARA FALLS
NIAGARA FALLS
CONTROL STRUCTURE
TOWER ISLAND
"NIAGARA QUEEN" BOAT DOCK
WELLAND R.
GRASS ISLAND POOL
NAVY ISLAND
TONAWANDA CHANNEL
CHIPPAWA CHANNEL
GRAND ISLAND
KILOMETERS
MILES
0 2 4 6 8 10
0 2 4 6
CANADA
UNITED STATES
INTERNATIONAL RAILWAY BRIDGE
GUESS' BOAT DOCK
FORT ERIE
BUFFALO
FORT ERIE PUMP HOUSE
PEACE BRIDGE
BUFFALO WATER INTAKE
LAKE ERIE
ICE BOOM
INTERNATIONAL BOUNDARY
LEGEND
WATER LEVEL GAUGE
UPPER NIAGARA RIVER

FIGURE I

METEOROLOGICAL INSTRUMENTATION

Location	Parameter Measured	Type of Instrument
Buffalo	Wind	Anemograph
Fort Erie	Air Temperature	Thermograph
Pumphouse	Air Temperature	Max-Min Thermometer
	Humidity	Hygrograph
	Precipitation	Recording Gauge
	Precipitation	Standard Rain Gauge
Guess Boat Dock	Radiation	Recording Net Radiometer
Navy Island	Air Temperature	Thermograph
	Air Temperature	Max-Min Thermometer
	Humidity	Hygrograph
Tower Island	Air Temperature	Thermograph
	Air Temperature	Max-Min Thermometer
	Humidity	Hygrograph
	Precipitation	Recording Gauge
	Wind	Anemograph
River Control	Air Temperature	Thermograph
Office	Air Temperature	Max-Min Thermometer
	Humidity	Hygrograph
	Wind	Anemometer: Centre of Dam
		Anemograph R.C.O.
	Radiation	Sensor Pier 2
		Recorder R.C.O.
Ice Breaker (Niagara Queen)	Radiation	Recording Net Radiometer

Figure 2

4. WATER TEMPERATURE

A knowledge of water temperatures is fundamental to an understanding of the mechanism of ice formation be it surface, frazil or bottom (anchor) ice and the calculation of energy exchanges. The precision of measurement must be of a high order especially at the freezing point to obtain meaningful data.

Open Scale Electric Water Thermometers

Two instruments have been employed as a standard for calibrating thermographs and other thermometers as well as for measuring point temperatures beneath the ice in Lake Erie and at other locations (Photograph 2).

The design, developed by the Research Division of Ontario Hydro, uses a thermistor sensing probe in conjunction with a wheatstone bridge manually balanced by operating three resistance decade switches using an integrated amplifier and 1 ma meter as a null detector. The instruments are fully portable and interchangeable as between sensors and require a 9-volt amplifier battery and two D size, 1½-volt cells for the bridge.

Two styles of probe have been made. One, for general use, is composed of four thermistors connected in series with a nominal total resistance of 23,000 ohms at 0°C. The other probe designed specifically for use in frazil ice studies is quite fragile and consists of a single 22,606 ohm at 0°C thermister mounted in a polycarbonate housing with the thermister bead covered with a very thin coating of polyvinyl chloride.

The instruments are capable of measuring water temperatures with an accuracy of about ± 0.005°C or better at the freezing point. The speed of response to temperature change is quite rapid for both sensors, however, care must be taken when using the four-bead probe to ensure that the water velocity is adequate to remove the heat from the rather large head.

Water Temperature Recorders

Permanent Installations. - Two water temperature recorders are installed at either end of the river reach in electrically heated shelters. One is located at the Niagara Queen Boat Dock upstream of the Control Structure, the output being telemetered to the Control Dam. The other is at Guess Boat Dock in Fort Erie. Photographs of the installation at Guess Boat Dock illustrate in Photograph 3, the recorders for water temperature and net radiation and in Photograph 4 the instrument shelter and radiometer mounting.

The recorders are self-balancing direct current bridge instruments requiring a 60 hertz, 115 volt supply. The temperature sensors are 100 ohm at 0°C platinum resistance bulbs. Two temperature ranges are provided, -1°C to + 2°C and 0°C to +30°C. An automatic range change switch is incorporated to operate at about 1.8°C, however, the range can be selected manually. Provision has been made to check the electrical calibration by incorporating a switch and circuitry to give an indication at 0°C and 10°C.

The recording accuracy of the system is nominally ± 0.01°C over the three-degree range and 0.1°C over the 30-degree range. By careful calibration and frequent checking, recordings of water temperature near the freezing point appear to be well within the ± 0.01°C tolerance.

Portable and Semi-Portable. - A Bristol two-point recorder of basically similar design to the foregoing was acquired to record temperatures at two depths simultaneously, (Photograph 5). This instrument is used on board the "Queen" for many investigations such as recording water temperatures while in motion for water temperature surveys across sections of the river, or when anchored, for frazil and bottom ice investigations. Fortunately the "Queen" provides a well regulated 60 hertz, 115-volt power supply which is required by the recorder. Many useful and interesting recordings of water temperature were obtained. It has also been used to record water temperature from a moving boat which did not have an alternating current supply. Power was provided by 12-volt lead-acid batteries through an inverter. Rather poor performance was experienced due to inverter frequency stability problems.

The instrument employs 200 ohm platinum resistance elements in the probes using a heavy duty 3-wire transmission cable. Three ranges are provided consisting of -05°C to 1.5°C, 0 to 20°C and 10 to 30°C. The instrument has an accuracy of ± 0.01°C on the narrow range and ± 0.1°C on the 20°C ranges. The recorder prints a black or red dot every eight seconds, one for each sensor probe.

The drawing, Figure 3, below is an example of the results obtained from a cross-section survey using the thermograph. The water temperature is recorded during several traverses of the channel with the sensors mounted 4 feet (120 cm) apart in the vertical. For each successive traverse, the sensor assembly is lowered to a greater depth by 10 foot (300 cm) increments.

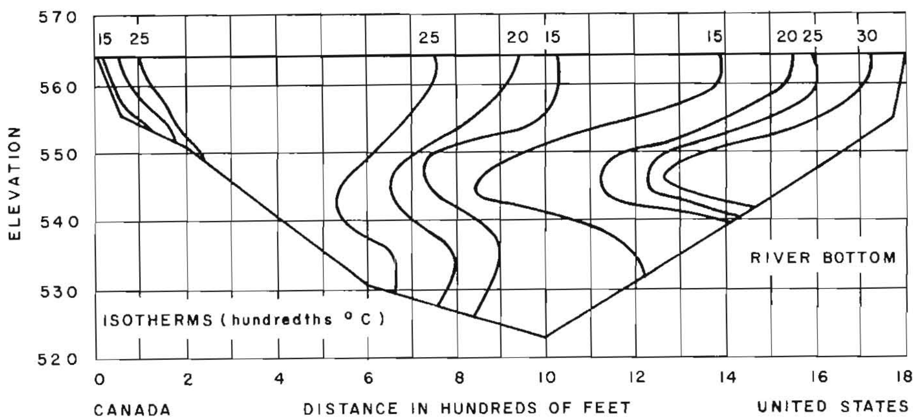


FIGURE 3 - WATER TEMPERATURE - GUESS BOAT DOCK - 0535 HOURS - FEB. 27, 1970

A second completely portable recording thermometer was recently acquired. The Research Division of Ontario Hydro carried out the design to modify an Esterline Angus Port-A-Graph Potentiometer recorder employing a thermistor for temperature sensing. The recorder can operate from a 60 Hertz, 115-volt supply, $1\frac{1}{2}$-volt dry cells, rechargeable nickel-cadmium cells (built-in charger) or external 12-volt automotive battery. The power consumption is only 2.8 VA. Twelve chart speeds from 20 to 360 mm per minute and 14 voltage spans from 2 mV to 50 V can be selected by front panel controls. Four temperature ranges are provided from range -0.1°C to 0.9°C to range -1°C to 9°C.

The instrument (Photograph 6), was installed at the Buffalo Water Intake to record temperature of water leaving Lake Erie from beneath the ice sheet at the Ice Boom. It was also used in a number of experiments involving the detection and formation of frazil ice.

5. UNDERWATER ICE OBSERVATION EQUIPMENT

Underwater Lighting

An underwater lamp (Photograph 7) was obtained to provide nighttime illumination for observing and photographing frazil and anchor ice formations. The lamp is mounted on the "Queen" by an assembly permitting four degrees of freedom of movement (Photograph 8). It is provided with Fairings and is sufficiently robust that it can be used while the "Queen" is in motion up to a speed of about 3 feet (one meter) per second.

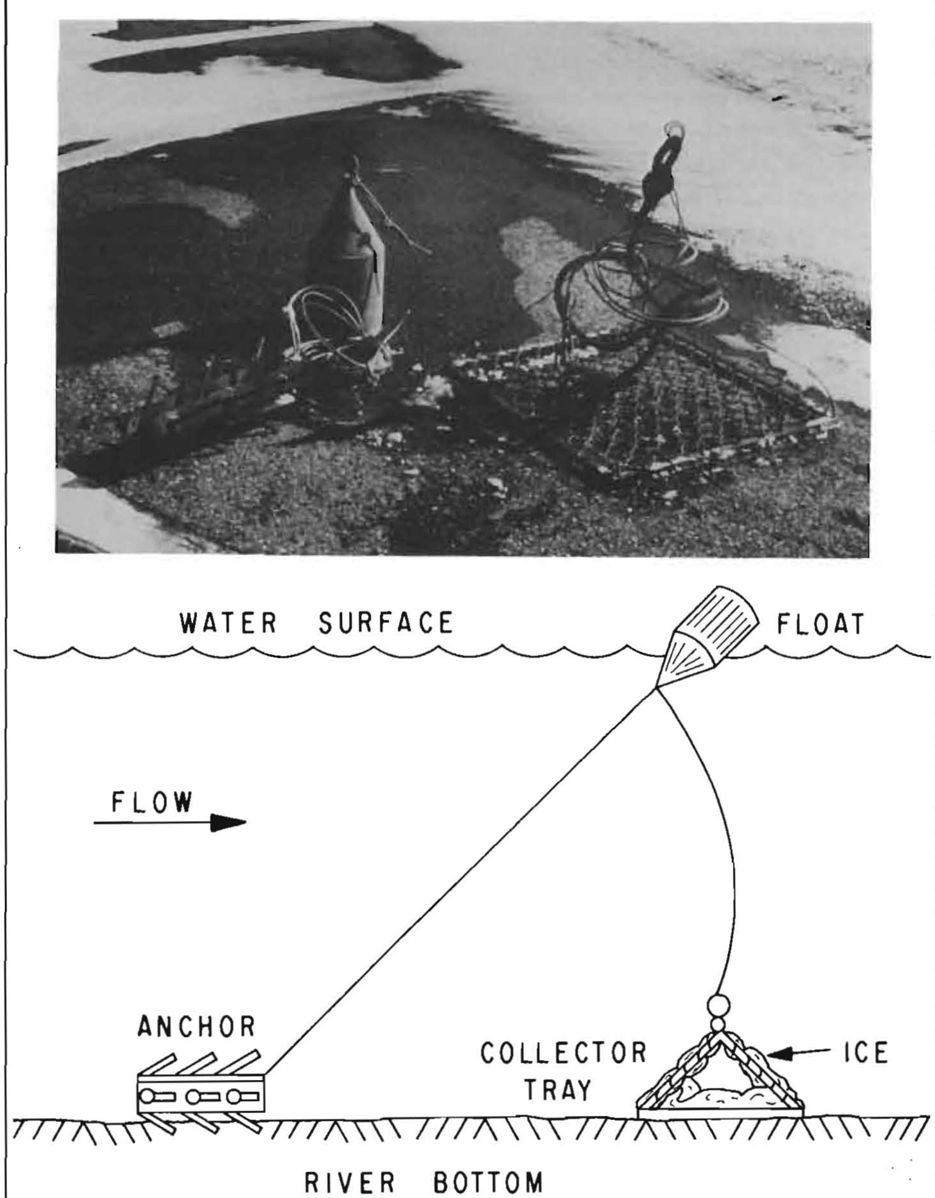
Bottom Ice Collector Trays

Wire screen trays were placed on the bottom of the river to detect the presence of anchor ice. Three or four were placed across a section which were lifted early in the morning for examination. Figure 4 is a schematic drawing illustrating deployment of the ice collector tray in the water. A photograph of the component parts of the assembly is included with the drawing.

Frazil Ice Strainer and Sampler

A number of experiments were carried out to detect the presence of frazil ice crystals in the flowing water. A frazil ice strainer (Photograph 9), was employed which could be lowered to a selected depth and the door on the upstream face opened for a short period in order to strain out ice on the screen. The assembly was raised out of the water and the screen removed and examined.

A frazil ice sampler (Photograph 10) was used in experiments attempting to quantify the amount of ice in a fixed (1 litre) volume of water. The device was lowered to a selected depth, the sample of ice and water obtained and the sampler, after retrieval examined. Internal temperature was recorded during the process.



COLLECTOR TRAY

FIGURE 4

6. CONCLUSION

In addition to the more important instruments described above, a number of pieces of equipment were produced and used for various experiments which for lack of space cannot be included here.

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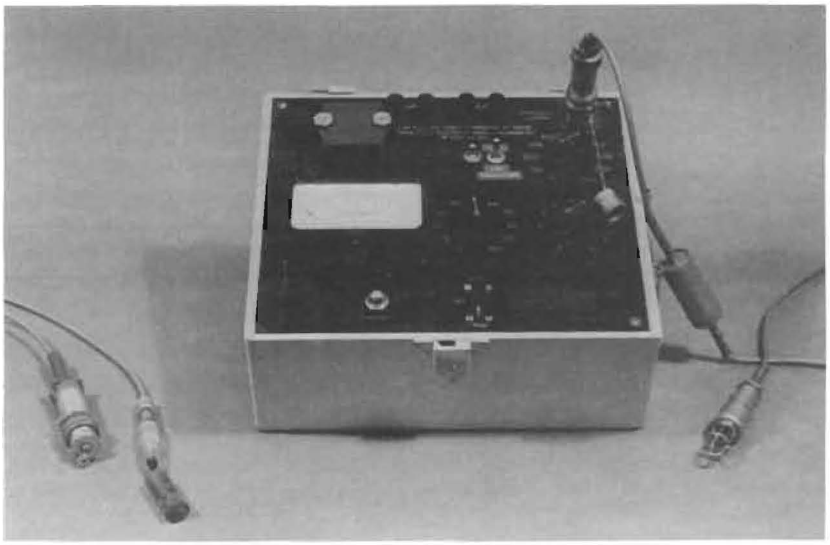
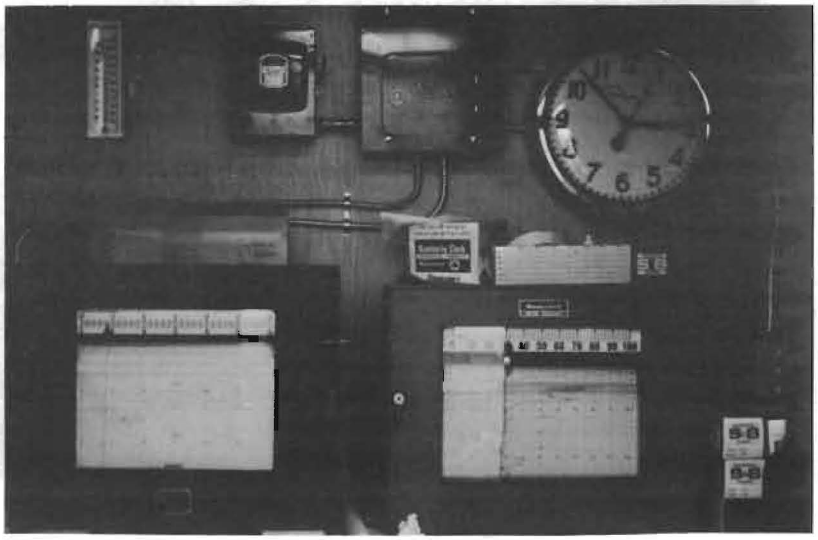
   Instrumentation for Study of Ice Formation on the Niagara River Can. Dept. Transport, cir. 4496, Tec. 627, pp. 11, 1966.

2. Wigle T.E.
   Investigations Into Frazil, Bottom Ice, and Surface Ice Formation in the Niagara River
   Proc. I.A.H.R. Symp. Ice and Its Action on Hydraulic Structures, September 1970.

Photograph 1 - Ice Breaker "Niagara Queen"
Length - 45 feet (13.7 m), beam - 15 feet (460 cm), draught - 3 feet (1 m)

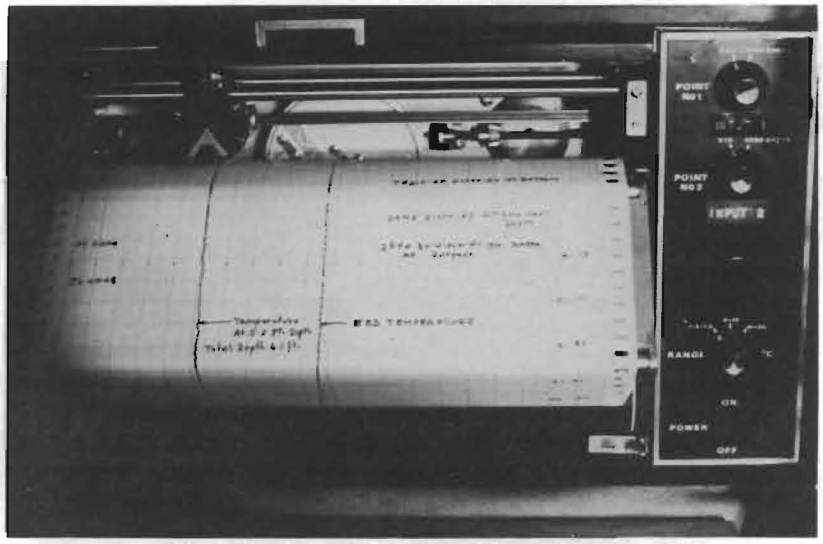


Photograph 2 - Open Scale Electric Water Thermometer.
Two thermister probes are shown: one conteining 4 thermisters, the other a single thermister.

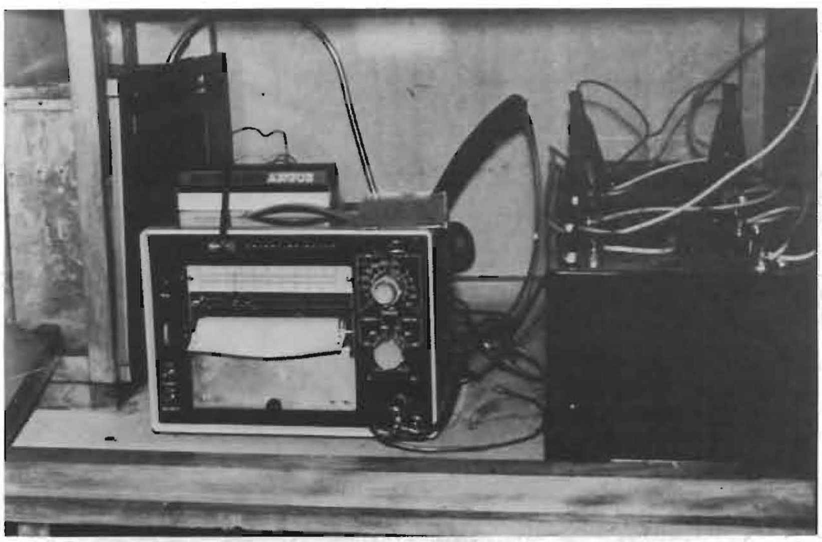


Photograph 3 - Water Temperature and Radiation Recorders.
Housed instrument shelter at Guess Boat Dock, Fort Erie.
Shelter heated by 115 volt, 60 Hertz heater.

Photograph 4 - Instrument Shelter and Net Radiometer Mounting.
At Guess Boat Dock.

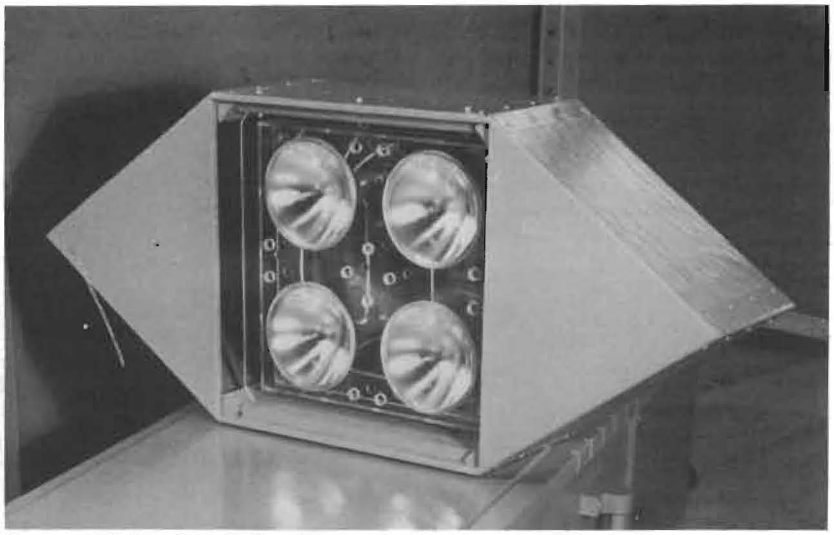


Photograph 5 - Two Point Water Temperature Recorder.



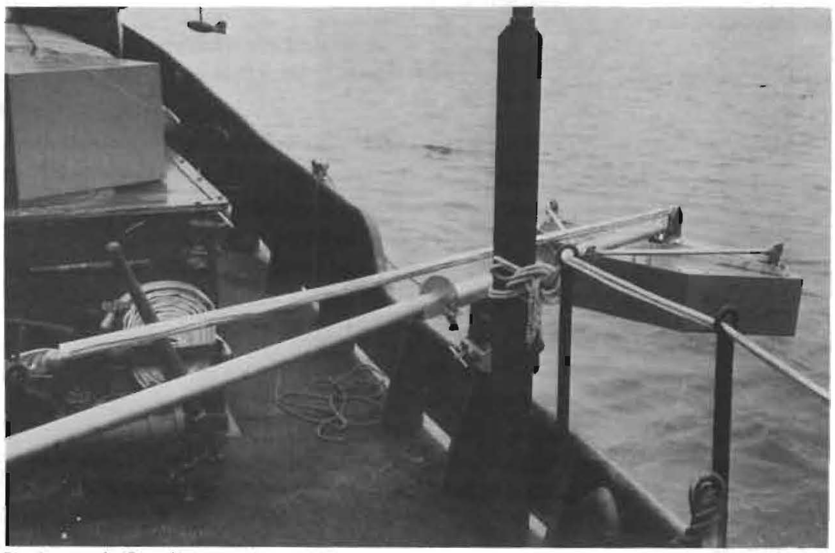
Photograph 6 - Portable Water Temperature Recorder.

Housed in plywood box. Heat provided by thermostatically controlled, sealed combustion chamber, propane heater. Electric power supplied by 4, 12 volt Lead-acid batteries; total capacity 280 ampere hours yealding 35-40 days operation.



Photograph 7 - Underwater Lamp.

Four 115 Volt sealed flood Lamps of 300 watts each mounted on ½ inch (13 mm) plexiglass sheet enclosed by plywood housing. Note fairings to provide streamlining.



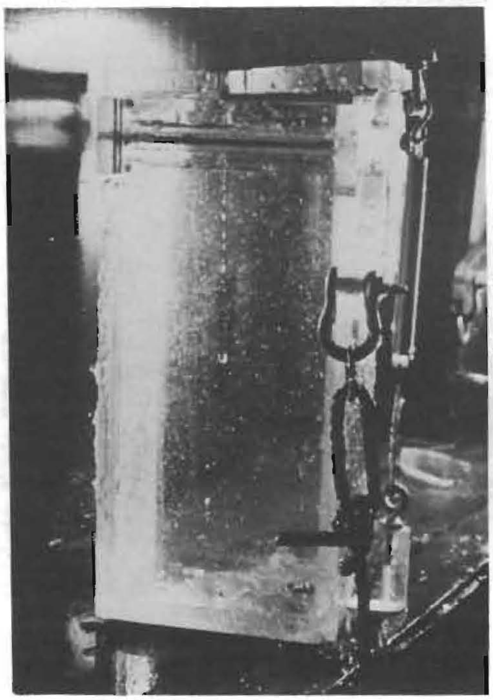
Photograph 8 - Underwater Lamp.

Lamp shown as mounted on "Queen". Used in water depths up to 6 to 8 feet (2 to 3 meters).

Photograph 9

Frazil Ice Strainer

Sheet metal box, 12 inches (30 cm) cube with hinged door at upstream end and removable screen (1mm mesh) at downstream end. Note large tail fin and weight to stabilize unit in current.



Photograph 10

Frazil Ice Sampler

Plexiglass container, contents 1 litre. Port at top opened by cam arrangement actuated by lanyard from ice breaker deck.

Discussion by MR. G.D. ASHTON

In the field measurement program recently conducted in Iowa, sub zero water temperatures were measured but in all cases the corresponding ice points made using river water samples were at slightly lower temperatures. There was little, if any frazil ice in the flow at these times.

In leboratory experiments the water temperature generally decreased to -0.02 to -0.03°C before the initial formation of frazil ice in the flow. Upon the formation of the frazil the temperature increased to a value very near 0°C, in a manner similar to the behavior described in detail by Carstens in reporting experiments conducted at Trondheim. In one case the temperature decreased to -0.5°C at which time there was a sudden massive formation of frazil. The conditions under which this unusual super cooling occurred have since been repeated but the large super cooling has not again been observed.

AUTHOR'S REPLY

It is true that the ice point of natural waters is somewhat below 0°C but it is probably not lower than -0.005°C for Niagara River water.

Observations in the Niagara River indicate that frazil ice appears in the surface layer of water when its temperature became super-cooled to about -0.01°C. Water temperatures as low as -0.07°C which is I believe a theoretical lower limit have been recorded at 2.5 m. depth in very cold weather.

Discussion by MR. T.M. DICK

Mr. Arden asked in his presentation if other investigators had obtained temperature readings a few hundredths of a degree below the freezing point of water.

In the course of a survey taken on behalf of the Hydraulic Section at the National Research Council, a thermistor calibrated to 0.01°F was installed in a fast flowing outlet from the Lake of Two Mountains in the Ottawa River. The velocities were too high to allow the formation of an ice cover at the thermistor location but immediately upstream, the flow emerged from under a continuous ice cover. The air temperature was low e.g. -20 to -30°F. Water temperatures slightly below zero were detected. The equipment was checked and found to be in order.

The condition did occur more than once but not sufficiently often to relieve doubts as to the correctness of the measurements. At the time it was thought that slight super cooling could have been present but it was just as likely to be a slight malfunction of the equipment. In summary, at the time, the super cooling effect was considered not proven.

AUTHOR'S REPLY

Near the close of my presentation, I was explaining that ice crystals forming on the sensor tip causes the indicated temperature to be higher than the true water temperature with a tendency to indicate 0°C after a period of time. The question then posed was whether we can be absolutely sure of the correctness of our temperature measurements and how can the problem of icing be overcome. I suggested that this would be investigated further in the Niagara River. Mr. Dick's comment I believe reinforces my position in this regard.

ICE SYMPOSIUM 1970
REYKJAVIK

FLOW MEASUREMENTS OF ICE-COVERED RIVERS ON HOKKAIDO

LE JAUGEAGE DES COURS D'EAU GELÉS EN HOKKAIDO

Koji Ohashi, Managing Director — Hokkaido Electric Power Co. — Sapporo
Tadashi Hamada, Civil Engineer — Civil Engineering Department — Japan

The authors describe the methods and results of the ice-covered river flow measurements carried out in the streams with comparatively small catchment area on Hokkaido, Japan. They study the interrelationship among velocity distribution, irregularity seen in the cross-sectional form and roughness coefficient of stream flow under the ice cover, and assume that roughness of the underside of the ice cover depends upon the hydraulic and climatic conditions as well as the form effect of the river section. The necessity of conducting further field measurements is emphasized to establish the correlation prescribing roughness factor under the ice cover.

Les auteurs exposent la méthode et les résultats des jaugeages des cours d'eau recouverts de glace pour des ruisseaux dont les bassins versants sont relativement petits dans la région du Hokkaido, Japon. Ils étudient la relation qui existe entre la repartition de la vitesse, les variations de la forme de la section transversalle de la couche de glace du torrent et le coefficient de rugosité de la face inférieure de la couche de glace. Ils supposent que la rugosité de la face inférieure dépend des conditions hydraulique et climatique aussi bien que de l'effet de la forme de la section du torrent. Et ils insistent sur la nécessité de continuer à recueillir d'autres données sur le terrain afin d'evaluer l'importance que joue le facteur de rugosité de la face inférieure de la couche de glace.

1. Ice-Covered Rivers on Hokkaido

Hokkaido lies at the northern extremity of Japan, and is situated between 41° N and 46° N. In wintertime from December to March, cold Siberian high atmospheric pressure rules the island, and the resulting coldness causes most of the rivers to be frozen.

Of 34 rivers and streams of which flow measurements are periodically carried out by the Hokkaido Electric Power Co., the state of ice-cover formation is as shown in Fig. 1. As seen in Fig. 1, ice is not always formed over all rivers on Hokkaido, and this may be explained by various reasons such as air temperature, snow fall, Froude number, the inflow of volcanic underground water, the peak regulation of up-stream hydro-power stations with daily discharge of water, and so forth.

2. Flow Measurements of Ice-Covered Rivers

In the instance of large rivers with complete ice cover, certain researchers state that the ice cover moves up and down with the change of flow discharge, that the underside of the ice cover is smooth, and that the underside of the ice cover shows a good correlation with flow discharge. However, in the instance of rivers and streams, medium or minor in size, as we have found on Hokkaido, the ice level does not show a good correlation with flow discharge, since the ice forms a bridge or an arch over the water surface and a space is formed between ice and water. Sometimes water runs down over the ice, and in most cases the underside surface of the ice cover is extremely rough and irregular because of the variety in formative processes of the ice.

In order to make accurate streamflow measurements in the ice covered rivers, therefore, it is not suitable to rely upon the water level, but to carry out minute velocity measurements at suitable intervals. For this reason, it is our established practice to make a series of streamflow measurements by using current meters through holes in the ice at adequate intervals, quite the same way as in the stream without ice cover (see Fig. 2).

3. Velocity Distribution in Ice-Covered Streams

When ice covers a stream, the stream becomes a closed conduit with lower velocity/discharge for the same cross-sectional area of the stream flow in the non-freezing seasons.

We conducted measurements at several gauging stations during last winter (1969-70), and compared the results with those that had been measured in non-freezing seasons at the same place on the rivers, and furthermore, with almost equal flow section. Some typical examples are shown in Table 1.

For almost equal cross-sectional area, the discharge decrease in the ice-covered streams ranges 3-29 % as compared with the non-freezing seasons, as seen in Table 1. Furthermore, the velocity distribution curves of the ice-covered streams are considerably different from those of non-freezing free flow, as shown in Figs. 3 and 4, and the change in the velocity distribution curves seems to become appreciably greater as the stream becomes smaller.

Table 1. Comparison of Measured Data

No	Name of gauging station	Catchment area	Measured during the non-freezing season			Measured during the ice-covered season			Rate of difference in discharge between non-freezing and ice-covered seasons
			A_1	v_1	Q_1	A_2	v_2	Q_2	
			Cross-sectional area	Average velocity	Discharge	Cross-sectional area	Average velocity	Discharge	$\frac{A_1 \Delta v}{Q_1}$ *
①	②	③	④	⑤	⑥	④′	⑤′	⑥′	
	(unit)	km²	m²	m/sec	m³/sec	m²	m/sec	m³/sec	%
1	Chitoh	2,505	85.69	.498	42.7	93.32	.360	33.6	-29
2	Setose	876	16.97	.374	6.37	16.54	.336	5.55	-10
3	Sahoro	237	7.62	.465	3.55	6.74	.420	2.82	-10
4	Ishikari	294	7.46	.529	3.95	8.01	.516	4.13	-3

* Figures in this column were computed by the following formula, because the water stages and sectional areas were not strictly the same in the measurements:

$$\frac{A_1 \Delta v}{Q_1} = \frac{\Delta Q}{Q_1} - \frac{v_1 \ \Delta A}{Q_1}$$

4. A Consideration on Roughness Factor

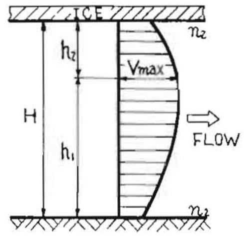
As stated above, the discharge in the ice-covered stream considerably decreases as compared with the non-freezing, open surface flow. We find an extreme example of the discharge decrease amounting to nearly 30 %.

In order to explain such remarkable decrease of flow discharge, we counted the roughness factor (Manning's n) of the underside of the ice cover.

Water flow under ice-cover is represented as follows by Pavrovski[1)]:

$$\frac{h_1}{H} = \frac{(n_1/n_2)^{1.33}}{1 + (n_1/n_2)^{1.33}} \quad \cdots\cdots (1)$$

where H is the full depth, h_1 is the depth from the river bed to the V_{max} point, n_1 is Manning's roughness coefficient of the river bed, and n_2 is that of the underside of the ice cover.



Formula (1) will be transformed:

$$n_2 = n_1 \left(\frac{H - h_1}{h_1} \right)^{\frac{1}{1.33}} \qquad \text{- - - - - - - (2)}$$

Accepting Pavrovski's formula, we can obtain n_2 by using n_1 that are computed from the observations in the non-freezing seasons, and the values n_2 thus obtained are as shown in Table 2.

Table 2. Comparison of n_1 and n_2

No.	Gauging station	n_1	$n_2(\times 10^{-4})$ measured											Average n_2
1	Chitoh	0.041	(L)	(14)	(18)	(26)	(34)	(42)	(54)	(66)	(78)	(90)	(102)	0.029
			n_2	31	35	41	29	41	25	23	20	27	21	
2	Setose	0.046	(L)	-	(9)	(13)	(17)	(21)	(25)	(29)	(33)	-	-	0.045
			n_2	-	39	35	45	62	42	57	32	-	-	
3	Sahoro	0.040	(L)	(12)	(14)	(16)	(18)	(20)	(22)	(24)	(26)	-	-	0.020
			n_2	19	25	17	18	20	29	14	18	-	-	
4	Ishikari	0.034	(L)	(16)	(18)	(20)	(22)	(24)	(26)	(28)	(30)	-	-	0.016
			n_2	21	12	15	13	15	15	17	17	-	-	

n_1 . . . . . Manning's roughness coefficient of river-bed, measured in non-freezing seasons.

n_2 . . . . . Manning's roughness coefficient of underside of ice-cover, computed from Pavrovski's formula.

L . . . . . Distance in meters between measured point and origin.

Scrutinizing Table 2 will show us several interesting facts:

1). The values n_2 are very variable even in the same section of the gauging station. Thus, $n_2 = 0.020 \sim 0.041$ around the average 0.029 for Chitoh G.S., $n_2 = 0.032 \sim 0.062$ around 0.045 for Setose G.S., $n_2 = 0.014 \sim 0.029$ around 0.020 for Sahoro G.S., and $n_2 = 0.012 \sim 0.021$ around 0.016 for Ishikari G.S.

2). The underside roughness of ice cover varies widely in value, $0.016 \sim 0.045$, while the roughness of river bed is almost the same, $0.034 \sim 0.046$.

3). In spite of the apparent irregular form of the underside ice cover, $n_2/n_1 \fallingdotseq 0.5 \sim 1$ and $n_2 < n_1$. It should be noted that in some cases $n_2 \fallingdotseq n_1$.

As seen in Figs. 3 and 4, the flow section under the ice cover is appreciably irregular, which suggests that certain form irregularity effect may be included into the value n_2. But, examining the measured data at Ishikari and Chitoh gauging stations as shown in Figs. 3 and 4 as well as in Table 2, we find that n_2 at Ishikari is smaller than that at Chitoh, though the sectional form at Ishikari is far more irregular than that at Chitoh. It upsets our expectation of finding a direct correlation between form irregularity and roughness factor of the underside of the ice cover.

In order to explain the problem more clearly, we must take into account the more abundant, accurate observation results of the hydraulic and climatic conditions which may affect the roughness factor of the underside of the ice cover. Such observation is, to our regret, insufficient so far. It is necessary for us to carry out further field measurements on various gauging stations under different conditions in order to establish a correlation to prescribe the roughness factor under the ice cover.

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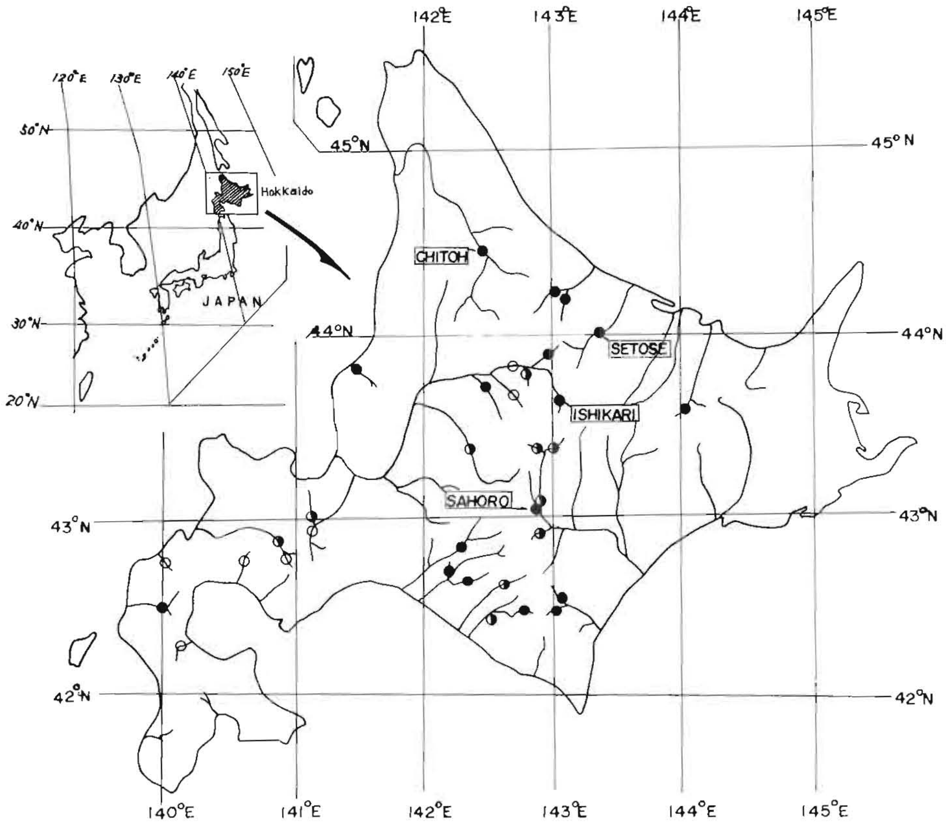

LEGEND:

● ----- GAUGING STATION WITH COMPLETE ICE-COVER IN WINTERTIME ---- 17 STATIONS

◐ ---- 〃 〃 WITH PARTIAL ICE-COVER 〃 ---- 10 〃

○ ----- 〃 〃 WITHOUT ICE-COVER 〃 ---- 7 〃

FIG. 1 GAUGING STATIONS
OPERATED BY HOKKAIDO ELECTRIC POWER CO.

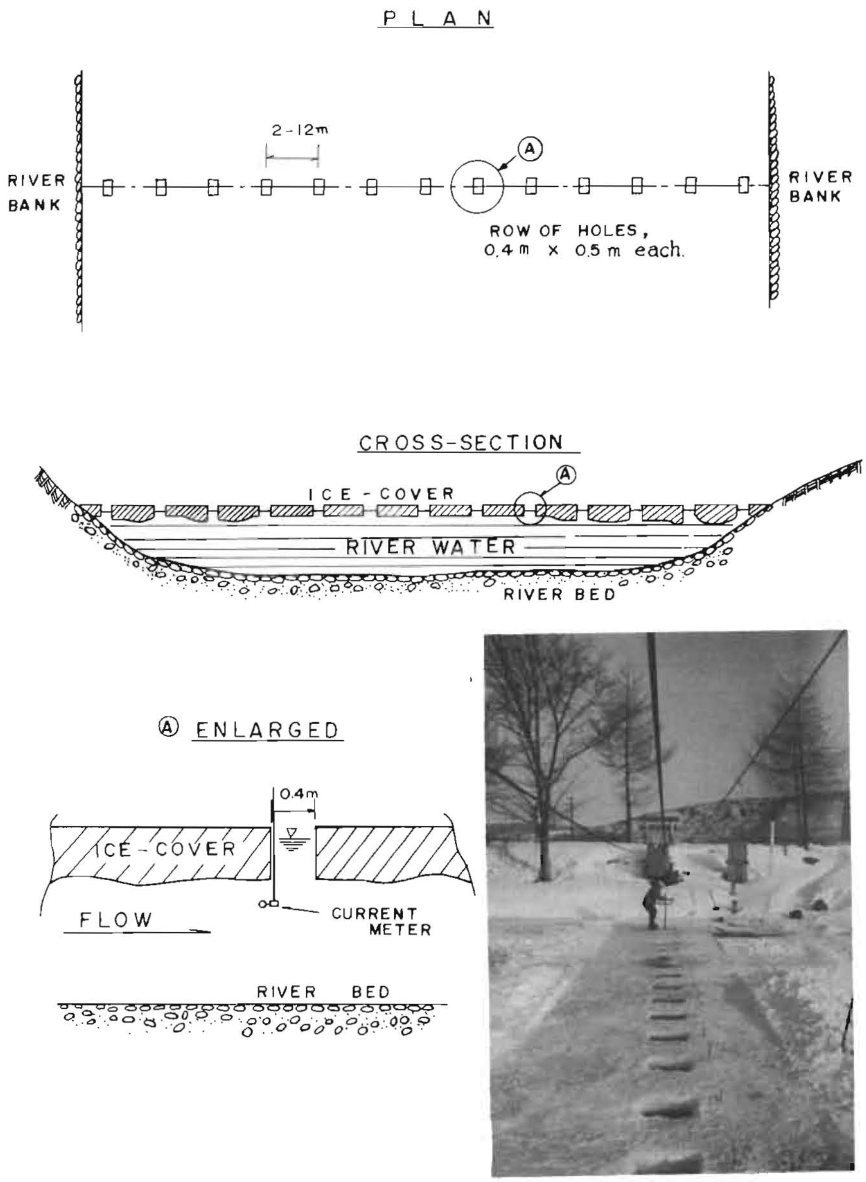

FIG.2 VELOCITY MEASUREMENT IN WINTERTIME AS PRACTISED ON HOKKAIDO, JAPAN.

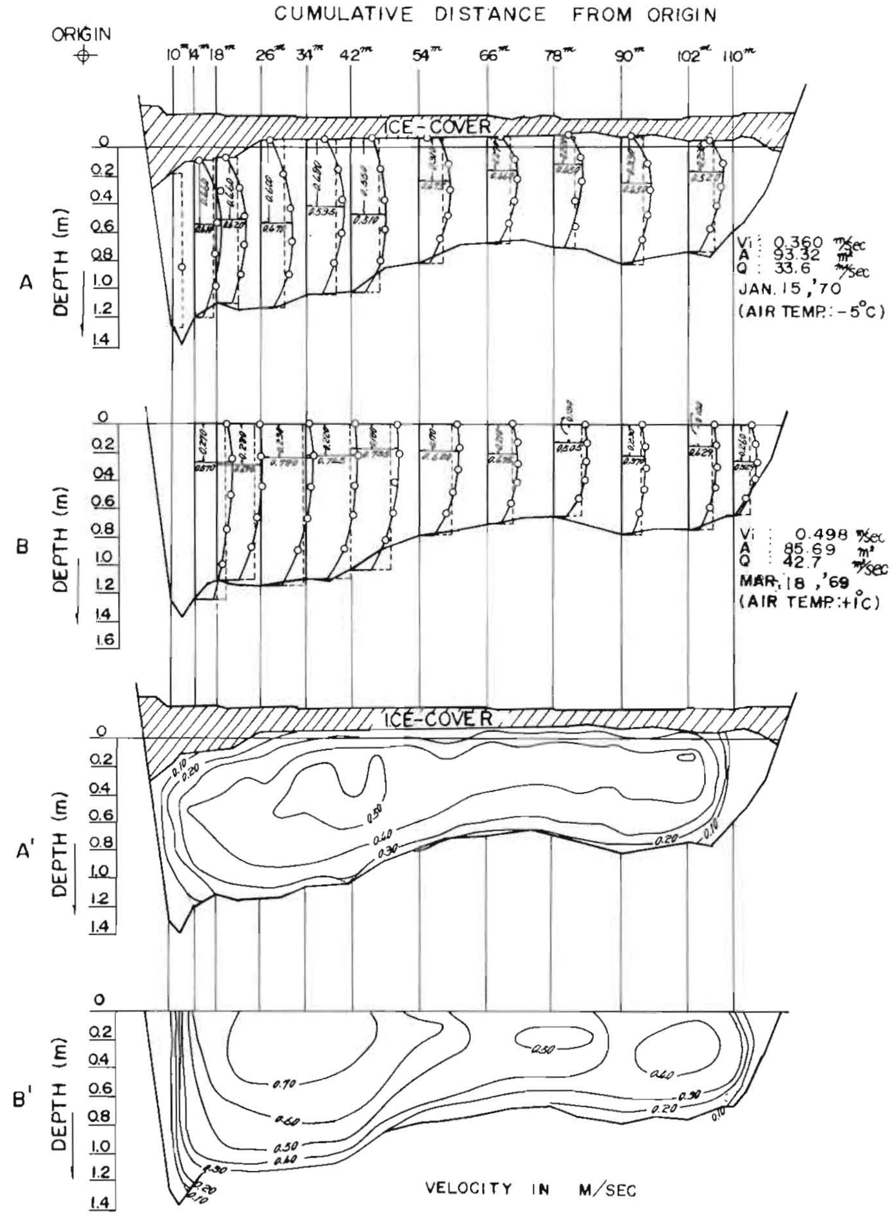

FIG. 3 VELOCITY DISTRIBUTION (CHITOH GAUGING STATION)

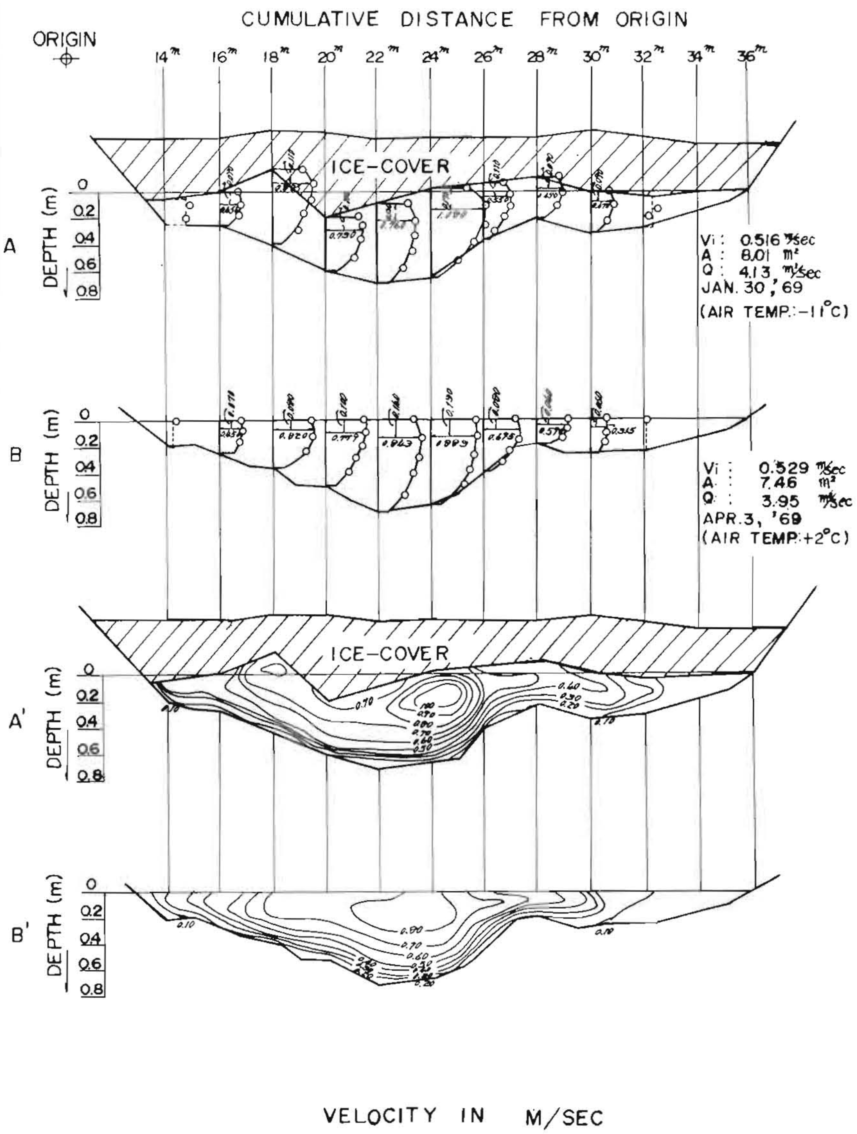

FIG. 4 VELOCITY DISTRIBUTION (ISHIKARI GAUGING STATION)

1.4

On paper by Ohashi, K. and Hamada, T. :

DISCUSSION by Coley, Ron W.

The authors conclude that the depth at which the maximum velocity occurs (h_1) depends on the flow turbulence due to the irregular form of the underside of the ice-cover in addition to the roughness coefficient of both the river bed and the underside of the ice.

I wish to suggest that the point of maximum velocity is also influenced by the effect of secondary currents in a manner similar to the effect of secondary currents in open channel flow.

The variation of the roughness coefficient for ice, n_2, across the channel is not surprising as it is reasonable to assume that the roughness can vary across the underside of the ice in a manner similar to the variation of roughness across a channel bed.

DISCUSSION by Yamaoka, I.

I agree with Mr. Coley's comments that suggested the effect of secondary currents and pointed out the similarity of the transverse variation of two roughness coefficients for ice cover and channel bed. And I suppose the authors misused or mistranslated the term of turbulent flow instead of the secondary flow with transverse velocity components in the original paper of the preprints.

(The authors have corrected some contradictions in their original paper and have made better changes to make their idea clear after careful reconsideration. They are grateful for the useful discussion and suggestion by the reviewer and Mr. Coley.)

ICE SYMPOSIUM 1970
REYKJAVIK

CALCULATION OF FRAZIL ICE PRODUCTION

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SYNOPSIS

In certain important reaches of some Icelandic rivers the ice production is predominantly frazil ice that is incessantly carried downstream. Calculations of the frazil ice discharge are of importance in connection with design and energy production studies and ice forecasts for planning operations. Methods for such calculations have been developed and good agreement between calculated and observed ice discharge has been obtained at the Burfell dam site on the Thjorsa River. This paper contains a description and discussion of the various factors that form the basis for such calculations with special reference to the Burfell dam site.

INTRODUCTION

The most serious ice problems at some potential power plant sites in the Thjorsa River System, Southern Iceland, and some other Icelandic rivers as well, are because of enormous quantities of frazil ice produced in reaches which remain open throughout the winter. These problems will be most pronounced at run-of-the-river power plants where accumulations of frazil ice might block the intakes for longer or shorter periods. The Burfell Power Plant, the first and, as yet, the only power plant on the Thjorsa River System is an excellent example of this type. A part of the river discharge is here used to flush the frazil ice over a specially constructed diversion dam (described in a paper by G. Sigurdsson (1)).

Calculations of the ice production at such sites are of considerable importance, e.g. for predictions to plan the operation from day to day, and not less in energy production studies as the water used for flushing ice is lost for power. During the last few years we have been trying to develop methods for such calculations and compared to the complexity of the problem the results are very good. A description of our approach to these calculations for the Burfell dam site are given below. For general description of the rivers and their ice conditions the reader is referred to a paper by S. Rist (2).

BASIS FOR ICE PRODUCTION CALCULATIONS WITH SPECIAL REFERENCE TO THE BURFELL DAMSITE

Calculations of the amount of ice that reaches this site are practicable because overwhelming part of the ice produced upstream is frazil ice that is carried downstream incessantly.

The main factors that must be known in order to carry out these calculations are the following:

1. The heat exchanges between the river and the atmosphere.
2. The size of the open water area upstream from the site.
3. The temperature and discharge of the rivers where they enter the open water area and the temperature and magnitude of groundwater inflow.
4. The heat from frictional heating and conduction from the river bed.
5. The amount of ice that accumulates upstream as anchor ice, ice jams etc.

These factors will now be discussed briefly separately.

Heat exchanges

In the first estimates of ice production on the Thjorsa River the old formulas of Dr. Olaf Devik (3) for the heat loss from the river surface were used. During investigations 1964-66 these formulas seemed to give too little heat loss and revised formulas based on measurements in calorimeters have been used for a few years (4).

In October 1968 direct measurements of the heat loss from a small river near Reykjavík were undertaken. The water temperature was continuously recorded at two places about 1 km distant. Meteorological observations with recording instruments for air temperature, humidity, wind velocity and wind direction were made at the river. The flow velocity between the thermometers was determined with the salt velocity method. The heat loss was calculated from the rate of change in water temperature during cooling by simplified equations (4). A comparison between measured and calculated heat loss (1-2 hours means) is shown on fig. 1. The calculated values should be expected to be higher than the measured because parts of the measured reach of the river were more sheltered than the meteorological station and other reasons, but nevertheless the revised Devik formulas seem to give substantially too high values. A reasonable good agreement is obtained by using Russian formulas (Rymsha-Donchenko) for the heat loss by evaporation and convection. These formulas have been used with encouraging results in U.S.A. (5) and we are now using them, but further investigations into these matters are planned. The set of equations for the heat loss from a water surface, now in use by us, is given in an appendix. The influence of snow on the ice discharge, especially blowing snow during blizzards can be substantial. This is not taken into account in the calculations. But as days with blowing snow are usually few during the winter and the ice production because of heat exchanges is often also great on these same days, this does not seriously depreciate the calculations. It is, of course, necessary to have meteorological observations at the rivers where ice production is to be calculated. From temporary meteorological observations at various places along the Thjorsa and Tungnaa Rivers (6) it has been established that there is a close correlation between air temperature and wind velocity at these places and at the Burfell dam site.

Open water area

The open water area upstream from the Burfell Damsite can vary between 2-3 km^2 and 10-11 km^2 through the winter, but in autumn before the upper parts of the rivers become ice covered it is far greater. The area has been determined a few times by aerial surveys. We have tried to calculate this area from meteorological and hydrological observations with some success. An equation of the following form was obtained with regression analysis (the statistical work was carried out by Mr. Helgi Sigvaldason Lic. Techn., Orkustofnun):

$$DF = a - b \cdot F_1 - c \cdot S + d \cdot Q,$$

where DF is the change in area during one week, F_1 the area at the beginning of the week, S the mean heat loss from 0^o C water surface during the week, Q the discharge of Thjorsa over the week and a, b, c and d positive constants. The effect of warm weather and rain is reflected in Q. A comparison between calculated area and observed points is shown in fig. 2.

Discharge and water temperatures

The flow of the rivers in the open reaches upstream from Burfell can be divided into three categories:

a) The water that enters the open reaches from beneath ice covers at the upstream ends or from ice covered tributaries. This is a major part of the flow and has a temperature close to 0^o C throughout the winter.

b) Water from some small, warm (spring fed) tributaries which are usually open. The area of these tributaries is not included in the area that is used in the calculations. The temperature of these rivers at the confluences with the main rivers is variable from 3-4 ^{o} C down to 0^o C in extreme cold.

c) Groundwater and springs in the river channels. In the Tungnaa River at least there are springs of considerable magnitude (10-20 m^3/sec).
These springs have a nearly constant temperature of 3-4^o C. The heat from groundwater that seeps into the river channels is not considered in the calculations as nothing is known of its temperature and presumably it is not high above 0^o C in wintertime. Overland flow is negligible during cold periods.

The heat from the warm tributaries and the springs must be taken into account in calculations of ice production. Together with frictional heating the reduction in ice production is temporarily estimated 2.5-3 tons/sec in moderate cold, decreasing to 1.5 tons/sec with increasing heat loss.

Frictional heating etc.

The heat gain by frictional heating is small compared to the heat exchange with the air. But as the combined differences in head in the open reaches can be up to 530 m it can not be neglected. The discharge of the rivers is closely related to the air temperature or heat loss and this justifies taking this factor into account as a function of the heat loss.

Of course, the frictional heating could be calculated more accurately by analysing a number of streamflow records. Conduction from the river bed is certainly very small compared to other terms and is neglected in the calculations.

Accumulation of Ice

Anchor ice and border ice is more or less continuously formed during cold periods. But in our case these ice formations are however small in magnitude compared to the frazil ice and besides some of the anchor ice is always loosening and floating to the surface. During extreme cold or blizzards ice jams may form at certain locations and store a considerable part of the frazil ice produced. In the calculations of ice production the ice stored upstream from the damsite, is neglected and thus the calculated ice discharge should be somewhat too high, but often the difference is presumably smaller than the accuracy of the calculations.

CALCULATIONS AND OBSERVATIONS

Assuming stationary conditions the frazil ice discharge is calculated by the formula:

$$I = \frac{1}{80} \cdot S \cdot F - DI \;;$$

$$\frac{\text{tonn}}{\text{sec}} = \frac{1}{\text{Mcal/tonn}} \cdot \frac{\text{Mcal}}{\text{km}^2\ \text{sec}} \cdot \text{km}^2 - \frac{\text{tonn}}{\text{sec}} \quad ;$$

Where I is the ice discharge and DI the reduction because of warm groundwater and frictional heating (1.5 to 3 tonn/sec).

Actual measurements of frazil ice have been made possible with the ice discharge gauge constructed by B. Kristinsson (7). Diagrams of measured and calculated ice discharge at Sandafell about 9 km upstream from the Burfell Damsite for Febr. - April 1969 are shown on figs. 3, 4 and 5 together with meteorological and hydrological data. The agreement between measurements and calculations is encouraging but the measurements cover only short periods and the ice production was small because of prolonged cold and consequently reduced open water area. These are the only continuous measurements made up to now. - During the winter 1969 - 70 rough estimates of the ice discharge were continuously made at the Burfell damsite. On the whole the agreement between these estimates and calculated ice discharge is good. The maximum ice discharge was about 10 tons/sec and the greatest monthly production was about $9 \cdot 10^6$ tons. - The period of one day (24 hours) for calculations of ice production seems adequate in Iceland during midwinter, but in March - April the solar radiation is becoming so strong that usually no ice is formed in daytime and 12 h periods are necessary for the calculations.

Under unsteady conditions in midwinter shorter periods than 24 h would even give better results, as the response in ice production to changes in weather is very quick indeed.

Up to now these calculations have mainly been used for hindcasting frazil ice production in connection with power production studies. Next winter (1970-71) regular forecasts based on weather forecasts will be tried at the Burfell power plant to plan the operation of that station and other interconnected power plants.

ACKNOWLEDGEMENTS

The work described in this paper was carried out for Orkustofnun (The National Energy Authority) and Landsvirkjun (The National Power Company) as a part of their ice investigation programs. The author is indebted to many persons at these institutions for their collaboration and to personnel of Rafagnatækni for cooperation on the ice discharge measurements.

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APPENDIX

$$s_1 = (13.18 \cdot 10^{-9} \cdot T_a^4 \cdot (0.46 - 0.06 \cdot \sqrt{e_a}) - G_o \cdot (1-a)) \cdot (1 - 0.012 \cdot N^2) + 13.18 \cdot 10^{-9} \cdot (T_w^4 - T_a^4).$$

$$s_2 = (k + 0.36 \cdot V_6)\,(T_w - T_a).$$

$$s_3 = (1.56 \cdot k + 0.56 \cdot V_6)\,(e_w - e_a).$$

$$k = 0.926 + 0.04 \cdot (T_w - T_a).$$

s_1 = heat loss by radiation, Mcal $km^{-2}\ s^{-1}$,

s_2 = " " " convection, "

s_3 = " " " evaporation, "

T_a = air temperature, degrees Kelvin,

T_w = water " , "

e_a = Wapour pressure of the air, mb,

e_w = " " over water (saturation vapour pressure of air at water temperature), mb,

G_o = global radiation with clear sky, Mcal $km^{-2}\ s^{-1}$,

a = albedo of the water surface,

N = cloud cover, 0-8,

V_6 = wind velocity at 6 m height, m s^{-1},

Mcal = 10^6 cal.

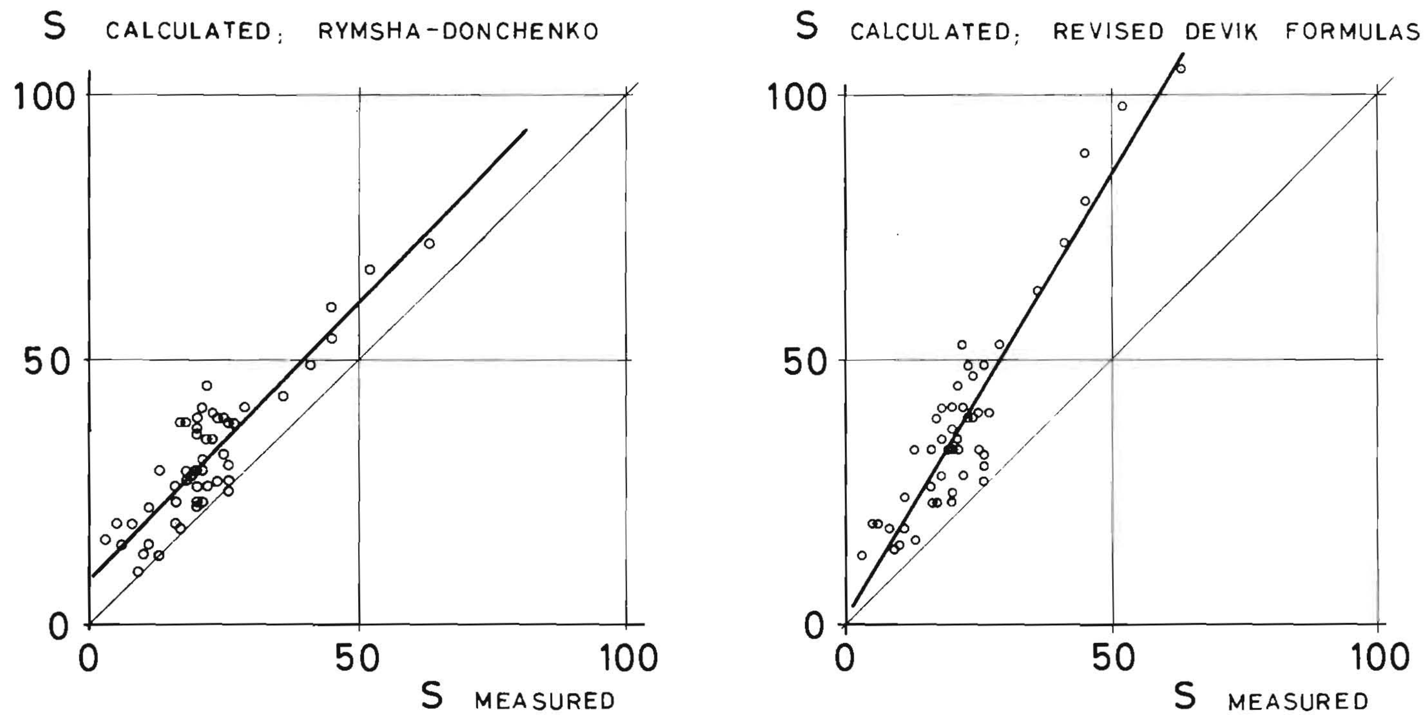
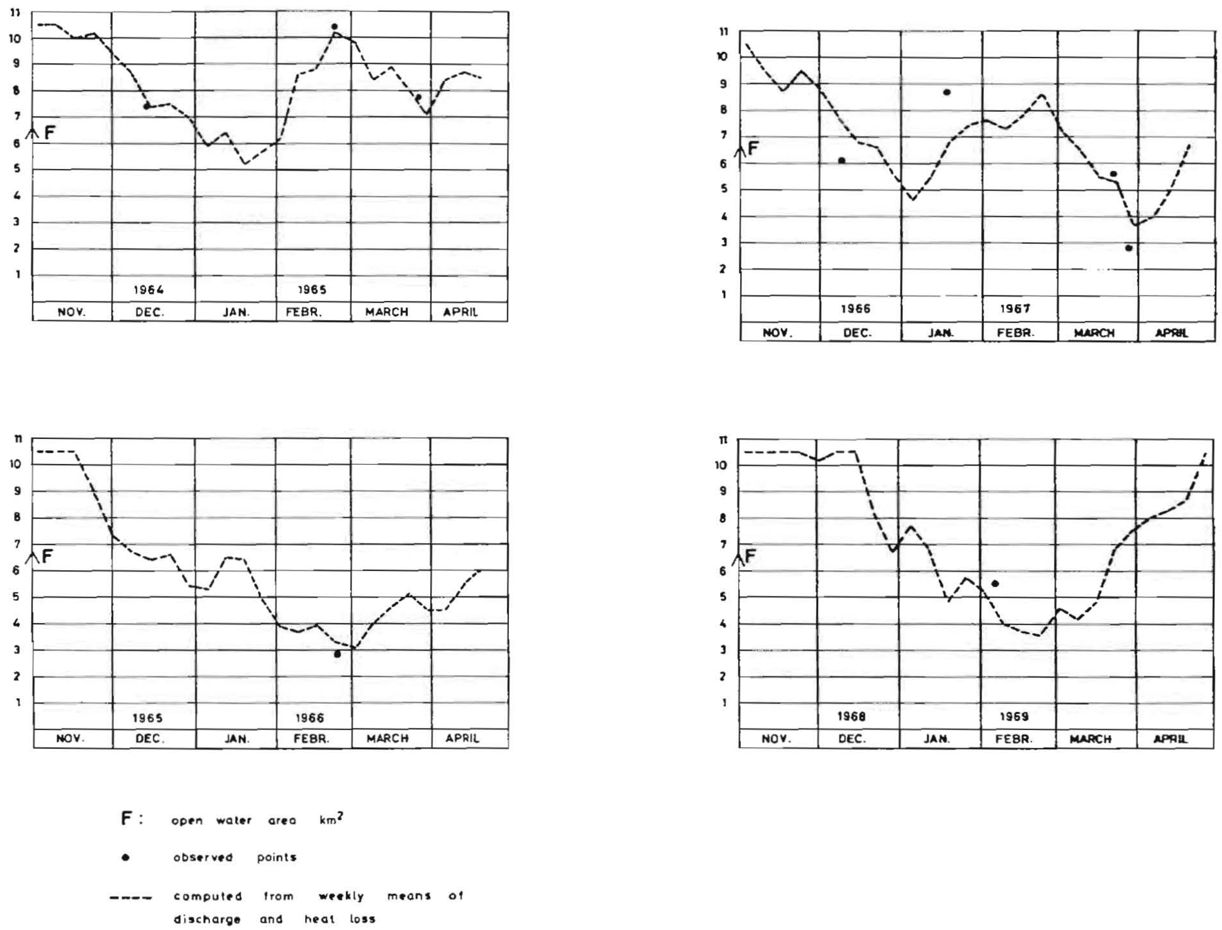


FIG. 1 COMPARISON OF MEASURED AND CALCULATED HEAT LOSS, S Mcal km^{-2} s^{-1}



11
10
9
8
7
6
5
4
3
2
1
F
1964
1965
NOV.
DEC.
JAN.
FEBR.
MARCH
APRIL
1966
1967
1965
1966
1968
1969
F : open water area km²
observed points
computed from weekly means of discharge and heat loss

Fig. 2

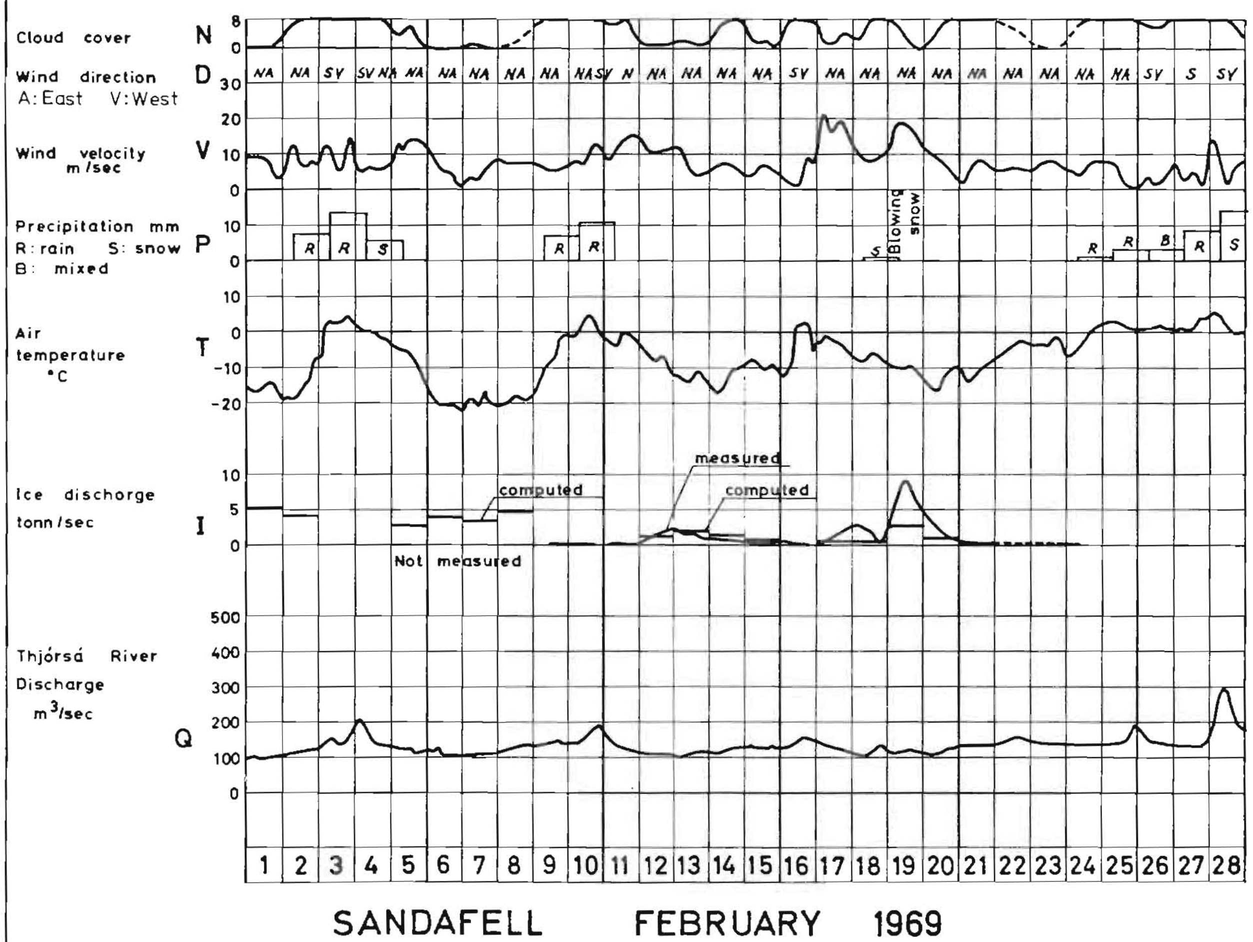


Fig. 3

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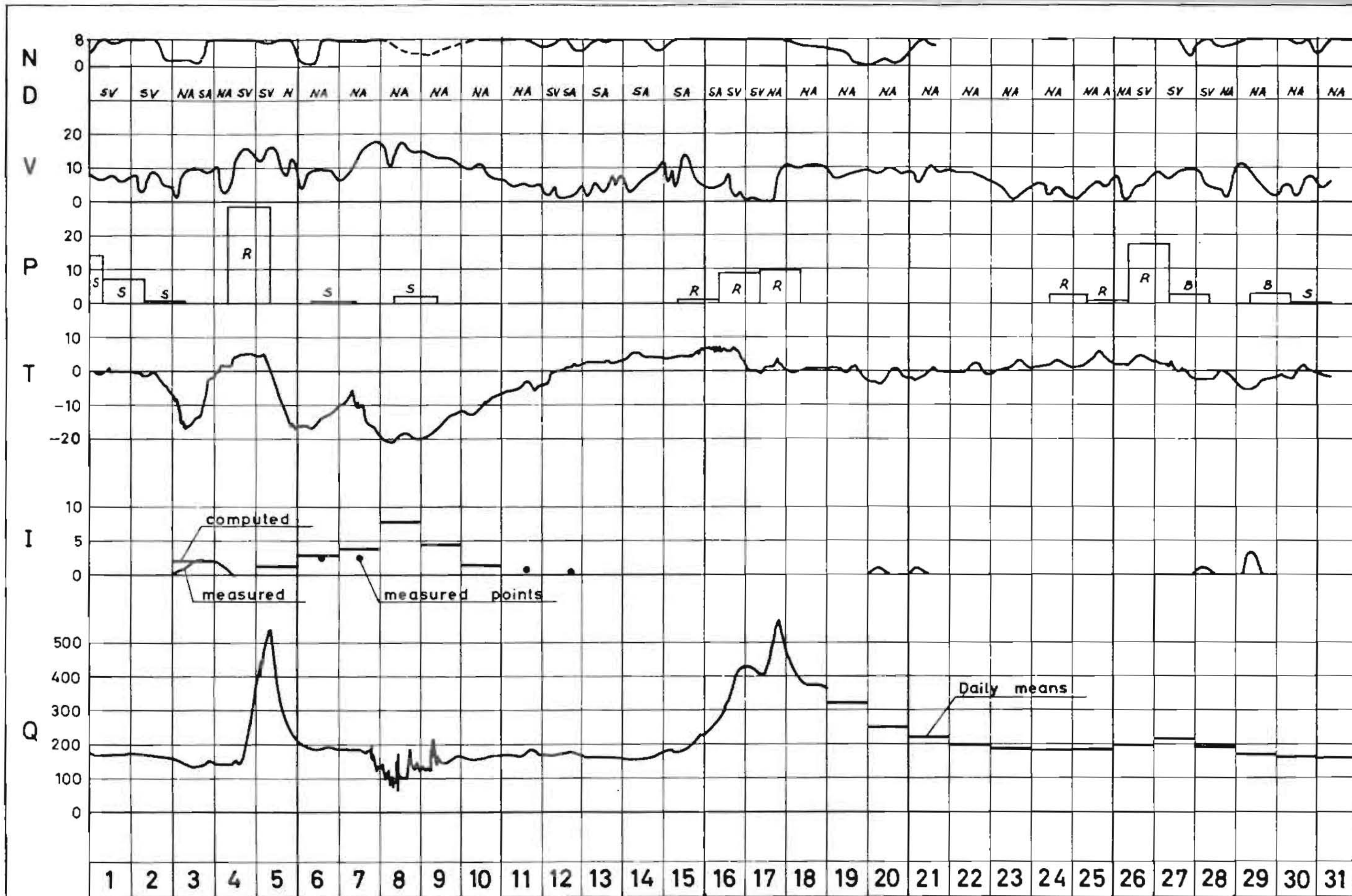

SANDAFELL MARCH 1969

Fig. 4

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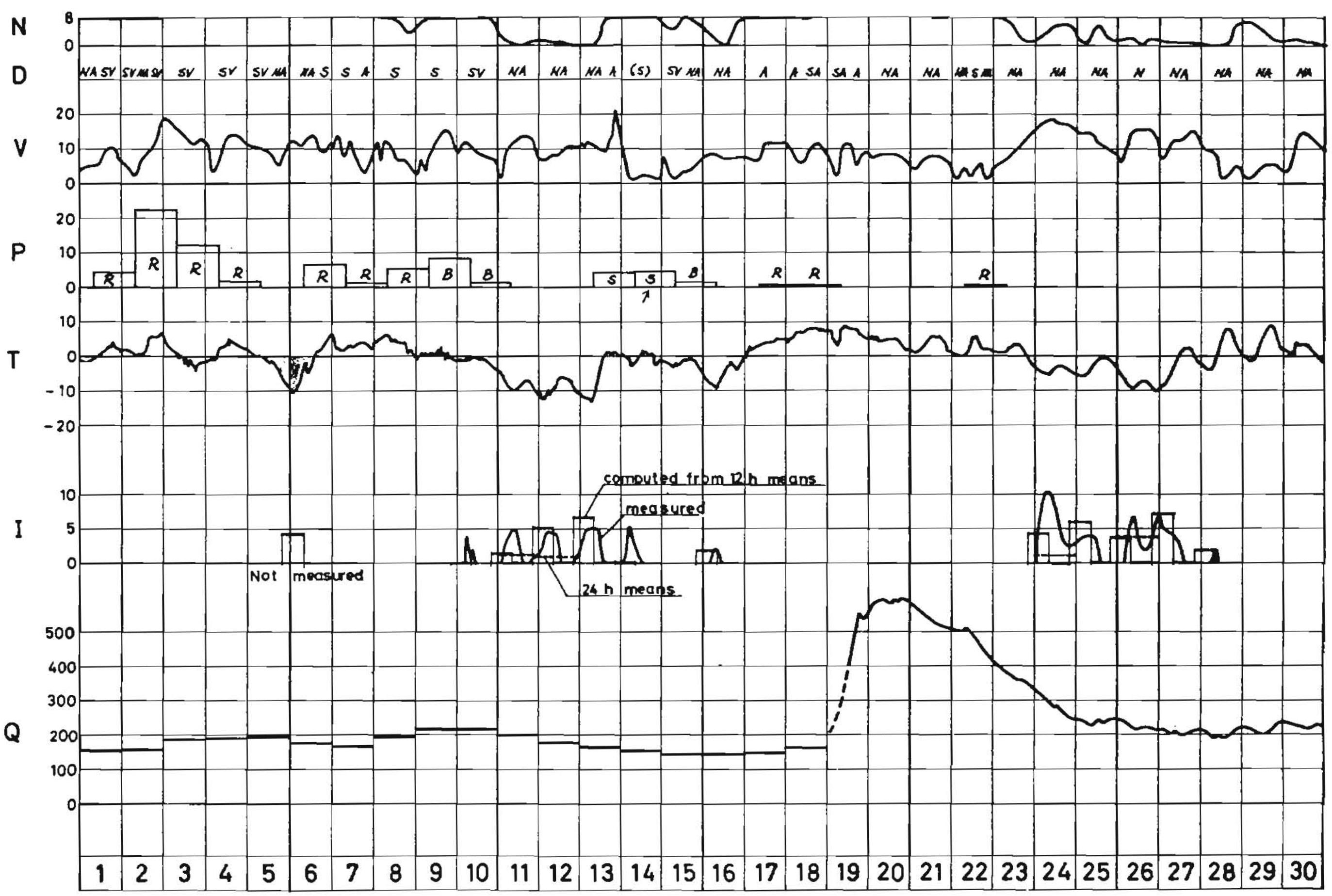

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Fig. 5

08.08.1.03

2.1

ICE SYMPOSIUM 1970
REYKJAVIK

ESTIMATION OF INCIPIENT ICE COVER FORMATION DATE OF RESERVOIRS IN HOKKAIDO BY USE OF A TIME SERIES OF DAILY ACCUMULATED AIR TEMPERATURE

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Japan

SYNOPSIS

As a first step towards understanding the annual variation in date of incipient ice cover formation in relatively large reservoirs in Hokkaido, the author presents an estimation method of annual date above mentioned by use of a discrete time series of daily accumulated air temperature Da and the linearly correlated relation of surface water temperature Tw to Da.

The relation $Tw = \alpha Da + \beta$, was derived in an elementary manner based on the convection boundary condition of heat transfer problems. The constant α was obtained as $0.031 (hr^{-1})$ by using five years data in 1964-1968 observed in two reservoirs in the middle part of Hokkaido. The constant α represents the quantity $Ah/mc\rho$ where h is the heat transfer coefficient.

RÉSUMÉ

Pour commencer à comprendre la variation annuelle de la date de la formation des champs de glace dans les réservoirs relativement larges en Hokkaido, Japon, l'auteur présente une méthode pour l'estimation de la date mentionée ci-dessus, employant la série du temps de la température atmosphérique accumulée dans un jour Da et la relation linéaire de la température superficielle de l'eau Tw à Da.

La relation $Tw = \alpha Da + \beta$ est dérivée de la manière élémentaire qui se base sur la condition à la frontière pour la convection dans le problème du transfert de la chaleur. La valeur de la constante α obtenue par l'observation dans les deux réservoirs au milieux de Hokkaido, de 1964 à 1968, est $0.031 (hr^{-1})$ et représente la quantité $Ah/mc\rho$ où h est la coefficient du transfert de la chaleur.

INTRODUCTION

Hokkaido is the northern-most island of Japan and the northern limiting line for unfreezing lakes passes through the island as shown in Fig. 1. The western part of the island is relatively low land.

Then, most reservoirs created by multipurpose dams have ice covers in every winter (see Fig. 6) and have rather large annual variations in incipient ice cover formation date as an example of the Katsurazawa reservoir shown in Table 1. As the storage water level of most reservoirs in Hokkaido is operated to fall gradually in this season of water shortage, the understanding of annual variation in incipient ice cover formation is needed for designing dams and utilizing reservoirs. However, fully long-period data for the incipient ice cover formation are not available in most cases at present.

As a first step towards statistic studies in this field, the author derived an estimation method of the date in each winter by use of temperature data of the air which are available in most sites for sufficiently long periods. In this study, observed data on the date of ice cover formation and surface air and water temperatures in the Katsurazawa and Kanayama reservoirs (see Fig. 1) were very helpful.

MECHANISM OF ICE COVER FORMATION [1)]

After the reservoir water reaches an unstable isothermal condition a few degrees below the 4°C temperature (about at the end of November in these two reservoirs), the ice cover formation of a reservoir begins on a cold day or night, freezing begins in the surface. And there are two factors in the formation of ice cover. One is the freezing of the upper water layer itself and this produces a smooth homogeneous sheet ice. The other is the fusion of individual ice masses produced by the breakup and refreezing of an ice sheet in its early stage of development or by the snow in the surface water. And in this conditions agglomeritic ice which is rough on the surface and nontransparent is formed.

In the Katsurzawa reservoir, the surface ice formation begins from near the shore, then the complete ice cover surface is formed from seven to ten days later without any ice-free surface. The term incipient ice cover formation in this study means the date when the entire surface is covered with ice sheet or agglomeritic ice. And the last stage of ice cover formation is begun near the center of reservoir. Now, a very complicated mechanism of cover formation by the combination of two factors is considered, however, in this study only an elementary heat transfer model of sheet ice which is produced by rapid freezing of the surface-water film is adopted.

HEAT BUDGET [1)]

Analytical heat budget treats the rates of heat transfer of the several forms of radiant and thermal energy. The equation for the storage of heat Q_t in the reservoir for practical use becomes

$$Q_t = Q_B \pm Q_s \pm Q_i - Q_E \tag{1}$$

Where Q_B is the net radiation surplus and Q_s is conduction of heat from the air when it is warmer than the water and transfer of heat from water to air, and Q_i is heat carried in by influent water, and Q_E is energy used in the evaporation process. The solution of an analytical heat budget requires data on the radiation flux, sufficiently detailed temperature series within the reservoir and in the air above the reservoir, and temperature of inflows and outflows in flow-through is significant. However, only heat budget near the reservoir surface is considered in this study and negligible terms are entirely neglected for simplicity. Then Eq. (1) becomes

$$Q_t = Q_s \tag{2}$$

HEAT TRANSFER FORMULATION [2)]

Heat energy is transfered between the air over the reservoir surface and the surface water which are at different temperatures T_a and T_w, respectively. In this case convection are considered as a mode of heat transfer and the convection boundary condition is used for this study. The heat flux across the reservoir surface may be taken as proportional to the difference between the surface water temperature T_w and the surface air temperature T_a. Eq. (2) then takes the form

$$\dot{Q}_s = hA\ (T_a - T_w) \tag{3}$$

where h is termed the (surface) heat transfer coefficient $[\mathrm{Kcal/m^2hr\,^\circ C}]$.

On the other hand, when the thin layer portion of water (depth: d) near the surface of reservoir is considered and the temperature of this water mass decreases from $(T_w)_1$ to $(T_w)_2$ after time t(hr), the rate of change of heat energy is to be

$$\dot{Q}_s = mc\rho \frac{dT_w}{dt} \tag{4}$$

where m = Ad is mass $[\mathrm{m^3}]$, C is the specific heat of water: 1 $[\mathrm{Kal/Kg\,^\circ C}]$, ρ is the density of the water: 1000 $[\mathrm{Kg/m^3}]$ and t is time [hr]. From Eq. (3) and Eq. (4)

$$mc\rho \frac{dT_w}{dt} = Ah\ (T_a - T_w)$$

By integrating

$$mc\rho \left[T_w(t)\right]_0^{24} = hA \int_0^{24} T_a(t)dt - hA \int_0^{24} T_w(t)dt \tag{5}$$

Then $\quad T_{w_{24}} = \alpha D_a + \beta$

where $\quad \alpha = hA/mc\rho \quad , \quad D_a = \int_0^{24} T_a(t)dt,$

$$\beta = \left[T_w(t)\right]_{t=0} - \alpha \int_0^{24} T_w(t)dt \tag{6}$$

DETERMINATION OF α AND β BY OBSERVED DATA

To obtain the values of α and β in Eq. (6), records of surface water temperature (at the depth of 0.1 m) and water temperature of inflow and a discrete time series of daily accumulated surface air temperature (at the Dam Control Office) were studied in each year (1964-1968). One example of them is shown in Fig. 2. The daily accumulated temperature is an algebraic sum of hourly temperatures in degrees obtained from self-recording continuous series of the surface air temperature. Two results of the crosscorrelation analysis of $D_a(t)$ and $T_w(t)$ are shown in Fig. 3, representing that a couple of time series without shifting has better correlation.

The results of computing α and β in Eq. (6) by using data above-mentioned were shown in Table 2 with sufficiently satisfactory correlation coefficients. And α was found to be almost a constant value in these two reservoirs and in each year, including the similar h. β was also found to have almost a constant value for each year in the same reservoir. However, β has a little different values for two reservoirs, since β depends on decreasing features of the surface water temperature and so on. The relation of T_w to D_a in Eq. (6) in 1964 is shown in Fig. 4 as an example. Data from July 1 to the incipient date of ice cover formation are used. Thus, the linearity of Eq. (6) obtained by observed data gave practical values to this simplest heat transfer model Eq. (6).

D_a AND T_1 REQUIRED FOR ICE COVER FORMATION

Considering the transfer of heat energy required to change the surface water of T_1°C of the depth d(0.1 m, for example) and unit area, to the water of 0°C of the same volume and to convert a part of the water (d_i in depth) at the freezing temperature into ice at the same temperature, the following equation is derived. In this case 79.67 Kcal/Kg of latent heat of conversion must be deducted in Eq. (6)

$$-mc\rho\, T_1 - 80\, m'\rho = hA(D_a - \int_0^{24} T_w dt) \tag{7}$$

where $m = 1 \times 1 \times d\ [m^3]$: mass of water,

$m' = 1 \times 1 \times d_i\ [m^3]$: mass of water to be converted to ice,

$c = 1\ [Kcal/Kg°C]$: specific heat,

$\rho = 1000\ [Kg/m^3]$: density,

$$T_w = -\frac{T_1}{24} t + T_1$$

Hence $$D_w = \int_0^{24} T_w(t)dt = \left[-\frac{T_1}{48}t^2 + T_1 t\right]_0^{24} = 12\, T_1$$

Then by Eq. (7)

$$D_a = -\frac{mc\rho}{h} T_1 + 12T_1 - \frac{80m'\rho}{h} = -\left(\frac{1000d}{h} - 12\right)T_1 - \frac{80d_i\rho}{h} \tag{8}$$

From Table 2 and Eq. (6)

$$\alpha = \frac{Ah}{mc\rho} = 0.031$$

Hence $$h = 0.031\frac{mc\rho}{A}$$

In this case $A = 1\ [m^2]$, $m = Ad = d\ [m^3]$, $\rho = 1000\ [Kg/m^3]$

Hence $$h = 31d \tag{9}$$

For example $h = 3.1$ [$Kcal/m^2\ hr\,^\circ C$] for $d = 0.1$ [m]

From Eq. (8) and Eq. (9)

$$D_a \doteqdot -20\ T_1 - 2580\frac{di}{d} \tag{10}$$

This relation of D_a and T_1 is shown in Fig. 5 with five plots of incipient ice cover formation data in the Katsurazawa reservoir (1964-1968). $T_1 - D_a$ diagram in Fig. 5 indicates mutual conditions required for a complete ice cover to be formed on the date. T_w of the date when ice cover formation is expected if only D_a becomes lower than some standard value, namely, T_1 may also be estimated by Fig. 5. D_m for T_1, is obtained as D_a for T_w in Fig. 4.

The thickness of ice formation d_i may be estimated as 0.003-0.009 [m] for d = 0.1 [m] in the region which covers five observed data. As this thickness means the last part to be closed with ice cover, the thickness of ice cover in most parts of the surface is to be much more.

PROCEDURE OF ESTIMATION

An estimation of incipient ice cover formation in the Katsurazawa reservoir in 1964 is shown for example (see Fig. 2). First, an average decreasing curve for D_a series is drawn by use of moving average of ten days or by a free hand method in order to know D_m value. And D_m becomes lower than -100 [°C hr] on Dec. 7 ($T_1 = 3$°C). Daily D_a after the date in its series is checked and D_a becomes lower then -140 [°C hr] on Dec. 9. And this date is estimated as the incipient formation date. This result is coincided with the observed one. The results of estimation for five years are relatively good with errors within four days.

ACKNOWLEDGMENTS

The author is grateful to Mr. M. Fujita and Mr. K. Hoshi for their help in various phases of the study and he is also grateful to Mr. K. Nakazawa and engineers of the Katsurazawa Dam Control Office, Hokkaido Development Bureau, for their continuous observations and surveys on ice problems which made this study possible.

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1) J. H. Zumberge, and J. C. Ayers: Hydrology of lakes and swamps, section 23 of the Handbook of applied hydrology, edited by V. T. Chow, Mc-Graw Hill Book Company, 1964.

2) J. G. Knudsen and D. L. Katz: Fluid Dynamics and Heat Transfer, Mc-Graw Hill Book Co., 1958.

Table 1 Observed date on ice cover in two reservoirs (After the Katsurazawa Dam Control Office, Hokkaido Development Bureau)

Name	Observed date on ice cover			Significant data for reservoirs
	Year	Formation, no ice-free surface	Breakup, no ice cover	
Katsura-zawa	1958 -1959	Dec. 28	Apr. 8	Drainage area: 299 sq. Km Surface area: 4.99 sq. Km Total capacity: 92.7 million m^3 Planned inflow: (Dec. 1-5) 5.75 m^3/sec Max. water level: 187 m Dam height: 63.6 m (concrete gravity) Dam completed in 1957
	1959 -1960	Dec. 18	Apr. 26	
	1960 -1961	Dec. 18	Apr. 24	
	1961 -1962	Jan. 4	Apr. 16	
	1962 -1963	Dec. 27	Apr. 14	
	1963 -1964	Dec. 28	Apr. 15	Drainage area: 470 sq. Km Surface area: 9.2 sq. Km Total capacity: 150.45 million m^3 Planned inflow: 8.72 m^3/sec (Dec. 1-5) Max. water level: 345 m Dam height: 59.7 m (Concrete hollow gravity) Dam completed in 1967
	1964 -1965	Dec. 9	May 8	
	1965 -1966	Dec. 17	Apr. 26	
	1966 -1967	Dec. 13	Apr. 23	
	1967 -1968	Dec. 13	Apr. 11	
	1968 -1969	Dec. 31	Apr. 24	
	1969 -1970	Dec. 16	May 4	
Kanaya-ma	1968 -1969	Dec. 31		

Table 2

Name	$Tw_{24} = \alpha D_a + \beta$			Eq.(6)
	Used data year	α	β	Correlation coefficient
Katsurazawa reservoir	1964	0.032	5.702	0.970
	1965	0.031	5.874	0.957
	1966	0.029	6.640	0.964
	1967	0.031	6.220	0.967
	1968	0.032	6.140	0.951
	Including 1964-1968	0.031	6.101	0.962
Kanayama reservoir	1968	0.031	7.079	0.938

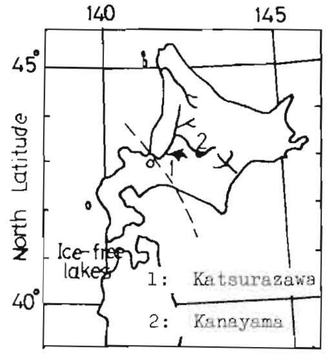

Fig. 1 Map of Hokkaido, Japan showing the observed reservoirs and northern limiting line for non-freezing lakes (After Yoshimura, 1933).

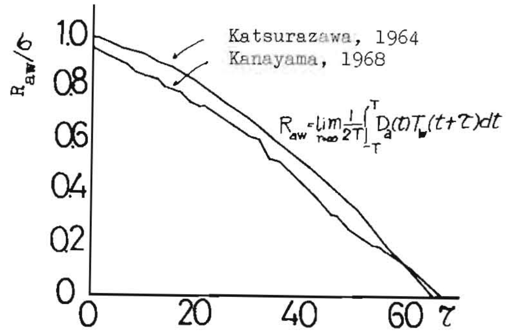

Fig. 3 Cross-correlation of T_w and D_a.

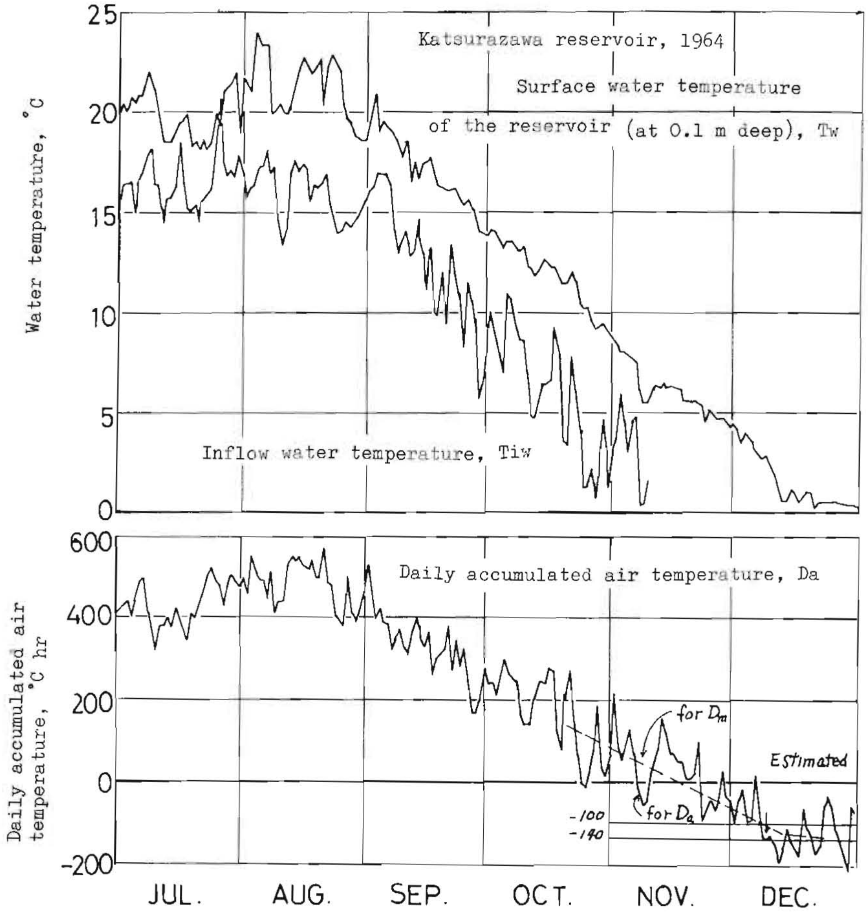

Fig. 2 Water temperature records and a time series of daily accumulated air temperature in 1964 as an example.

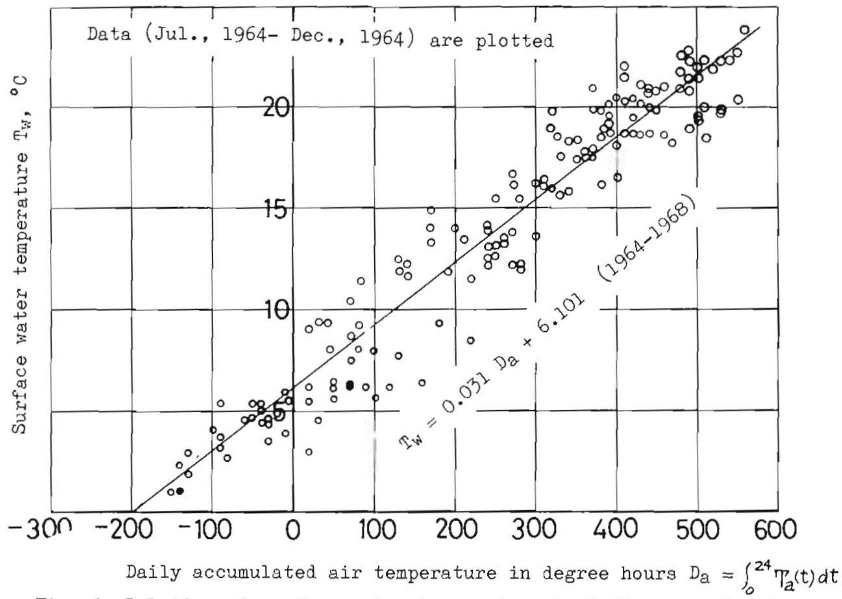

Fig. 4 Relation of surface water temperature to daily accumulated air temperature .

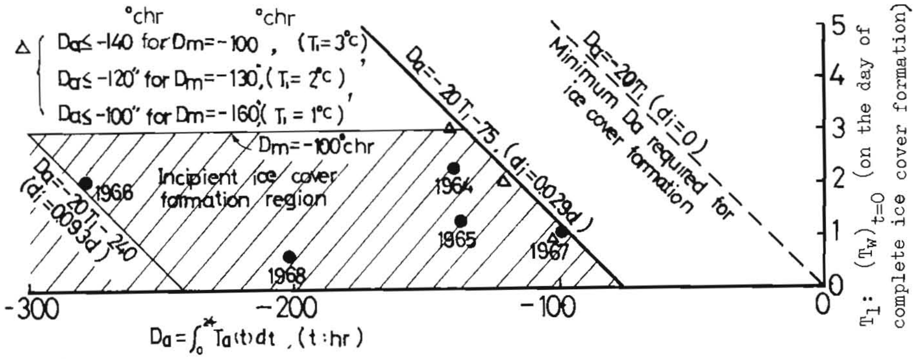

Fig. 5 T_1 - D_a diagram for incipient ice cover formation showing Katsurazawa data (1964-1968).

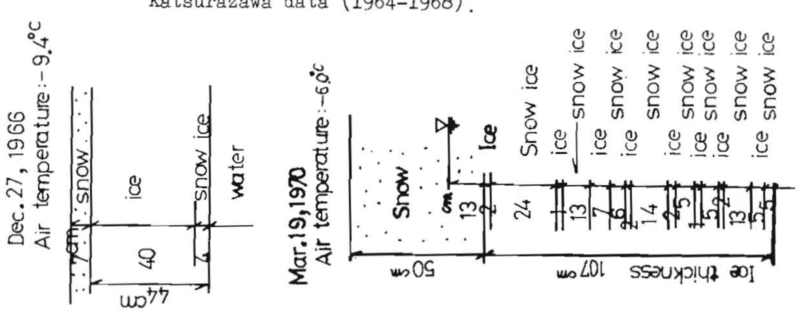

Fig. 6 Developed ice cover profile as examples

On paper by Yamaoka, I. :

DISCUSSION by Tsang, G.

I would like to see some physical argument supporting Prof. Yamaoka's assumption of d, the depth of the affected water layer, equal to 0.1(m). Otherwise the physical and mathematical refinement in the first half of his paper may be tarnished.

DISCUSSION by Yamaoka, I.

I thank Mr. Tsang for his discussion on my paper. My assumption of d to be 0.1(m) as an example is a trial (on the convenience of my having long-term data observed at the depth of 0.1(m) in this case) taking into account that d should be fairly small as I adopt an elementary heat transfer model of ice sheet produced by rapid freezing of the surface-water film.

In order to obtain a further knowledge for assuming appropriate d, some typical water-temperature gradients for surface-temperature variation from T_1 °c to 0 °c were studied by computing the solution of the thermal diffusion equation (or Fourier heat conduction equation) with κ (thermal diffusivity) of 0.000472 (m^2/hr). The convection flow is neglected in this equation. Computed gradients for various values of T_1 show almost linear variations in temperature from the water-surface (0 °c) to depths of d(T_1 °c at the depth lower than d) for the values of d which are less than or approximately equal to 0.1, 0.2, 0.4 and 0.5(m) for T_1 of 0.6, 2, 5, and 10 °c, respectively. In my data, initial water temperatures (at the depth of 0.1(m)) on the beginning date of freeze-up T_1 are scattered between 0.6 and 2 °c as shown in Fig.5. Hence, d should be less than or equal to 0.1–0.2(m). Thus, the assumption d = 0.1(m) may unobjectionably be supported for practical problems. (In my paper the transfer of heat energy required to change the surface water of T_1 °c of the depth d and unit area to the water of 0 °c of the same volume is considered schematically. And the heat energy transferable to the air may be considered the same even in the practical case described in this comments as the surface-water temperature (in Fig.2) has daily periodical variation effected by the air temperature and T_1 may be assumed as the mean value of $2T_1$ and 0 °c for T_1 of low temperature such as 0.6–2 °c.

Namely, $-\frac{1}{2} 2T_1 dAc\rho = -mc\rho T_1$)

ICE SYMPOSIUM 1970
REYKJAVIK

TEMPERATURE GRADIENTS IN A LAKE ICE COVER

Samuel S. Lazier Professor, Queen's University Kingston, Canada.
Michel Metge Research Assistant, Queen's University Kingston, Canada

SYNOPSIS

In order to acquire a better knowledge of static ice forces, the authors developed a finite difference method for calculating temperature gradients in an ice cover. This method was checked against Taylor's (1945) analytical solution and it proved to be precise and simple.

Measurements of ice temperatures were carried out during the winter 1969 in Kingston harbour. These experiments showed that the gradients predicted with given ice surface temperatures were fairly close to those measured, and also that there is a fundamental difference between the changes in temperature gradients due to solar radiation and those due to changes in air temperature only.

SYNOPSIS

Dans le but d'obtenir une meilleure idée des forces exercées par un champ de glace statique, les auteurs ont développé une méthode de calcul par différences finies pour evaluer les gradients de température dans un champ de glace. Cette méthode a été comparée á la solution analytique de Taylor (1945) et s'est révélée simple et précise.

La température de la glace fut mesuree pendant l'hiver 1969 dans le port de Kingston. Ces mesures prouvent que les gradients calculés pour des températures données de la surface de la glace sont assez proches des gradients mesurés, et aussi qu'il une différence fondamentale entre les variations de gradient de température causées par les radiations solaires et celles causées uniquement par une variation de la température de l'air.

INTRODUCTION

The changes which occur in the temperature gradient within an ice cover are of great importance when dealing with the problem of static ice forces on structures.

When the ice warms up, it expands and the ice cover may exert forces at its boundaries. The amount of expansion is directly proportional to the temperature change, but since ice is a visco-elastic material the resulting force is related to the rate of change of the temperature gradient in the ice cover.

When the ice temperature falls sharply the contraction which results from this action may cause cracks to form. These may fill with water which ultimately freezes, this explains the lateral growth of the ice cover and the forces exerted by it. Several repetitions of this procedure may result in the failure of the ice sheet and the formation of "pressure ridges".

Several years ago the senior author undertook a research project designed to resolve the problem of static ice forces on structures, using the ice sheet which forms annually in Kingston Harbour as a full scale laboratory. This paper is a report on some of the initial work done on this project and is concerned primarily with temperature gradients in the ice cover. The study of these gradients included two steps:

- i) The recording of the actual temperatures at several depths in an ice sheet, and
- ii) the development of a method of computation in order to simulate these temperature gradients, using ordinary meteorological data.

METHOD OF COMPUTATION

In the past, two methods of computing temperature gradients in an ice sheet were proposed.

- Rose (1947) used a graphical method based on finite differences. This method has a limited accuracy, and cannot take into account the thermal boundary layer or emissivity (sic) effects.

- Taylor (1945) developed an analytical solution which treated the problem of the thermal boundary layer effects and took into account solar radiation but only by assuming a rise in air temperature which was linear with respect to time, and an initial temperature gradient of nil throughout the ice sheet.

Both of these methods were developed before the advent of digital computers and, as a result, could not deal with complex boundary conditions. The finite difference method,developed by the junior author,is similar to Rose's method, but the use of a digital computer permits small increments of depth and time to be handled conveniently. As well, the thermal boundary layer effect, ice growth and the change in the properties of ice with time and depth can be accommodated.

The computer program leads to the solution of the diffusion equation (1), by employing an explicit finite difference method.

$$\frac{\partial T}{\partial t} = h^2 \frac{\partial^2 T}{\partial x^2} \qquad (1)$$

T = temperature

x = distance from the bottom

h^2= diffusivity of the ice

That is, at time t the curvature of the temperature gradient $\frac{\partial^2 T}{\partial x^2}$ is calculated by a central difference formula, then the rate of change of temperature is known ($\frac{\partial T}{\partial t}$) and the temperature at time $t + \Delta t$ can be obtained by extrapolation.

Precision of the Method

The precision of the method was analysed as follows:

Firstly the precision of Taylor's analytical solution was checked, and it was found (by two different methods) that for average values of the characteristics of the ice sheet* the following precisions could be obtained.

Number of Terms taken into account in Taylor's series solution	Precision Obtained
5 terms	± .4°C
7	± .1°C
15	± .01°C.

Secondly, the finite difference program was applied, using Taylor's hypotheses, and compared to Taylor's solution. The results of both methods were very close: using a time step of 90 seconds and a depth increment of 2 cm. the finite difference method did not differ from Taylor's by more than ± .05°C. As Taylor's solution (taking into account 15 terms) may be assumed to be exact, it was concluded that the error due to calculations in the finite difference method did not exceed ± .05°C, which is an adequate precision for engineering purposes.

EXPERIMENTAL PROGRAM

During the winter of 1969 some preliminary measurements were made on the ice sheet in Kingston harbour at a site some 400 feet offshore. The apparatus consisted of a set of nineteen thermocouples buried in the ice at various depths. The output of the thermocouples was read by a digital voltmeter, the reference junction being at the underside of the ice cover. Figure 1 shows, schematically, this test set up.

Preliminary Results

Figures 2 and 3 show some typical results from the 1969 field study. Solar radiation was negligible during the time that the results shown in Figure 2 were taken, while such was not the case for the data shown in Figure 3.

---

* depth = 60 cm.

heat transfer coefficient = .0007 Cal/cm °C sec

Rate of air temperature rise = .002°C/sec

The temperatures in figure 3 are somewhat suspect because the thermocouples absorb radiation and may not be at the same temperature as the ice.

One of the weaknesses of the data obtained in 1969 was that the air temperature was not obtained at the test site. This was observed some distance away on the shore (due to vandalism at the site) and was not completely representative of the site conditions. Since neither the air temperature nor the surface temperature of the ice was known, a comparison between the measured gradient and that calculated by the finite difference method could only be made starting at one inch below the ice surface, where the first thermocouple was located. It is seen in Figure 4 that the measured and calculated gradients are of the same form.

Even though this kind of simple check could be expected to be conclusive it still shows promise. Further experiments were performed during the winter 1970 (figure 5 shows the field station) which may help to correlate the ice surface temperature to meteorological data.

DISCUSSION

This study tends to prove that the computation of temperature gradients in an ice cover can be simple and accurate.

Two important problems remain to be solved.

(1) how to relate ice surface temperature to air temperature?

(2) what is the influence of solar radiation?

If these can be solved it would be easy, knowing the air temperature, solar radiation, wind speed and other pertinent factors, to calculate static ice forces with more accuracy than is possible now.

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Rose, 1947 "Thrust Exerted by an Expanding Ice Sheet", ASCE Trans. 1947

Taylor, 1945 "Temperature Change in an Ice Sheet", Internal publication Col. University, Denver, Col.

FIGURE I: DISPOSITION OF THE INSTRUMENTS

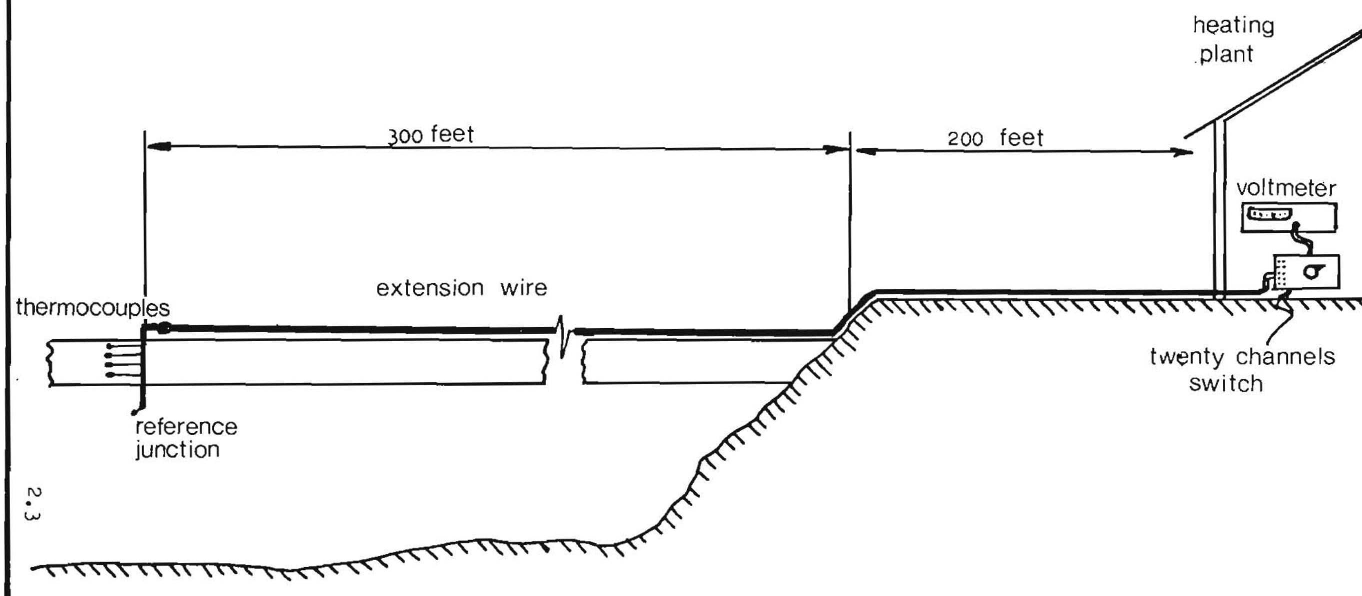
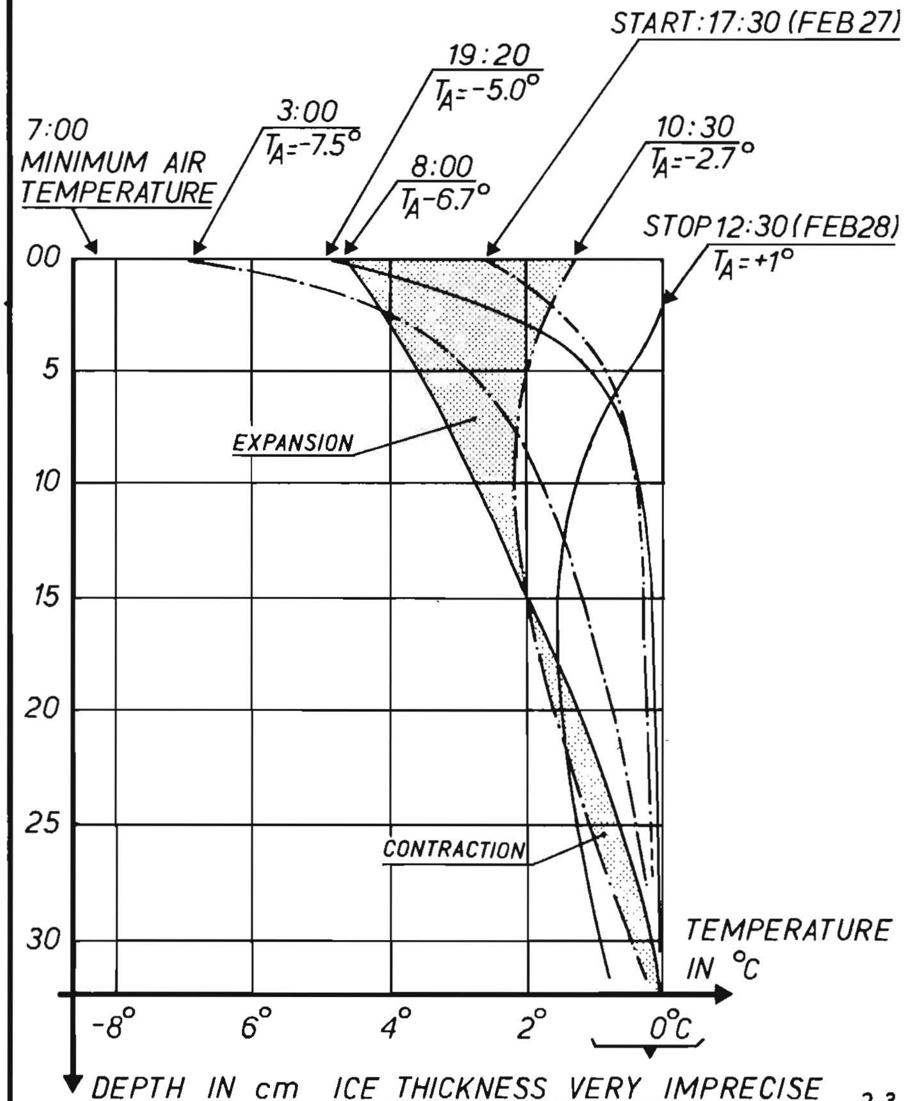
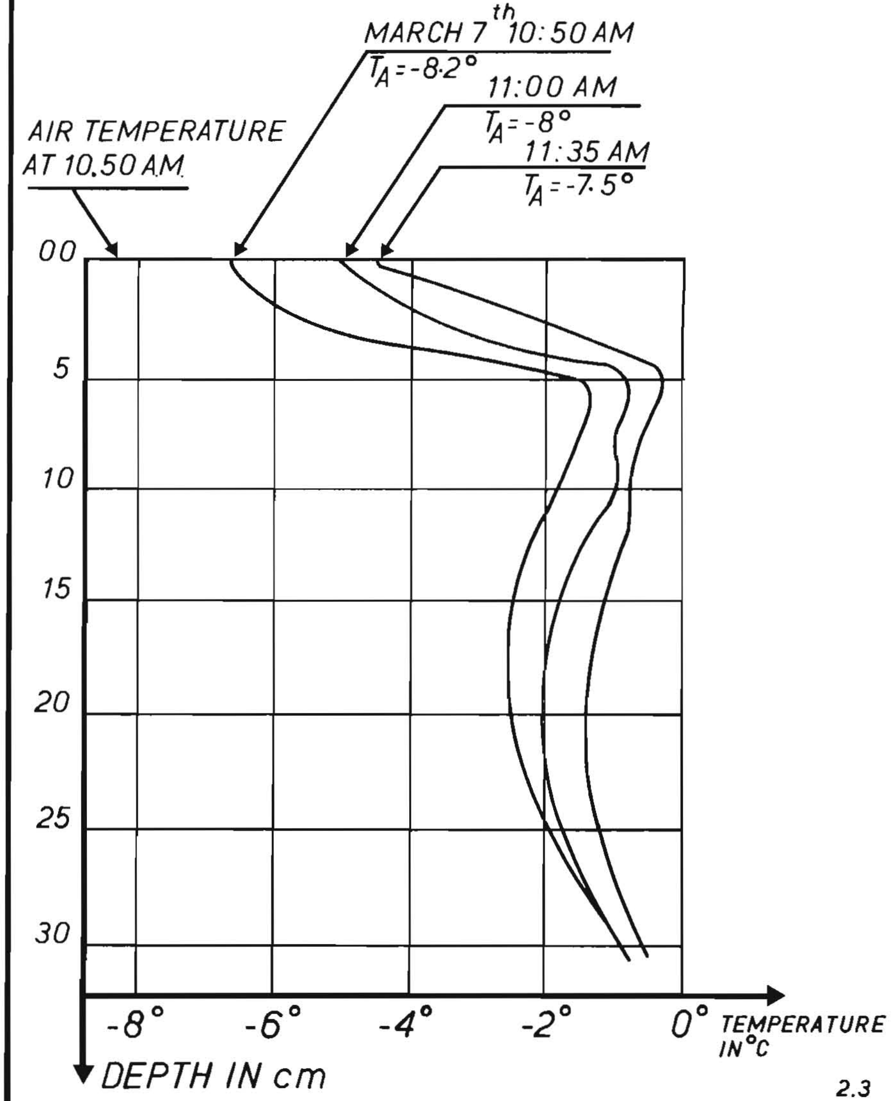


FIGURE 2 : CHANGES IN TEMPERATURE GRADIENTS
EXPERIMENTAL RESULTS



2.3

FIGURE 3 : CHANGES IN TEMPERATURE GRADIENTS
EFFECT OF SOLAR RADIATION (EXPERIMENTAL RESULTS)



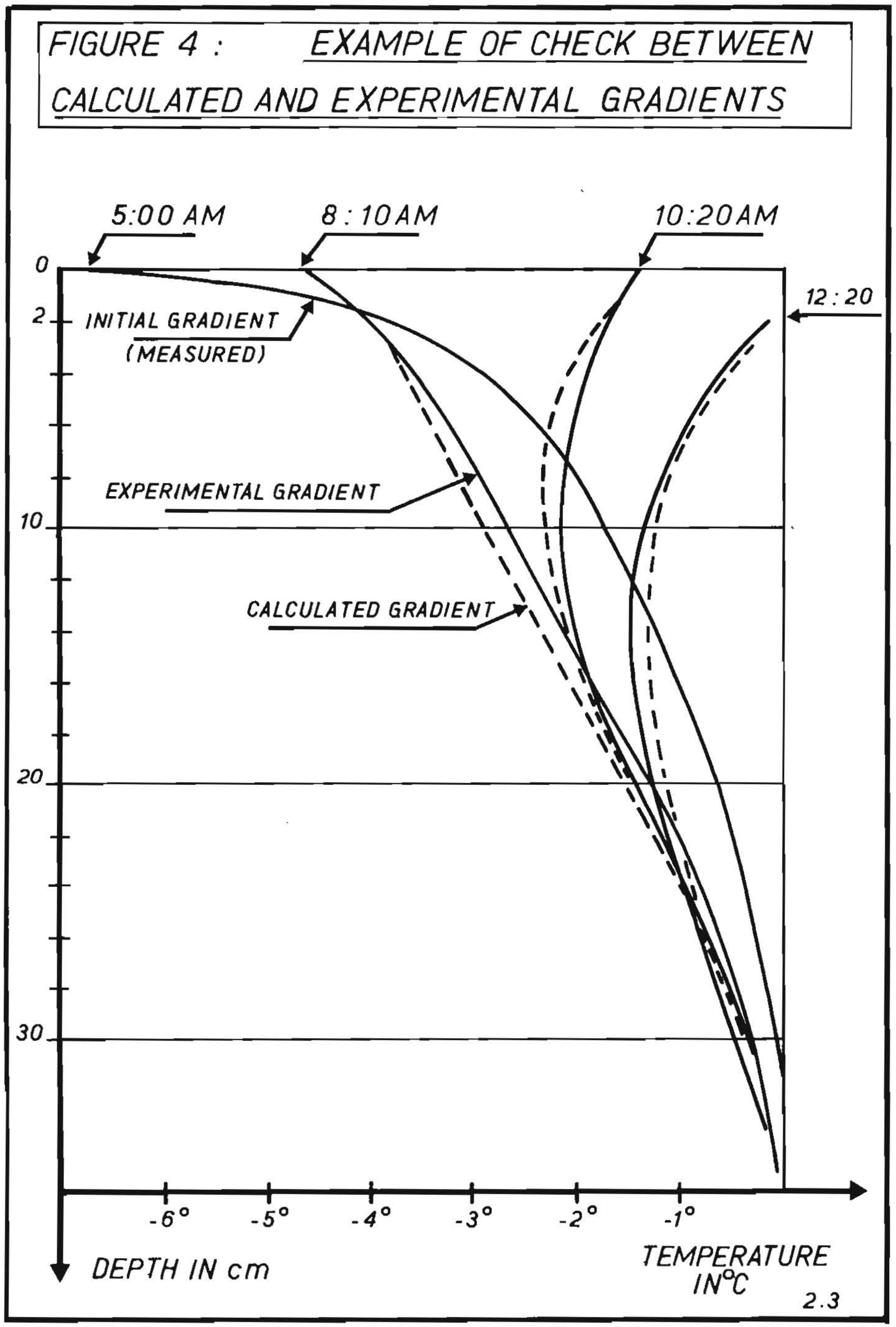


FIGURE 4 : EXAMPLE OF CHECK BETWEEN CALCULATED AND EXPERIMENTAL GRADIENTS
5:00 AM
8 : 10AM
10:20AM
12 : 20
INITIAL GRADIENT (MEASURED)
EXPERIMENTAL GRADIENT
CALCULATED GRADIENT
0
2
10
20
30
-6°
-5°
-4°
-3°
-2°
-1°
DEPTH IN cm
TEMPERATURE IN°C
2.3

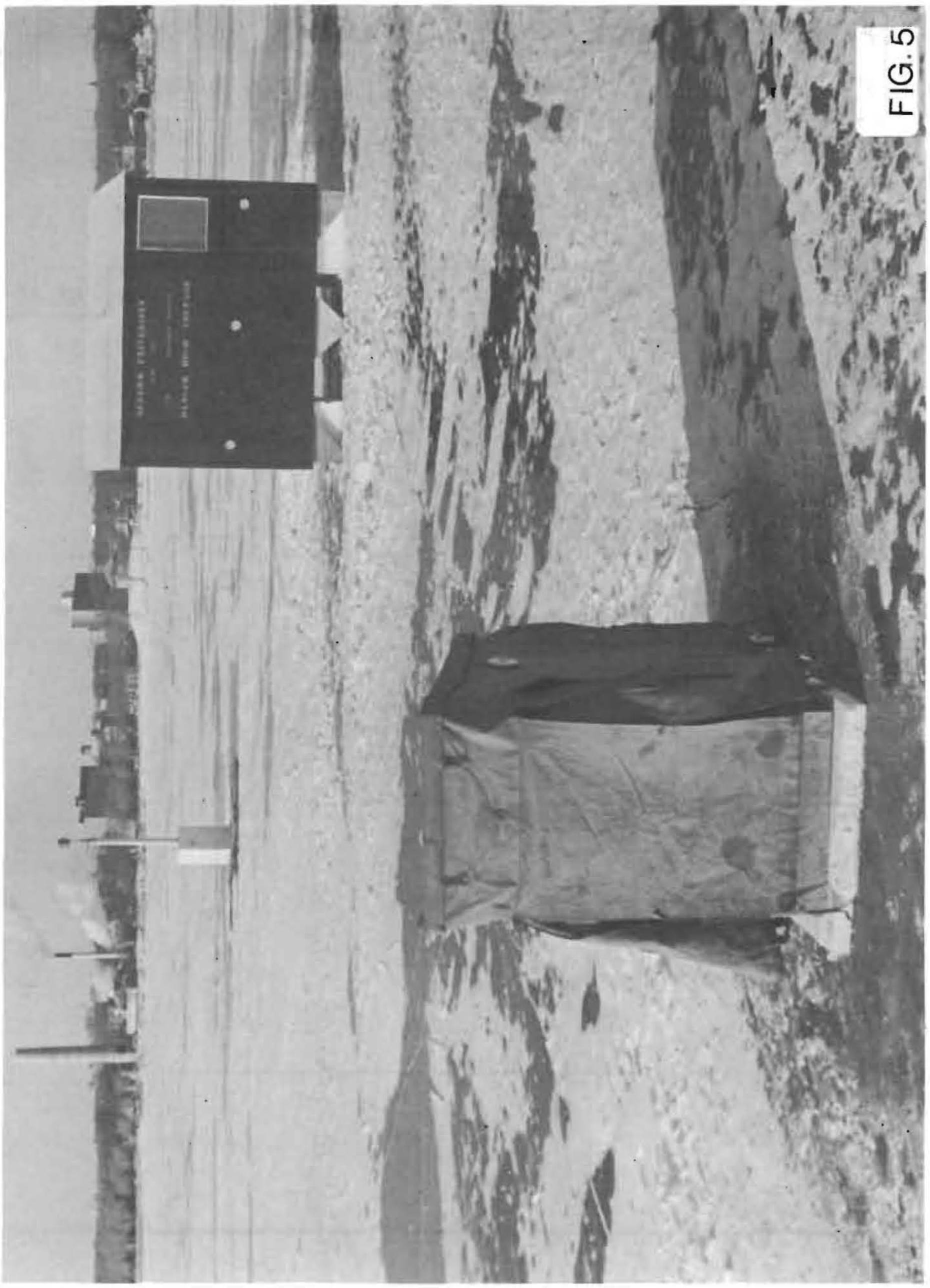


FIG. 5

2.3

DISCUSSION by S. HANAGUD

on paper by S. S. LAZIER, M. METGE

Q. I would like to know if the computer calculations included the effect of radiation?

Was there any melting of the surface of the ice observed or recorded in any of your measurements?

A. The computer calculations did not include solar radiation and a comparison between calculated and experimental gradients could only be made when solar radiation was negligible (at night or under heavy cloud cover).

Sometimes surface melting was observed, but at these times the temperature of the ice was very close to $0^{o}C$ at all levels in the ice cover.

INTERNATIONAL ASSOCIATION FOR HYDRAULIC RESEARCH
ICE SYMPOSIUM REYKJAVIK 8-10 SEPT. 1970

DISCUSSION by H. R. CROASDALE

on paper by S. S. LAZIER , M. METGE

Q. What is the snow cover relevant to the results shown in Figure 2?

Presumably the calculation technique can take into account the snow cover?

A. The snow cover is of great importance as far as temperature gradients are concerned.
For these particular experiments, the calculation technique did not take it into account because the surface temperature was unknown and the temperature that we used in order to calculate the rest of the gradient, was the temperature at one inch below the surface.

DISCUSSION by G. FRANKENSTEIN, A. ASSUR

on paper by S. S. LAZIER 2/3

Q. Systematically higher temperature readings throughout the depth of the ice as given in Fig. 3 could be simply the result of neglecting to shield the thermocouples from the sun. A simple device producing a shadow could have helped.

A. Yes in cases like in Fig. 3 the readings are affected by solar radiation.
But we believe that this effect is fairly small; theoretical calculations tend to show that the maximum difference between the true temperature of the ice and the temperature indicated by the thermocouples (when affected by solar radiation) is of the order magnitude of .5°C.
A simple device producing a shadow would have helped somewhat but this is not as simple as it seems:
If the shield is large it affects the temperature of the ice itself.
If the shield is small it cuts only the direct solar radiation, but the scattered radiation which is often as important or more important than the direct radiation remains unchanged and affects the thermocouples.

DISCUSSION by G. FRANKENSTEIN

on paper by S. S. LAZIER, M. METGE

Q. Were the thermocouples shielded?
Where was your reference junction located and what temperature did you assume your reference temperature and how did you verify this temperature.

A. No, the thermocouples were not shielded and the results in Fig. 3 are somewhat suspect.
The results in Fig. 2 should be exact because the temperatures were measured at night or under heavy cloud cover.

The reference junction was located at about 1 foot below the bottom of the ice cover, it was assumed that the temperature of the reference junction was $0^{\circ}C$. In order to verify that the reference junction was at $0^{\circ}C$ we checked the gradient of temperature in the water. We found that the temperature of the water was the same at all levels, (this being due to the slight current in the channel) and concluded that this temperature was $0^{\circ}C$.

DISCUSSION by SCHWARZ

on paper by 2/3 S. S. LAZIER, M. METGE

Q. Has there been an investigation into the rise of the temperature of an ice-sheet, in relation to time when the ice-sheet being dry (or grounded) is suddenly floated in water (e.g. by incoming tide)?

A. No, we have not studied this particular case.

ICE SYMPOSIUM 1970
REYKJAVIK

TEMPERATURE AND FLOW CONDITIONS DURING THE FORMATION OF RIVER ICE

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Graduate Research Associate — Director

Iowa Institute of Hydraulic Research
The University of Iowa
Iowa City, Iowa, U.S.A.

SYNOPSIS

An investigation of the temperature and velocity characteristics of flow in rivers during the onset and occurrence of ice covers is described. Vertical and lateral temperature and velocity distributions, and ice thickness and configuration were measured in an Iowa river at frequent intervals during the period of ice cover. Variations in the lateral and vertical temperature distributions are reported. The undersurface of the ice was observed to remain plane as the ice thickened and to become wavy as the ice melted. The shear velocity associated with the wave forms are determined. Just prior to breakup the ice was observed to become very porous in the lower portions of the ice cover. Preliminary observations of the diurnal temperature variation prior to the formation of an ice cover are described.

RESUME

Une étude des températures et des vitesses de l'écoulement en rivieres avant et pendant la formation de glace est présentée. La distribution verticale et latérale des températures et des vitesses, l'épaisseur et la forme de la couche de glace recouvrant la riviere Iowa ont été mesurées a de frequents intervalles. Les variations daus la distribution verticale et latérale des températures sont notées. Il a été observé que l'interface glace-eau reste plane lorsque la couche de glace epaissit tandis que des vagues apparaissent lorsque la glace fond. La vitesse de frottement associée á la forme des vagues est calculée. Il est apparu que juste avant quelle ne se rompe la glace devient trés poreuse dans la zone de contact avec l'eau. Les observations preliminaires des variations diurnes de la température avant le formation de la glace sont décrites.

INTRODUCTION

The purpose of the study described herein was to investigate the temperature and velocity characteristics of flow in rivers during the onset and occurrence of ice covers. Vertical and lateral temperature and velocity distributions, and ice thickness and configuration were measured in an Iowa river at frequent intervals during the period of ice cover. The information obtained is being used to guide the design of controlled experiments to be conducted in the Iowa Low Temperature Flow Facility.

SITE DESCRIPTIONS

The primary site of the investigation was the Cedar River near Conesville, Iowa. At this site the river's drainage area is 7,785 square miles and the average discharge during the period of investigation was 1,490 cfs. The river is located in a rural area, with the nearest city upstream located 50 miles upriver. The U.S. Geological Survey operates a continuous-record gaging station at the site, and hence continuous records of stage and discharge were available. During periods of thin ice or no ice, gaging was conducted from a bridge. When the ice was thick enough to walk on safely, measurements were taken approximately 150 feet upstream from the bridge.

Some data were also obtained on the Iowa River at Iowa City, and on the Mississippi River at Muscatine. The Iowa River at the site of the investigation has a drainage area of 3,270 square miles. The site was in the backwater portion of a low overflow dam one-half mile downstream. The river is also affected by another small overflow dam two miles upstream and a flood control reservoir five miles upstream. The Mississippi River at Muscatine has a tributary area of 90,000 square miles. The measurements on the Mississippi were conducted from a bridge located one-and-one-half miles downstream from a navigation dam with a lift of 9 feet.

INSTRUMENTATION

Velocity distributions were measured with a Price current meter when gaging from a bridge, and with a vane meter mounted on an ice rod when working from the ice surface. Temperature measurements were obtained with a thermistor enclosed in a 5/32 inch diameter stainless steel tube extending from a waterproof tube enclosing the cable connectors. This tube was mounted 3-1/2 inches above the current meter or on the end of a staff gage. The lead wires of the thermistor probe were enclosed in plastic tubing and extended to a specially designed and constructed resistance meter, with components selected to operate in ambient temperatures as low as -20°C. The entire unit was battery operated and could be hand carried. The resistance meter had a resolution of 1/10 ohm. The thermistors used had a nominal resistance-temperature variation of 350 ohms per degree Centigrade, with a resultant temperature resolution capability of 1/3500 °C for the system. The thermistors were calibrated at the ice point and at other points using a precision thermometer graduated to 1/100 °C.

DIURNAL TEMPERATURE VARIATIONS

Four observations of the temperature variation over periods ranging from 1 to 6 hours were made to determine the nature of diurnal variations in the average temperature of natural rivers prior to the formation of ice. The first observation was on the Cedar River during clear sunny weather with little wind and an air temperature ranging from -2.3°C to +1.0°C. In a period of 1.25 hours the water temperature at the center of the river increased from 0.82°C to 1.18°C, corresponding to the rate of increase of 0.36°C per hour. Since there was little wind and the air temperature was very near the water temperature, most, if not all, of the increase is attributed to heat gain by solar radiation. Since it is clear that most of the cooling under such conditions takes place at night, observations were made on the Iowa River with the thermistor probe fixed at mid-depth of the flow. The result is presented in figure 1. In the early part of the night, when there was a low, dense cloud cover overhead, the water temperature increased at a rate of 0.06°C per hour. At about 2:15 A.M. the sky began to clear and by 3:15 A.M. it was completely clear. After clearing the temperature decreased at a rate of about 0.046°C per hour. A similar observation on 19 December at the same site, with an air temperature of -7.5°C and a clear sky showed a cooling rate of -.074°C per hour. The significance of a cloud cover is evident, and any attempts to predict the diurnal variation of river temperature must consider this variable.

VERTICAL TEMPERATURE DISTRIBUTIONS

Vertical temperature profiles were obtained at the Cedar River site on 12 dates from 4 December 1969 to 23 February 1970. Measurements were taken in the thalweg and at least three other points of the cross-section. Prior to the formation of an ice cover the maximum difference in temperature over any single vertical profile was about 0.015°C. Short-term temporal variations, believed to be associated with the turbulence structure of the flow, exhibited temperature variations of up to 0.06°C. After formation of a complete ice cover the vertical variations were much less, as were also the short-term temporal variations; both types of variations decreased as the ice cover increased in thickness. With an ice cover of about 11 inches, vertical and temporal variations were less than 0.003°C and 0.001°C, respectively. Typical vertical temperature profiles for different conditions of ice cover are shown in figures 2 and 3. The discrepancy in figure 2 between the upward and downward traverse near the ice is attributed to warming effects originating at the access hole.

Vertical temperature profiles were obtained on the Mississippi River prior to the formation of an ice cover on two occasions, when the water temperatures were 3°C and 0.2°C. Temperature variations over any vertical profile were less than 0.04°C. No discernible vertical gradients of temperature were observed.

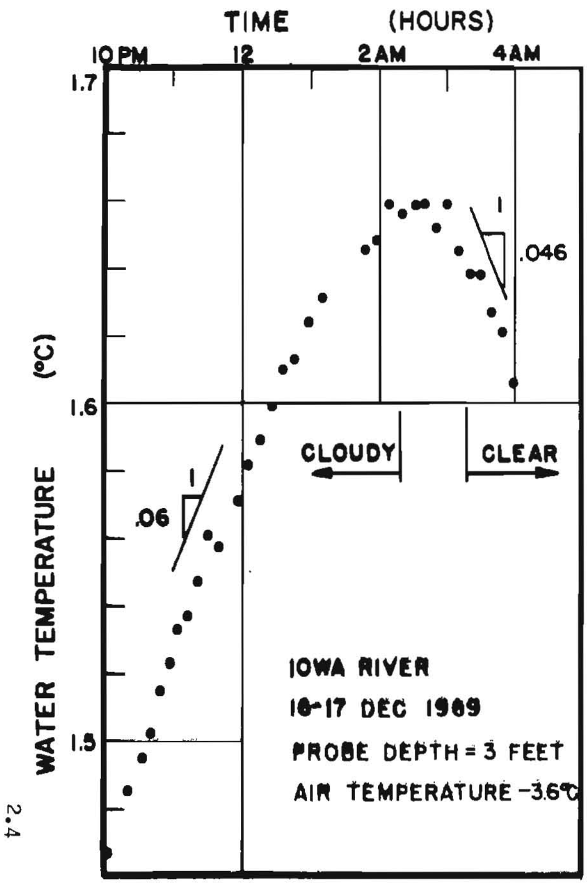


Figure 1

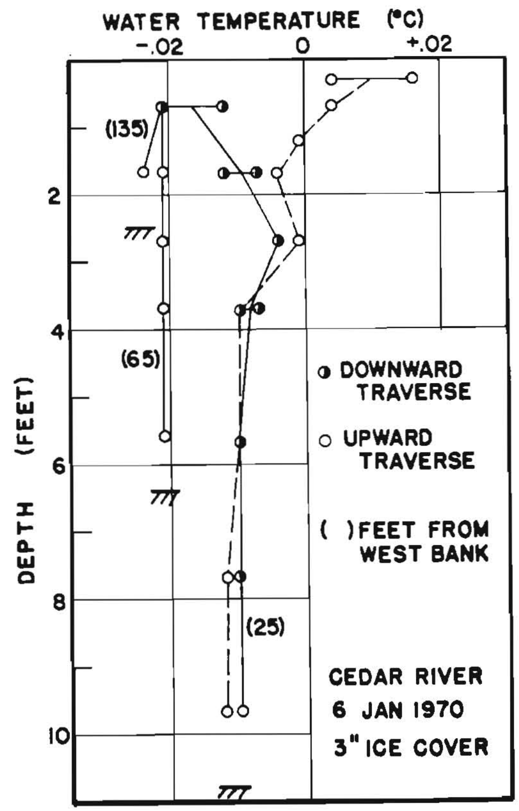


Figure 2

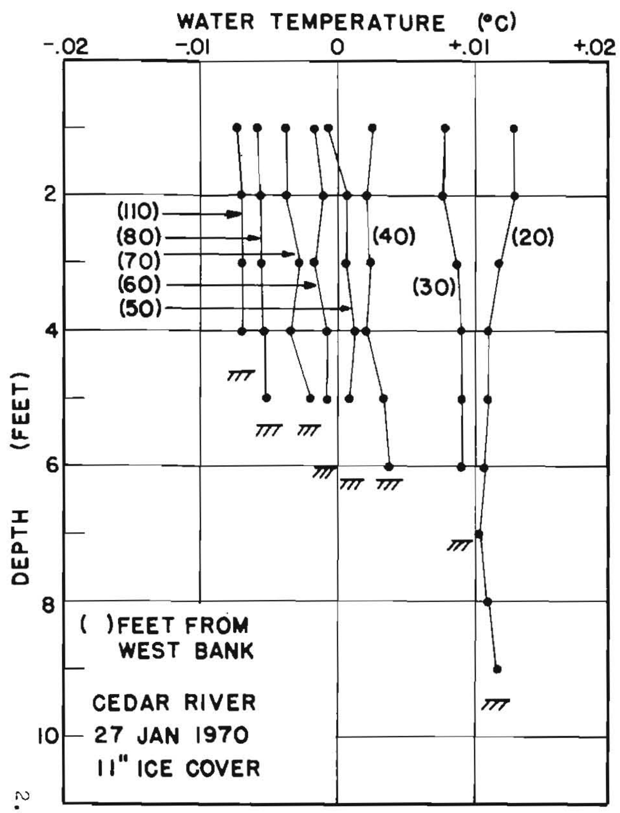


Figure 3

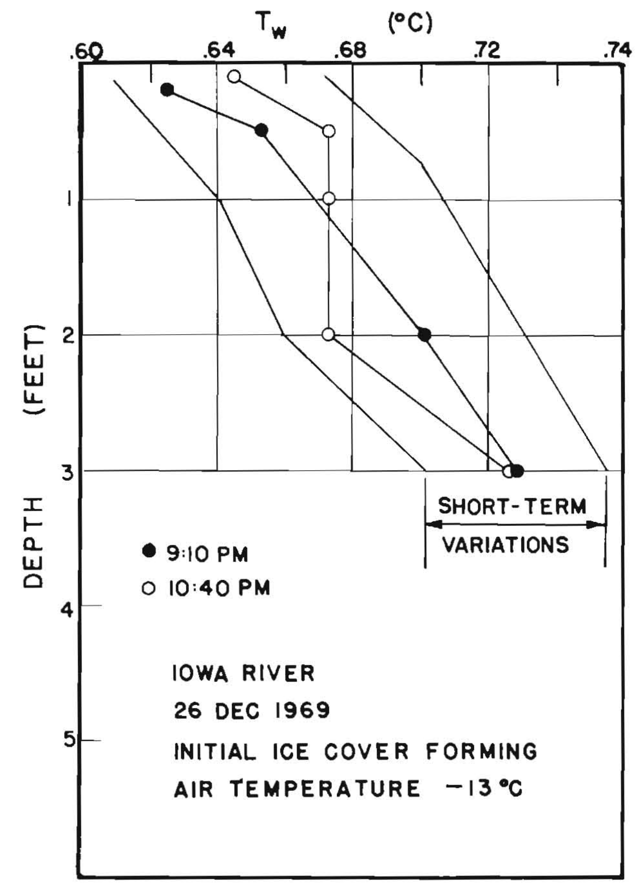


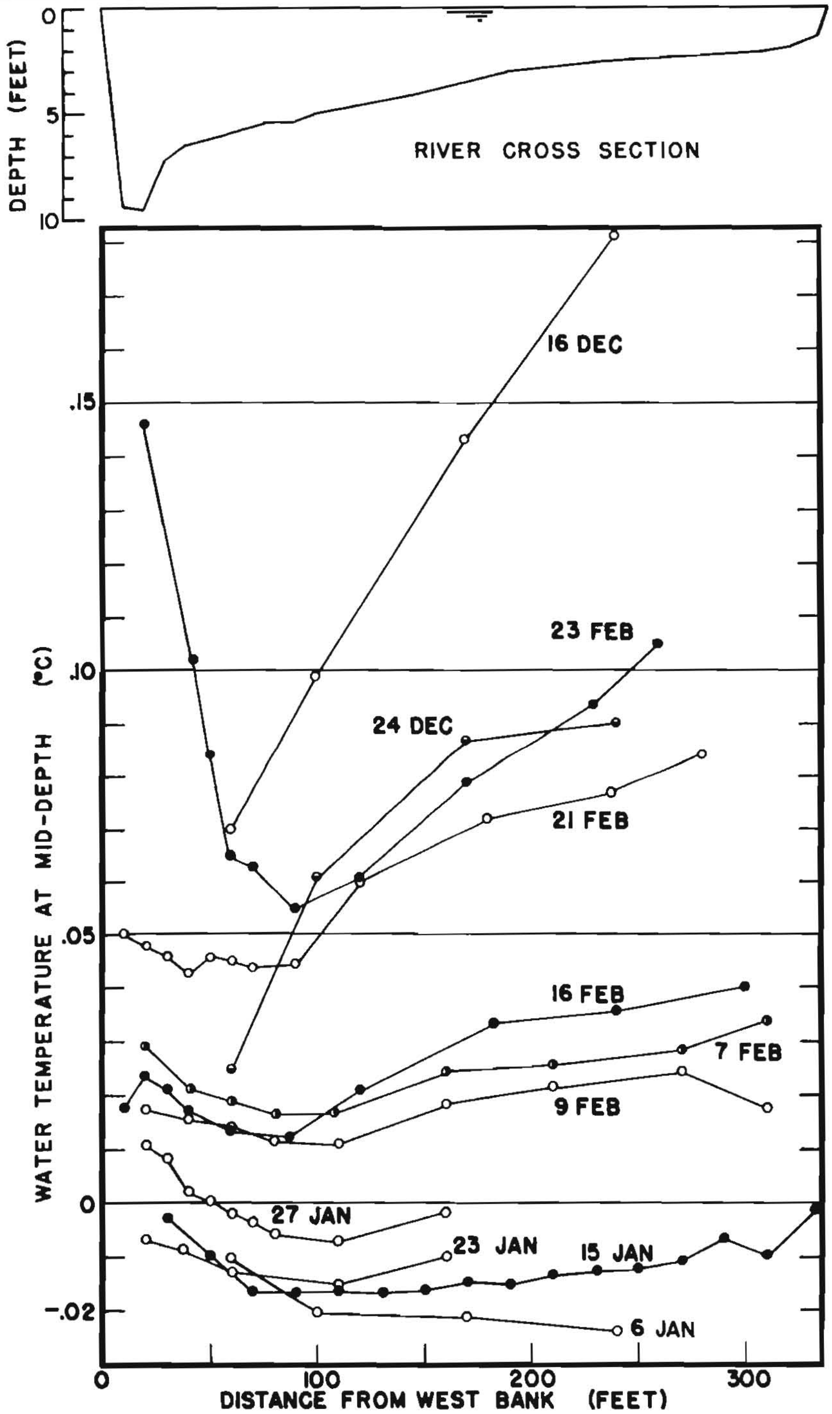
Figure 4

On the night of 26 December 1969 a vertical temperature profile was obtained as an initial ice cover, in the form of grease ice, was forming on the surface of the Iowa River. The temperature distributions are presented in figure 4 together with an indication of the short-term temporal variations which limited the accuracy of the measurements. The air temperature was -13°C and the water temperature was 0.6°C at about 1 inch below the water surface; hence a large thermal gradient existed near the water surface. At the time of measurement the velocity was 0.8 feet per second and the total depth was 6.0 feet. The long-term variation of temperature at mid-depth over the period from 7 P.M. to 12 P.M. was slight and no general pattern of cooling was observed, indicating that the heat exchange was concentrated near the surface of the water.

LATERAL TEMPERATURE VARIATIONS

The detailed temperature surveys conducted on the Cedar River provided a number of observations of the lateral variation of temperature in a natural river during the period of ice cover. The results are presented in figure 5. On 16 December and 24 December there were only minor amounts of shorefast ice, but there were some floating frazil ice patches concentrated primarily in the thalweg area. These frazil patches rapidly diminished in size during the day. The temperature of the flow was observed to have a marked lateral variation with the floating frazil ice causing low temperatures in the thalweg area. By 6 January the river was entirely ice covered with an average ice thickness of about 3 inches. The water temperature was nearly uniform across the cross-section, with only slightly higher temperatures occurring in the thalweg area. Since the measured water temperatures were below 0°C, ice points were measured using river water to determine if the sub-zero temperatures represented supercooling. The freezing points of the river water ranged from -0.027 to -0.019°C, and in all cases were lower than the minimum water temperatures observed at the time of collection of the samples. The depression in the freezing point is attributed to dissolved impurities in the water. The following week, and thereafter until general melting began, higher temperatures were observed near the banks, with the minimum temperatures occurring in the central part of the river. As the winter progressed the water gradually warmed while maintaining this general temperature variation. On 21 February rapid melting of the undersurface had begun and a larger lateral temperature variation was observed. By 23 February all ice within 1 to 10 feet of the shores had melted completely. It is seen in figure 4 that there were very strong temperature gradients between the shores and the central part of the river. These gradients are attributed to lateral diffusion of warmer water originating at the bank areas where the water was in contact with the air, which was at 5°C. General ablation of the ice cover was occurring and the lower 5 to 6 inches of the 8-inch thick ice cover was very porous and rotten.

Lateral variation in temperature was also observed in the Mississippi River



DEPTH (FEET)
0
5
10
RIVER CROSS SECTION
WATER TEMPERATURE AT MID-DEPTH (°C)
.15
.10
.05
0
-.02
16 DEC
23 FEB
24 DEC
21 FEB
16 FEB
7 FEB
9 FEB
27 JAN
23 JAN
15 JAN
6 JAN
0
100
200
300
DISTANCE FROM WEST BANK (FEET)

Figure 5

2.4

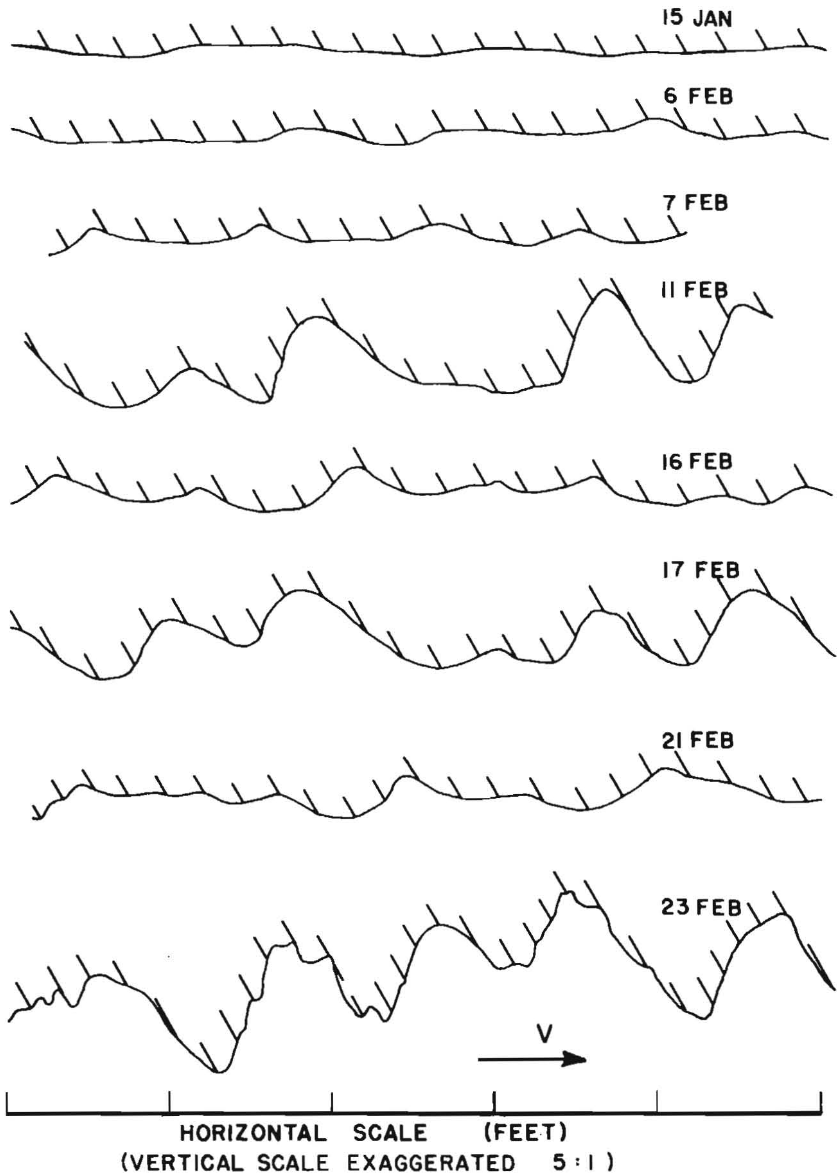
prior to the formation of an ice cover, with a maximum lateral variation of 0.16°C observed when the water temperature was 3°C. The main channel section tended to have the lowest temperature while the higher temperatures occurred near the banks. A detailed lateral survey was not obtained.

VELOCITY DISTRIBUTIONS

Flow velocities were also measured during the period of ice cover. Prior to the formation of the ice cover the velocity profiles exhibited the expected form, the maximum value occurring near the free surface and the velocity diminishing monotonically to zero at the bed. After the formation of the ice cover the velocity decreased from a maximum near mid-depth toward zero at the top and bottom of the flow section.

To investigate the roughness characteristics of the undersurface of the ice a number of slabs, averaging 5 feet by 1-1/4 feet, with the long dimension parallel to the flow direction, were cut, overturned, and profiles measured (see figure 6). In the early part of the season the undersurface of the ice was flat and hydraulically smooth. As the season progressed the undersurface became rough, with a relief pattern very similar to those observed by Carey (1,2) and by Larsen (3). The relief pattern resembled sand ripples. Three stages were observed in the development of the relief pattern. The first stage consisted of a wavy pattern with the crests and troughs transverse to the flow direction. Wave lengths were typically between 11 and 19 inches, although some larger values were observed early in the season. There was a tendency for the wave length to become shorter as the season progressed with a typical wave length being about 12 inches at the end of the season of ice cover. As the relief pattern developed, most of the undulations became sharp-crested at the upstream end of the trough and invariably had a small step, about 1/8 inch in height, at the upstream end of the region of greater melting. Finally as the relief pattern became pronounced, and as general ablation of the ice cover was occurring, there was observed a "pocking" of the undersurface with the pockmarks initially concentrated primarily in the troughs of the relief pattern. These pockmarks had a diameter of about 3/4 inch and depths into the ice of as much as three inches.

Figure 7 summarizes data obtained on maximum and minimum daily air temperatures, T_a; the ice thickness, h ; the amplitude of the ice waves, A ; the wave form index, A/L ; the minimum water temperature, T_w; the shear velocity, u_*; and the stage height, H . Examination of this figure indicates that the relief pattern did not form during the period of general accretion, except for some very long wave lengths, and that not until there was general melting did the undulations begin to develop. Beginning about 6 February the ice cover started melting and the interfacial relief pattern appeared and tended to increase in intensity with further melting. In the latter stages of the melting process, the pocking appeared and it was observed that the ice cover "rotted" from below, becoming very porous in the



TYPICAL PROFILES OF THE
ICE-WATER INTERFACE

Figure 6

2.4

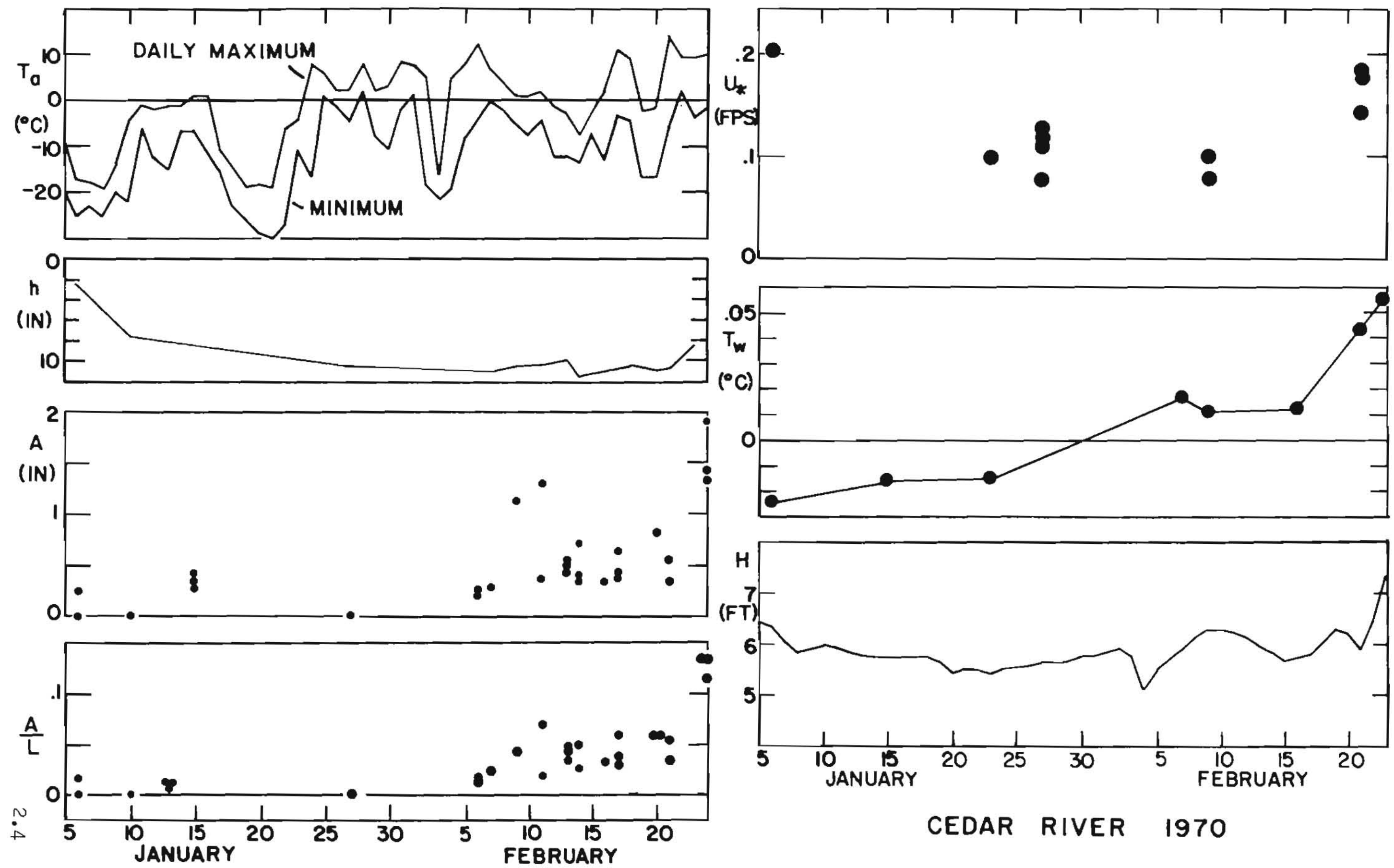

DAILY MAXIMUM
MINIMUM
T_a (°C)
h (IN)
A (IN)
$\frac{A}{L}$
U_* (FPS)
T_w (°C)
H (FT)
JANUARY
FEBRUARY
CEDAR RIVER 1970

Figure 7

lower portions.

The effect on the velocity distribution of the roughness pattern of the ice relief was investigated by calculating the magnitude of the shear velocity, u_*, from the velocity profiles using the relationship

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln y/y_o + \text{const}$$

There was a tendency for u_* to have a high value at the beginning of the period of ice cover, to decrease to a value of about 0.1 fps during most of the season, and then to increase near the end of the period of ice cover. The initially high value is attributed to the presence of slush ice beneath the solid ice cover during the early period of formation. Observations after 15 January revealed little or no slush ice. This is in general agreement with the observations reported by Nezhikovskiy (4), that the slush ice concentration decreases through the winter season. The increase in u_* near the end of the season coincided with the appearance of the pocking phenomena which obscured the previously smooth wave forms (see in particular the profile of 23 February in figure 6).

CONCLUSIONS

The vertical temperature distributions measured were very uniform over the depth, with a maximum difference over any one vertical profile decreasing from about 0.015°C prior to the formation of an ice cover to less than 0.003°C under a thick ice cover. Short-term temporal variations, associated with the turbulence structure of the flow, also decreased with the formation and thickening of the ice cover. The lowest temperatures in the lateral direction were consistently in the central parts of the flow with warmer temperatures observed near the banks. The lateral variations under a thick ice cover were found to be small, of the order of a few hundredths of degrees or smaller, until just prior to breakup when larger thermal gradients were found near the shore areas. It was found that an ice cover could form with a temperature of 0.7°C at mid-depth of the flow when the thermal regime of the river is modified by reservoir and urban effects. Cloud cover was found to exert significant effects on the diurnal variation of river temperature prior to the formation of a permanent ice cover.

The effect of an ice cover was found, as expected, to change the velocity distribution from the usual open channel velocity profile to a profile typical of closed conduits. The relief pattern at the ice-water interface, and the consequent roughness of the undersurface of the ice cover, was studied in some detail. It was found that as the ice thickened the bottom surface of the ice tended to remain flat, but as ablation set in it became undular with wave form oriented transverse to the flow direction. Wave length tended to decrease, and amplitudes to increase, as the season progressed. Just prior to breakup the ice was observed to become very porous and "rot" in the lower portions. There was a tendency for the shear velocity to decrease in the early part of the period of ice cover and to increase with the appearance and development of the wavy relief pattern.

ACKNOWLEDGMENTS

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DISCUSSION BY G. TSANG

As explained by the author that he obtained the friction velocity u_* by using the fully turbulent velocity distribution law

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln \frac{y}{y_o} \quad \text{const}$$

by plotting ln u versus lnln y/y_o, using data points for the first 3 or 4 feet. The shear stress so obtained is invalid since the equation is only accurate for regions having a constant stress. It is well known that the distribution of shear stress between two parallel plates is linear from maximum at the boundary to zero at the point of maximum velocity. Therefore, it is my opinion that the value of u_* so obtained should be considered as an indication of the order of magnitude. The shear stress so calculated would also tend to be underestimated.

REPLY

The determination of u_* was made by plotting the velocity, u, versus the log of y with y taken as the distance to the point of measurement from the bottom surface of the ice at the access hole cut for the measurements. The slope of the line was then used to determine u_* assuming 0.4 for κ. Deviation from this straight line generally occurred at approximately 2.5 to 3.0 feet from the ice boundary. The validity of this logarithmic law is based on the commonly used assumption that the shear stress throughout the region considered is constant (see Schlichting (5), Chapter XX).

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ICE SYMPOSIUM 1970
REYKJAVIK

CALCULATION OF WATER TEMPERATURE VARIATION ALONG A CHANNEL OF GREAT LENGTH WITH VARYING FLOW CONDITIONS

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Water Transport Institute
Leningrad, U.S.S.R.

In this work are given the analytical relationships for calculating water temperature averaged over depth, along a channel of great length when the free flow movement is non-stationary.

Dans le travail on améne les dépendances analitiques pour le calculage de la profondeur l'average de temperature le long du canal de grande longueur, quand le mouvement du courant n'est pas stationaire.

The purpose of given paper is to obtain the analytical relationships for calculating water temperature, averaged over depth, along a channel of great length when a direct wave of free flow propagates in the same. For finding a solution to the problem it is assumed that the bed heat output and ambient air temperature are constant.

Heat transfer due to thermal conductivity is neglected for most of heat is transfered by the water flow.

- " -

Heat content variation per unit of width in a section dx of a two-dimensional flow during dt can be expressed as:

$$dS = c_t \gamma \left(\frac{\partial Q}{\partial x} \tau dx dt + Q \frac{\partial \tau}{\partial x} dx dt + \frac{\partial Q}{\partial x} dx dt \frac{\partial \tau}{\partial x} dx \right) \qquad (1)$$

where: Q, τ, C_t, γ present the flow rate, temperature, heat capacity and specific weight of water, respectively.

Heat content variation dS must be equal to:

a) heat output of the open water surface dS_1;

b) heat inflow from the ground, dS_2;

c) heat increment due to the section volume variation dS_3.

Heat output of water surface area with the length of dx and width equal to a unit within period of time dt, is:

$$dS_1 = -(k\tau + M) dx dt \qquad (2)$$

where: k and M are coefficients depending on the wind velocity, air moisture deficit and temperature of the air.

The heat inflow from the bed ground during the same period of time:

$$dS_2 = q_{zp.} \cdot dx dt \qquad (3)$$

here $q_{zp.}$ is the heat output of the bed per unit of the surface.

The increment of heat due to the section volume variation is:

$$dS_3 = -C_t \gamma \left(\frac{\partial \omega}{\partial t} dx dt \tau + \omega dx dt \frac{\partial \tau}{\partial t} + \frac{\partial \omega}{\partial t} dx dt \frac{\partial \tau}{\partial t} dt \right) \qquad (4)$$

Comparing Equation (1) with Equations (2), (3) and (4), and dividing all the terms by $dx, dt \cdot c_t \gamma$, one obtains:

$$\tau \left(\frac{\partial Q}{\partial x} + \frac{\partial \omega}{\partial t} \right) + \frac{\partial \tau}{\partial x} \left(Q + \frac{\partial Q}{\partial x} dx \right) + \frac{\partial \tau}{\partial t} \left(\omega + \frac{\partial \omega}{\partial t} dt \right) = A\tau + B \qquad (5)$$

WHERE: $A = -\frac{k}{C_t \cdot \gamma}$ and $B = \frac{q_{zp.} - M}{C_t \cdot \gamma}$ (6)

In view that $\frac{\partial Q}{\partial x} + \frac{\partial \omega}{\partial t} = 0$, we can write:

$$\frac{\partial \tau}{\partial x}\left(Q + \frac{\partial Q}{\partial x}dx\right) + \frac{\partial \tau}{\partial t}\left(\omega + \frac{\partial \omega}{\partial t}dt\right) = A\tau + B \qquad (7)$$

Considering the second order terms as negligible we finally obtain:

$$\frac{\partial \tau}{\partial x}Q + \frac{\partial \tau}{\partial t}\omega = A\tau + B \qquad (8)$$

or:

$$\frac{\partial \tau}{\partial x}v h + \frac{\partial \tau}{\partial t}h = A\tau + B \qquad (9)$$

In the case of propagation of a low amplitude wave in a channel of constant depth, Equation (9) is written in the form of:

$$\frac{\partial \tau}{\partial x}(H_o + h)(v_o + v) + \frac{\partial \tau}{\partial t}(H_o + h) = A\tau + B \qquad (10)$$

Here: H_o and v are the depth and velocity of stream with no free flow wave;

h - is the free flow wave amplitude;

v - is the velocity of stream, average over cross-section with the free flow wave present (the initial velocity being not taken into account).

We assume that on the starting line of direction, viz. x = 0, the water table variation is in conformity with linear equation $h_{x=o} = \alpha \cdot t$. The the last expression can be rewritten as:

$$\frac{\partial \tau}{\partial x}\left[mz\left(1 + \frac{v_o}{c}\right) + nz^2 + Q_o\right] + \frac{\partial \tau}{\partial t}\left[H_o + zu\right] = A\tau + B \qquad (11)$$

where $m = H_o \alpha_1 c$; $n = H_o \alpha_1^2 c$; $u = H_o \alpha$; $z = x - ct$

$$\alpha_1 = -\frac{\alpha}{H_o c} ; \qquad c = \sqrt{gH_o}$$

Solving Equation (11) we obtain the first integrals:

$$C_1 = \frac{(z - z_1)^{B_1}(z - z_2)^{B_2}}{e^t} \qquad (12)$$

$$C_2 = \frac{(z - z_2)^{A_2}}{(z - z_1)^{A_2}(\tau + B_o)^{1/A}}$$

When there is no initial flow or when its velocity, v_0 , is considerably small compared to that of propagation of the wave front, the differential Equation (11) assumes the form of:

$$\frac{\partial \tau}{\partial x}(mz + nz^2) + \frac{\partial \tau}{\partial t}(H_0 + uz) = A\tau + B \qquad (13)$$

The last Equation is solved quite similarly as Equation (11), the only difference lying only in the values of coefficients.

In the given case:

$$Z_1 = \frac{1}{\alpha_1}\ ;\ Z_2 = -\frac{1}{\alpha_1}\ ; \qquad A_1 = -A_2 = \frac{1}{2m} = \frac{1}{2\alpha_1 H_0 C}$$

Having in mind that $Z_1 = -Z_2$ the first integrals resulting from solving Equation (13) will be:

$$C_1 = \frac{(Z + Z_2)^{B_1}}{e^t}$$

$$C_2 = \frac{(Z - Z_2)^{A_2}}{(Z + Z_2)^{A_2}(\tau + B_0)^{1/A}} \qquad (14)$$

The general solution of Equation (13) is of the form:

$$\psi\left[\frac{(Z + Z_2)^{B_1}}{e^t};\ \frac{(Z - Z_2)^{A_2}}{(Z + Z_2)^{A_2}(\tau + B_0)^{1/A}}\right] = 0 \qquad (15)$$

where: ψ is an arbitrary function of C_1 and C_2. The value of this function can be found from boundary conditions. Thus if a boundary condition is x = 0 then $\tau = \tau_0$, hence the relationship between C_1 and C_2 will be:

$$C_1 = \frac{\left\{\dfrac{-2Z_2}{C_2^{1/A_2}(\tau_0 + B_0)^{1/D}}\right\}^{B_1}}{\exp\dfrac{Z_2[C_2^{1/A_2}(\tau_0 + B_0)^{1/D} + 1]}{C[C_2^{1/A_2}(\tau_0 + B_0)^{1/D} - 1]}} \qquad (16)$$

or:

$$\frac{(Z+Z_2)^{B_1}}{e^t} = \frac{\left\{\dfrac{-2Z_2\,(Z+Z_2)(\tau+B_0)^{1/\mathcal{D}}}{(Z-Z_2)(\tau_0+B_0)^{1/\mathcal{D}}-(Z+Z_2)(\tau+B_0)^{1/\mathcal{D}}}\right\}^{B_1}}{\exp\dfrac{Z_2\left[(Z-Z_2)(\tau_0+B_0)^{1/\mathcal{D}}+(Z+Z_2)(\tau+B_0)^{1/\mathcal{D}}\right]}{C\left[(Z-Z_2)(\tau_0+B_0)^{1/\mathcal{D}}-(Z+Z_2)(\tau+B_0)^{1/\mathcal{D}}\right]}}$$

where: $B_1 = -\frac{1}{2\alpha_1}$; $B_0 = \frac{B}{A}$; $\mathcal{D} = A \cdot A_2$

It is not possible to directly solve the last Equation for τ , that is why the solution is to be found by trial-and-error method. This method was used as an example of calculation made according to the following initial data:

1 - water level in the initial range of channel is lowering according to the formula $h = \alpha t$ where α = 1 m/hr;
2 - initial depth of water in channel H_0 = 4m;
3 - the temperature of water in the initial range τ_0 = 5°C;
4 - air temperature θ = -10°C;
5 - air moisture deficit Δ = 3 mm mercury;
6 - wind velocity W = 5 m/sec.

In Fig.1 is given a diagram of varying water temperature according to the length of the channel for different time moments. This diagram was obtained as a result of calculations being made.

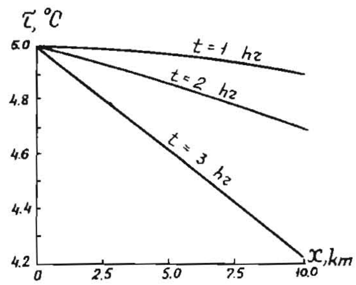

Fig. 1. Diagram of varying water temperature according to the length of the channel.

The above given data were described more detailed by the author in his work issued in 1968.

B.S.Borodkin: "Calculating variation of water temperature, averaged over the depth, along a channel of great length under conditions of non-stationary flow movement", Proceedings of coordinating conferences on hydrotechnology, Issue,42; Leningrad, 1968.

ICE SYMPOSIUM 1970
REYKJAVIK

WINTER THERMAL REGIME OF NON-FREEZING CANALS AND CONTROL OF ICE TROUBLES DUE TO WATER LEVEL FLUCTUATIONS

REGIME THERMIQUE D'HIVER DES CANAUX NON CONGELES ET LUTTE CONTRE LES DIFFICULTES DE GLACE DUES AUX VARIATIONS DE NIVEAUX D'EAU

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Synopsis

In a general case non-freezing canals are divided into five reaches characterized by specific ice conditions - absence of ice, formation of frazil ice anchor ice or frazil ice cover. The equations are presented for determining the length of the reaches, water temperatures and ice formations; the effect of heat transfer by turbulent heat conductivity along the flow is analysed. The data are given on freezing - over of hydraulic structures due to water level fluctuations and on the intensity of ice cohesion to the surfaces covered by hydrophobic materials.

Résumé

En cas général les canaux non congelés se divisent en 5 tronçons avec régimes des glaces correspondants - absence de la glace, formation du sorbet, de la glace de fond, du tapis de sorbet. On donne les formules pour déterminer la longueur des tronçons, la température de l'eau, les formations de glace: on analyse l'effet du transfert de chaleur par conductivité thermique turbulente suivant le courant. On présente les données concernant la glaciation des ouvrages aux variations de niveau et la force de cohésion de la glace avec surfaces revêtues de matériaux hydrophobes.

§1. A non-freezing canal can be divided into reaches with various thermal regimes. In a general case five reaches are distinguished. The first stretch is characterized by water temperatures above 0°C over the whole depth of the flow; along the second stretch the layer at the water surface becomes supercooled; in the third stretch supercooling reaches the bottom; in the fourth stretch water temperature rises up to 0°C; in the fifth stretch water temperature in the surface layer is approximately 0°C, while in the bottom layer it is above freezing.

Each of the stretches considered is characterized by specific ice conditions - absence of ice, formation of frazil ice, anchor ice or frazil ice cover. The Laboratory of Rivers and Reservoirs Winter Regime of the All-Union Research Institute of Hydraulic Engineering (VNIIG) studies temperature gradients over the flow depth, the length of the stretches mentioned above, in particular that of the stretch most dangerous due to probable ice troubles. As a result, the curves for water temperature changes over the depth and along the canal are plotted, the significance of heat transfer by turbulent heat conductivity along the flow is analysed. Further investigations are performed on preventive measures against freezing-over of canal slopes and hydraulic structure elements due to water level fluctuations.

The data obtained are used in designing of canals intended for operation under various winter conditions.

§2. The thermal regime of a non-freezing canal is described by the following differential equation:

$$\lambda_x \frac{\partial^2 t}{\partial x^2} + \lambda_z \frac{\partial^2 t}{\partial z^2} - c\gamma v \frac{\partial t}{\partial x} + q_i = 0 \qquad (1)$$

The boundary conditions are:

$$t|_{x=0} = t_0 ; \qquad (2)$$

$$-\lambda_z \frac{\partial t}{\partial z}\Big|_{z=0} = \alpha(\vartheta_e - t_{z=0}) \qquad (3)$$

where x, z are the coordinates; h -the canal depth; t -water temperature; t_0 -initial temperature of water; λ -coefficient of thermal conductivity; α -heat release factor; ϑ_e-effective temperature of the ambient air; v-flow velocity; c - heat capacity of water; γ -specific weight of water.

In case heat transfer due to conduction along the flow is neglected for the period prior to the intensive frazil and bottom ice formation $/ q_i = 0 /$ the problem can be solved with the help of equation

$$\theta \equiv \frac{t - t_0}{\vartheta_e - t_0} = \cos\left[\sqrt{B_i}(1-\eta)\right] \exp(-M_i), \qquad (4)$$

where

$$B_i \equiv \frac{\alpha h}{\lambda_z}; \qquad \eta \equiv \frac{z}{h}; \qquad M_i \equiv \frac{\alpha x}{c\gamma h v}.$$

The mean temperature of water can be found by equation

$$\bar{\theta} \equiv \frac{\bar{t} - \vartheta_e}{t_0 - \vartheta_e} = \frac{\sin\sqrt{B_i}}{\sqrt{B_i}} \exp(-M_i) . \qquad (5)$$

From Eqs./4/ and /5/ the section is established where the supercooling of the flow starts, i.e. the length L along which $t=0$ with $\eta=0$, as well as the section of maximum supercooling, i.e. the length L', where $\tau = t_{min_0}$ with $\eta=0$; the thickness of the supercooled layer is also determined therefrom.

When water becomes supercooled to the bottom, anchor ice forms in the canal. The rate of ice production depends upon temperature gradients across the flow depth.

§3. When the water temperature gradient over the depth is insignificant, instead of Eq./1/ we have

$$\lambda_x \frac{\partial^2 t}{\partial x^2} - c\gamma v \frac{\partial t}{\partial x} = \alpha(\vartheta_e - t) \qquad (6)$$

The analytical solution of Eq./6/ under the conditions given by Eq./2/ is as follows:

$$\theta \equiv \frac{t - \vartheta_e}{t_0 - \vartheta_e} = exp\left[\frac{P_e}{2} - \sqrt{\left(\frac{P_e}{2}\right)^2 + P_e M_i}\right] \qquad (7)$$

where $P_e \equiv \frac{c\gamma v x}{\lambda}$. The analysis of Eq./7/ shows that heat transfer by conduction along the flow plays an essential part only at a relatively short reach of the stream, so it can generally be neglected.

From the section with a zero izotherm the total heat loss into the air can be considered to account for frazil ice production; in this it is necessary to include the presence of partial or continuous frazil ice cover on the flow surface. The transporting capacity of the flow depends not only upon the canal depth and flow velocity, but also on the ambient air temperature.

§4. The problems of freezing-over of canal slopes and hydraulic structure elements as well as control of ice troubles due to water level fluctuations, though very important, have been inadequately studied. One of the preventive measures against freezing-over of the surface of structural elements is application of hydrophobic materials, the efficiency of this measure depending on the porosity of the surface.

The analysis of interaction between fluid and a solid surface shows that ice adhesion to a porous concrete surface is reduced due to hydrophobic materials. On the other hand, application of hydrophobic materials to non-porous metal surfaces does not lead to a considerable decrease in the ice adhesion because, irrespective of the intensity of hydrophobization, the specific surface of water-to-metal contact is the same for all the forms of mechanical bond.

Testing of various hydrophobic materials by means of specially designed installation helped to establish the dependence between the strength of ice adhesion to a surface /R/ and temperature, specimen dimensions, loading rate, salinity of water (ice), number of test runs.

Test results are given in Fig. 1.

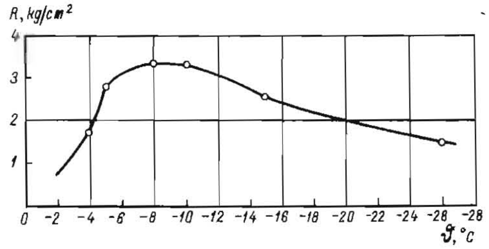

Fig.1.

The dependence of R upon temperature is most representative. The range of 4-12°C appeared to be most dangerous from the point of view of maximum adhesion. R decreases with the increase in the salinity of ice.

Cryophobic properties of various coatings were tested at a number of actual power plants. Observations of the behaviour of coatings, the character of their freezing-over and ice adhesion to protected and unprotected concrete surfaces confirmed that the cryophobic coatings (some organo-silicon compounds) reduce ice adhesion; these surfaces being considerably easier to deice.

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REYKJAVIK

L'ASSURANCE DE L'ALIMENTATION ININTERROMPUE PAR L'AMENÉE D'EAU OUVERTE EN HIVER

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A method by which a failureless winter operation in an open service water supply channel in Northern Bohemia is successfully secured by utilizing heated wastewater from a steam power plant is presented. The length of the supply channel is 21,9 km and the max. discharge 3.3 m^3/sec. The results of the theoretical solution of the thermal balance in the supply channel are compared with the measurement results of the individual balance items.

Dans le rapport on décrit une méthode de l'assurance supplémentaire fructueuse de l'exploitation ininterrompue pendant tout l'hiver d'un canal d'amenée ouvert pour les besoins industriels, c.à.d. à l'aide de l'eau de réfrigération de l'usine thermique d'électricité. La longueur du canal d'amenée comporte 21,9 km et le débit maximum est de 3,3 m^3/sec. On fait la comparaison des résultats de la solution théorique du bilan thermal du canal avec les résultats obtenus par les mesurages de facteurs particuliers du bilan.

En 1961 on a construit au nord de la Bohême un canal pour les besoins industriels. La station de pompage refoule l´eau par la conduite de refoulement dans un bac de neutralisation et de là par un canal d´amenée ouvert. La longueur totale du canal d´amenée, y compris la conduite par tuyau, est de 26 km environ, la longueur du canal d´amenée ouvert étant de 21,9 km.

Le profil du canal d´amenée pour le débit projeté de 3,3 m^3/sec est trapézoidal, d´une largeur de 1,00 m dans le fond, d´une profondeur de 1,20 m, les pentes étant 1 : 1,5 /Fig. 1/ et la pente du fond 0,5 jusqu´à 0,9 pour-mille. Le canal est muni sur toute sa surface d´une enveloppe en béton qui repose sur une sous-fondation d´arène. Dans les agglomérations, dans le terrain défavorable et aux deux aqueducs le profil est rectangulaire, dans deux secteurs courts le canal d´amenée est couvert. Les quatre croissements avec les communications et avec un ruisseau ont été résolus pas les siphons. Le canal d´amenée est muni par quatre ouvrages de déchargement: par un déversoir libre et par trois siphons.

Le canal d´amenée, projeté à l´origine pour être exploité en été seulement, s´est montré, comme on s´y attendait, inconvenable pour être employé en hiver. Dans les périodes d´hiver jusqu´à l´époque de l´introduction de l´eau chaude en février 1968, on a mis chaque fois l´ouvrage hors service pour période de 90 jusqu´à 115 jours.

Il fallait garantir l´exploitation durant tout l´hiver. On a élaboré des études basées sur deux principes tout à fait différents:

A. La couverture du canal d´amenée en quatre variantes.

B. Adjonction dans le canal d´amenée de l´eau chaude provenant de l´usine thermique d´électricité à Prunéřov.

C´est cette solution qui s´est motrée plus avantageuse du point de vue économique et de même de celui d´exploitation. La quantité et la témperature ont été définies par les calculs théoriques du bilan thermique du canal d´amenée.

La figure 2 représente la dépendance du refroidissement total Δt_v de l'eau dans le canal d'amenée à partir du km 2,2 jusqu'au km 21,8 à la température initiale de l'eau t_{vo}, étant données différentes situations météorologiques.

Sur la figure 3 sont dessinés les profils thermiques longitudinaux du canal d'amenée, les conditions étant les mêmes, pour les températures initiales $t_v = 5^o$, 10^o et 20 °C.

En comparant les deux cas fondamentaux les plus défavorables on voit que l'on peut attendre à peu près la même intensité de refroidissement étant données les conditions dangereuses au cours d'une nuit claire glacée comme au cours d'une nuit à une forte chute de neige et à une température plus élevée de l'air.

L'eau chaude de l'usine électrique est fournie par une pompe d'une capacité Q = 280 ℓ/sec. La température de l'eau fournie varie de +22 °C à 32 °C.

L'exploitation d'à présent du canal d'amenée a confirmé que l'addition de l'eau de réfrigération de l'usine thermique d'électricité de l'eau échauffée garantit d'une manière avantageuse, même au point de vue d'investissement, l'exploitation d'hiver sans perturbations du canal d'amenée ouvert. Une quantité d'eau d'addition relativement faible a suffi de garantir la sécurité de service du canal d'amenée d'une longueur de 20 km. Les calculs théoriques du bilan thermique, compte tenu des connaissances gagnées, se sont révélées suffisantes pour l'élaboration d'un projet économique des mesures nécessaires.

L'incertitude quant à la solution du bilan thermique du courant d'eau et la différence des résultats obtenus d'après les différents auteurs ont amené aux mesurages systématiques dont le but est de vérifier la validité des méthodes publiées dans les conditions de notre climat et cela pour un lit artificiel d'un profil menu. Les travaux de mesurage ont été exécutés durant plusieurs années; avec le mesurage intégral de tous les paramètres on a commencé en hiver 1968-69. On l'exécute dans sept profils caractéristiques du canal d'amenée. Dans tous les profiles on mesure les températures de l'eau et de l'air, l'humidité, la vitesse du vent, la profondeur de l'eau et la température de l'eau de drainage. Au km 15,2 on mesure le bilan total du rayonnement et le rayonnement solaire, absorbé.

Comme spécimen des différences entre les valeurs calculées et mesurées on présente la table 1.

Dans les rangées 1, 2, 3 on montre le profil thermique en long. On voit que les différences de température dans le profil final ne sont pas grandes. Les rangées 4 et 5 nous montrent les différences dans les composantes de la radiation et aussi quelle influence elles exercent à la température de l'eau dans le profil final.

Dans les rangées 6 et 7 se trouvent les calculs touchant le secteur du canal d'amenée d'une longueur totale de 4,7 km dont 1,0 km est couvert.

On peut voir l'influence de cette couverte en observant les résultats, car il y a une différence entre les valeurs calculées et celles que l'on a obtenues par le mesurage, la couverture n'ayant pas été prise en considération au cours des calculs. Quand le bilan de radiation est négatif le refroidissement est moindre que pour le secteur ouvert entier, quand il est positif le rechauffement par contre est plus lent.

On ne peut pas cependant tirer des conclusions univoques sur la base du mesurage et de la restitution partielle. Le mesurage continue et on évalue les résultats. Mais il est possible de signaler certaines différences qui sont tout à fait évidentes.

Ce sont les composantes de radiation qui présentent les plus grandes différences par rapport aux valeurs calculées. Il s'agit des expressions pour lesquelles la nébulosité a une grande importance. Dans ces expressions on n'attache pas suffisement d'attention à la densité des nuages, ce qui représente la cause principale quant aux grandes différences par rapport auc valeurs obtenues par mesurage. Mais le mesurage n'a pas entièrement confirmé ces relations même quand le ciel était clair.

Au cours des calculs il faut tenir compte des conditions locales, de la construction dans son ensemble et de ses ovrages d'art.

TABLE 1

				DONNÉES D'ENTRÉE											DONNÉES CALCULÉES [KCAL / m^2 h]							DIFFÉRENCE		
				KILOMÈTRAGE	Q m^3/s	B m	I	TEMPS	t_v °C	t_{vz} °C	W m/s	e mm	n	t_z °C	S_{1+2}	S_3	$S_{(4+5)a}$	S_6	S_7	ΣS	t_{v1} °C	$t_{v2}-t_{v1}$ °C	$\Sigma S' - \Sigma S$	$S'_{1+2} - S_{1+2}$
				1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1	13.1 1970 DATE DE MESURAGE	50.5° LATITUDE NORD	-21.00° DÉCLINATION	2,3 8,0 15,2	1,10 1,10	3,1 2,9	0,0007 0,0009	8,30 10,30 13,05	0,40 0,50 0,50	-2,4 -2,0 -1,1	0,2 0,2 0,2	3,7 3,9 4,1	1,0 1,0 1,0	4,1 2,1 4,1	-24 -20	-22 -22	34 88	17 17	2 3	22 66	0,4 0,8	0,1 -0,3	15 -66	21 -12
2				2,3 8,0 15,2	1,00 1,00	3,0 2,9	0,0007 0,0009	9,30 11,30 14,05	0,50 0,50 0,50	-2,0 -1,5 -1,2	0,2 0,2 0,2	3,8 3,9 3,9	1,0 1,0 1,0	4,1 2,1 4,1	-22 -19	-22 -22	65 88	17 17	2 3	40 67	0,7 0,9	-0,2 -0,4	-40 -67	-7 -13
3				2,3 8,8 15,2	1,10 1,10	3,1 2,9	0,0807 0,0009	10,00 12,00 14,35	0,50 0,50 0,60	-2,2 -1,6 -0,8	0,2 0,2 0,2	3,7 3,9 4,1	1,0 1,0 1,0	4,1 2,1 4,1	-23 -18	-22 -22	76 81	17 17	2 3	50 61	0,7 0,8	-0,2 -0,2	-50 -42	-9 4
4				2,3 15,2	1,10	3,0	0,0008	10,30 15,05	0,50 0,60	-2,0 -0,9	0,2 0,2	3,8 4,1	1,0 1,0	3,3 4,1	-19	-22	87	20	2	68	1,2	-0,6	-58	-5
5				2,3 15,2	1,10	3,0	0,0008	11,00 15,35	0,50 0,60	2,0 -0,8	0,2 0,2	3,8 4,1	1,0 1,0	3,3 4,1	-19	-22	81	20	2	62	1,1	-0,5	-52	-4
6				15,2 19,9	1,10	3,1	0,0005	11,00 13,30	0,30 0,40	-1,2 -1,1	0,2 0,2	3,9 4,0	1,0 1,0	4,1 2,8	-16	-21	90	20	1	74	0,6	-0,2	-47	10
7				15,2 19,9	1,10	3,1	0,0005	11,30 14,00	0,30 0,40	-1,2 -1,1	0,2 0,3	3,9 4,0	1,0 1,0	4,1 2,8	-16	-21	88	20	1	72	0,6	-0,2	-45	10

NOTES : Q - DÉBIT DANS LE CANAL ; B - LARGEUR AU NIVEAU DE L'EAU ; I - PENTE LONGITUDINALE ;
t_v - TEMPÉRATURE DE L'EAU ; t_{vz} - TEMPÉRATURE DE L'AIR ; W - VITESSE DU VENT ; e - TENSION RÉELLE DES VAPEURS AUDESSUS DU NIVEAU DE L'EAU ; n - NÉBULOSITÉ EN FRACTIONS DU CHIFFRE 1 ;
t_z - TEMPÉRATURE DE L'EAU DE DRAINAGE ; S_{1+2} - PERTE DE CHALEUR PAR CONVECTION ET ÉVAPORATION ;
S_3 - PERTE DE CHALEUR PAR RADIATION ; $S_{(4+5)a}$ - RADIATION SOLAIRE ABSORBÉE ;
S_6 - ECHANGE DE CHALEUR ENTRE LE LIT ET L'EAU ; S_7 - CHALEUR PROVENANT DU TRAVAIL DES FORCES DU FROTTEMENT INTERNE ; t_{v1} (t_{v2}) - TEMPERATURE CALCULÉE (MÉSURÉE) DE L'EAU DANS LE PROFIL FINAL ;
S' - VALEURS MÉSURÉES

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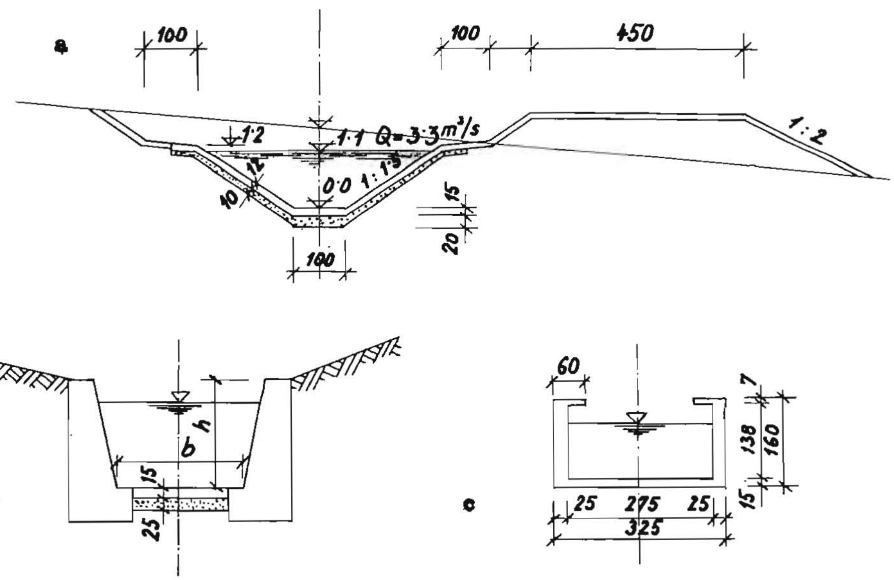

Fig. 1. Les profils transversals du canal d'amenée

a - le profil-type d'une forme trapézoidale
b - le profil-type rectangulaire:
km 12,8 - 13,3 : b = 3,80 m, h = 2,80 m
km 15,7 - 16,0 : b = 2,40 m, h = 1,40 m
c - la coupe de l'aqueduc

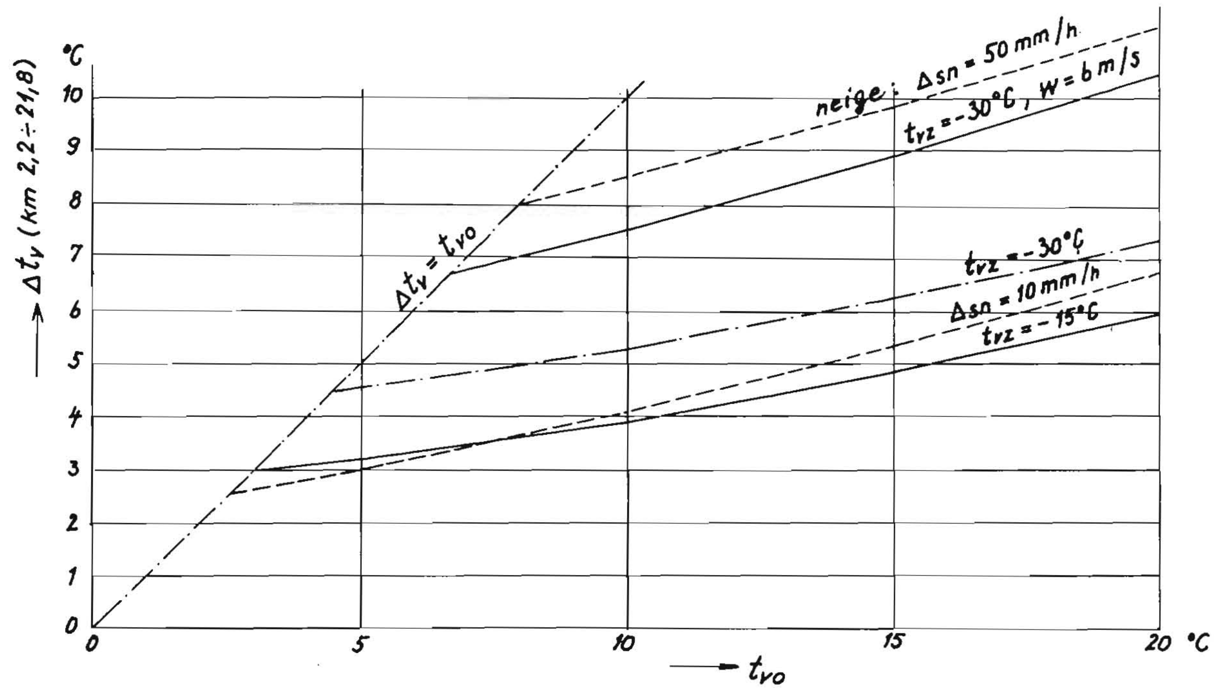

Fig.2. La dépendence du refroidissement total de l'eau Δt_v dans le canal d'amenée (km 2,2 - 21,8) à sa température initiale t_{vo}.

A (la nuit claire): $n=0, t_{vz}=-15$ °C (-30 °C); e=3 mm Hg, w=2 m/s (6 m/s), $Q = 2,2$ m^3/s, v = 1,16 m/s

B (la nuit à une chute de neige): n = 1, $t_{vz} = -5$ °C, w = 4 m/s, Δ_{sn} = 10 mm/h(50 mm/h)

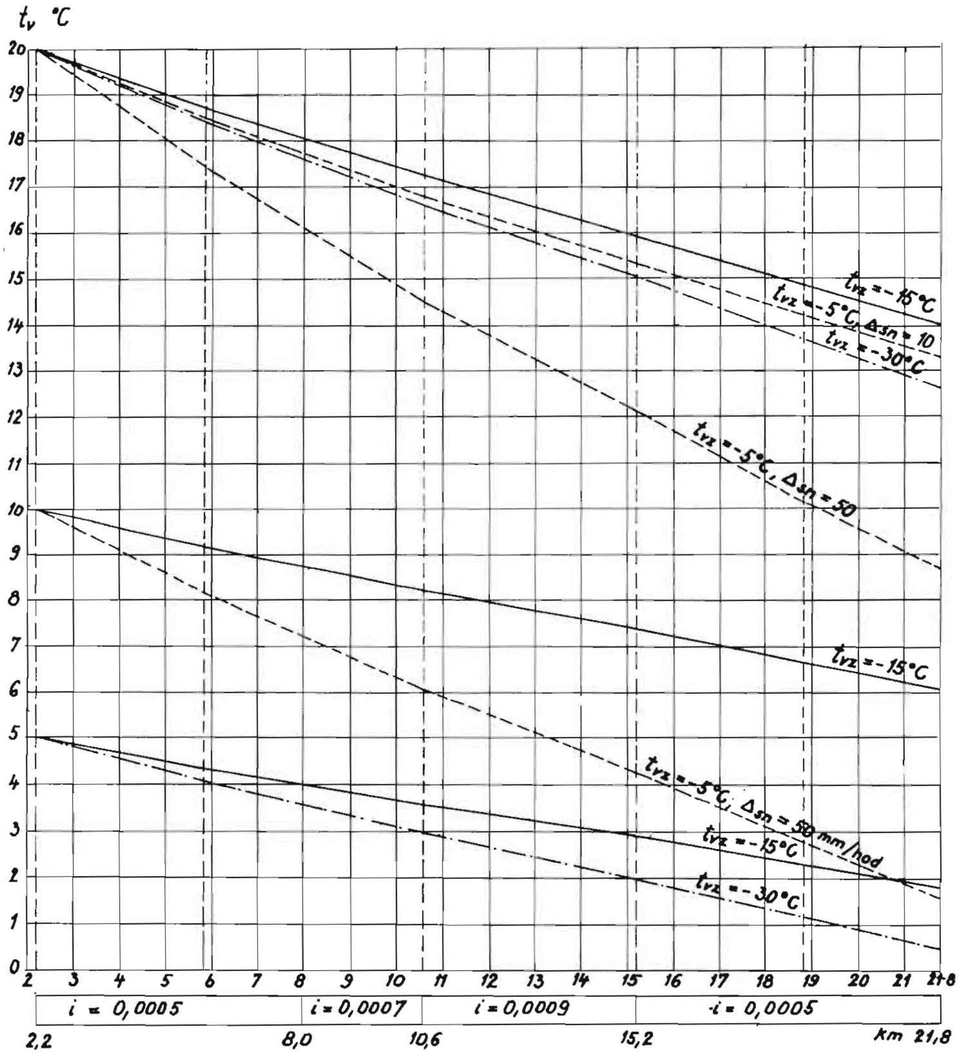

Fig. 3. Les profils longitudinals thermiques du canal d'emenée

/ Legende v. la fig. 2./

ICE SYMPOSIUM 1970
REYKJAVIK

INVESTIGATIONS INTO FRAZIL, BOTTOM ICE AND SURFACE ICE FORMATION IN THE NIAGARA RIVER

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Flow retardation due to ice in the Niagara River is at times considerable, and causes large reductions in power production. Investigations to obtain an understanding and to establish a practical relationship between ice production and meteorological parameters began in 1966. The initial surveys and observations are described. Bottom ice formations were found to cover most of the river channel. Direct observations of the evolution of some ice forms are summarized. Surface ice crystals formed when the average water temperature was still above the ice point and were drawn down into suspension by the flow turbulence. When the water temperature becomes 0°C or below, the crystals adhere to underwater objects contributing largely to the underwater ice formations. Positive information as to whether ice forms directly on underwater objects has not been acquired.

A certains moments le débit du Niagara est considérablement diminué par la glace, ce qui réduit sensiblement la production d'énergie hydroélectrique. Dès 1966 on étudie cette glace en vue d'établir un rapport pratique entre sa formation et les paramètres météorologiques. L'exposé décrit les études initiales et les observations effectuées. On a constaté que la glace de fond couvre la plupart du lit du fleuve. L'évolution de certaines formes de glace observée est résumée. Les cristaux de glace se forment à la surface quand la température moyenne est encore au-dessus du point de congélation et la turbulence de l'écoulement les entraîne en suspension vers le fond. Quand la température de l'eau descend à 0°C ou au-dessous, les cristaux adhèrent aux objets submergés, ce qui contribue largement à la formation de la glace sous l'eau. D'autre part on n'a pas encore obtenu des indices positives que la glace se forme directement sur les objets submergés.

INTRODUCTION

The investigation of Niagara River Ice with particular reference to the formation and action of frazil and anchor ice began during the winter of 1966-67 and has continued yearly since then. This project was proposed by Ontario Hydro as an Ontario and Canadian contribution to the International Hydrologic Decade.

The study is confined to observations of ice in the Upper Niagara River between the outlet of Lake Erie and the International Control Structure above the Falls (Figure 1).

The Niagara River is the waterway connecting Lake Erie to Lake Ontario. The flow velocity ranges from 3 ft/sec (1 m/sec) to 10 ft/sec (3 m/sec) in the 18 mile (29 km) reach. Maximum depth is around 30 feet (9 m), with an average depth of approximately 12 feet (4 m). The average winter flow is approximately 190,000 cu ft/sec (5400 cu m/sec). No natural ice cover forms on the river under present conditions. The normal maximum and minimum air temperatures are: December 35.1, 20.9°F (1.7, -6.2°C); January 31.8, 18.8°F (-0.1, -7.3°C); February 31.1, 16.9°F (-0.5, -8.4°C); March 38.9, 23.7°F (3.8, -4.6°C) respectively.

The purpose of this paper is to serve as a progress report describing the methods and equipment used to obtain the observations presented. A description of the instrumentation used in this project is presented in the paper by Arden[1].

Project Goal

On many cold clear nights each winter the apparent outflow from Lake Erie is suddenly reduced 15 to 20 per cent. One of the ultimate goals of the project is to establish a means of forecasting the flow retardation due to ice in the Niagara River based upon the meteorological parameters.

INITIAL ICE OBSERVATIONS

The initial ice surveys of the project consisted of observing the river from a helicopter the mornings after a cold clear night. Great masses of white-capped, brown chunks of bottom ice ranging in size from a few inches to 3 or 4 feet (approximately 5 cm to 1 m) in diameter were observed floating on the surface (Photograph 1) three to seven hours after sunrise.

Bottom Ice Locations

At the beginning of the investigations the extent of the bottom ice formation in the river was not known. The question arose as to whether it uniformly coated the whole river channel or just certain areas. In attempting to answer these basic questions an experiment was designed to determine whether subsurface ice would form on an object which could be retrieved and examined. It was reasoned that the bottom ice-producing areas could be determined by amounts of ice that formed on them. After some experimentation the arrangement as illustrated in figure 2 and shown in photograph 2 was developed. The trays were 30 inches (75 cm) square having an angle iron frame with two inch (5 cm) mesh screen covering the frame.

A forty-five foot (13.6 m) twin propeller, diesel powered boat with a closed internal cooling system built specifically for ice breaking was used as a survey boat.

Results of Collector Tray Program

In general ice formed on all the trays at all the locations in depths to about 20 feet (6 m) during January and February when the air temperature was 20°F (-6.7°C) or below and the net radiation was -5 langleys or more except in the area above the Peace Bridge (the outlet of Lake Erie) and a 500 foot (150 m) band along the US mainland. It is believed the reason for the latter anomaly is that there was a band of relatively warm water (industrial effluent) flowing down the US shoreline.

The large quantities of porous ice that were found on the trays (Photograph 3) initially created the impression that this was anchor ice forming on the spot. The large mats (Photograph 4) of porous ice that were observed rolling to the surface in various reaches of the river after a cold clear night established further the idea that the collector trays gave a representative sample of the bottom ice accumulation.

Description of the Ice Formations

The ice found on the collector trays was composed of flat crystals or platelets roughly oval in outline and usually not more than several inches (3 to 4 cm) across. The formation on the individual wires of a tray initially was radial, that is the platelets were approximately all parallel to each other and perpendicular to the wire. The polypropylene rope leading from the tray to the marker float was usually similarily coated with ice platelets, forming a cylinder up to 18 inches (45 cm) in diameter (Photograph 5). As the formation grew, the whole collector tray became a block of porous ice (Photograph 3). Dissecting a portion of the buildup sometimes showed that the crystals of the inner portion of the formation were much coarser than the outer (Photographs 6 & 7). Sand and gravel were interspersed throughout the whole mass of ice. On the wire screen of the collector tray unique formations such as ice platelets radiating from central points like petals of a flower (Photographs 8 & 9) and spherical formations the size of tennis balls (Photograph 10) were sometimes present.

DIRECT ICE OBSERVATIONS

The production of ice in the Niagara River is cyclic and not a continuous process during the winter months. The River temperature usually rises to 0.10 to 0.40°C each day due to heating by the sun. Rapid cooling takes place when the sun sets, the water temperature falling to 0°C or below in 4 to 6 hours when the air temperature is approximately 20°F (-6.7°C) or below. Supercooled water temperatures as low as -0.05°C are quite common.

Equipment

In order to obtain an understanding of ice formation in the river, the evolution of the various forms of ice was studied by anchoring the boat in various areas of the river. An underwater light was constructed, using 4 standard 300 watt 110 volt AC lamps (Photograph 11), in order to view any impending underwater phenomena.

The water temperature at the local observing sites was monitored by using a two point thermograph, accuracy ± 0.01°C and two electric open scale (thermister, manual balance bridge) thermometers accuracy ± 0.005°C. The water temperature at both ends of the 18 mile (29 km) reach of river were continuously monitored throughout the ice season by two recording thermographs.

In order to determine the presence of the small crystals, a strainer was constructed (Photograph 13). The apparatus consisted of a 1 x 1 x 1 foot (30 x 30 x 30 cm) box made of sheet metal with two ends open so that water could flow through. One open end was arranged so that framed screens (1 mm apertures) could be inserted and removed; the other end was fitted with a hinged door. The method of operation consisted of lowering the strainer with screen in place and opening the door for a short period of time, closing it, then retrieving the strainer and finally examining the screen for ice crystals.

The observation procedure was to monitor the local water temperatures, starting at dusk, when the water was still above zero and continuing sometimes until dawn. The full evolution of river ice production, surface and subsurface, was observed for a great range of meteorological conditions.

Observations

Invariably it was observed that if the air temperature was 20°F (-6.7°C) or less and the wind velocity was 10 to 15 miles per hour (4 to 6 m/sec) or greater, small ice crystals would form on the surface after sunset even though the average river temperature was still 0.20 to 0.30°C. It was reasoned that the abstraction of heat from the surface goes on at a rate much faster than can be maintained from the interior of the mass, consequently the top layer reaches the ice point and produces ice. The crystals, initially small and irregular in outline (Photograph 12) were observed being drawn into suspension in large quantities by the flow turbulence.

Subsurface ice - This concentration of underwater "surface ice" existed for about one hour after the average water temperature reached zero, thereafter diminishing in quantity. It is thought that the rate of surface ice growth increased when the average water temperature reached zero, due to the elimination of the warmer water below and its tempering effect. The surface ice crystals subsequently became larger and froze together forming bigger pieces, having a greater buoyancy; consequently the tendency to be drawn into suspension was reduced. At this time then there is a reduction in the quantity of "surface crystals" in suspension. When the water temperature reached zero the ice in suspension was observed to adhere to other platelets and underwater objects. Small packs of ice crystals several inches (approximately 5 cm) in diameter were observed in the underwater light beam to flow in suspension, then upon striking a down current rolling or bouncing along the bottom for a few feet (0.5 to 1 m) becoming coated with sand or gravel and then drifting up into suspension again. Several packs of ice were observed to strike one of the collector trays partly sticking and partly breaking away. No ice was observed to stick to the river bottom in this particular instance. Subsequently it was found that bottom ice forms only on rocky bottoms and not on sandy bottoms due to the lack of any anchoring effect of the latter.

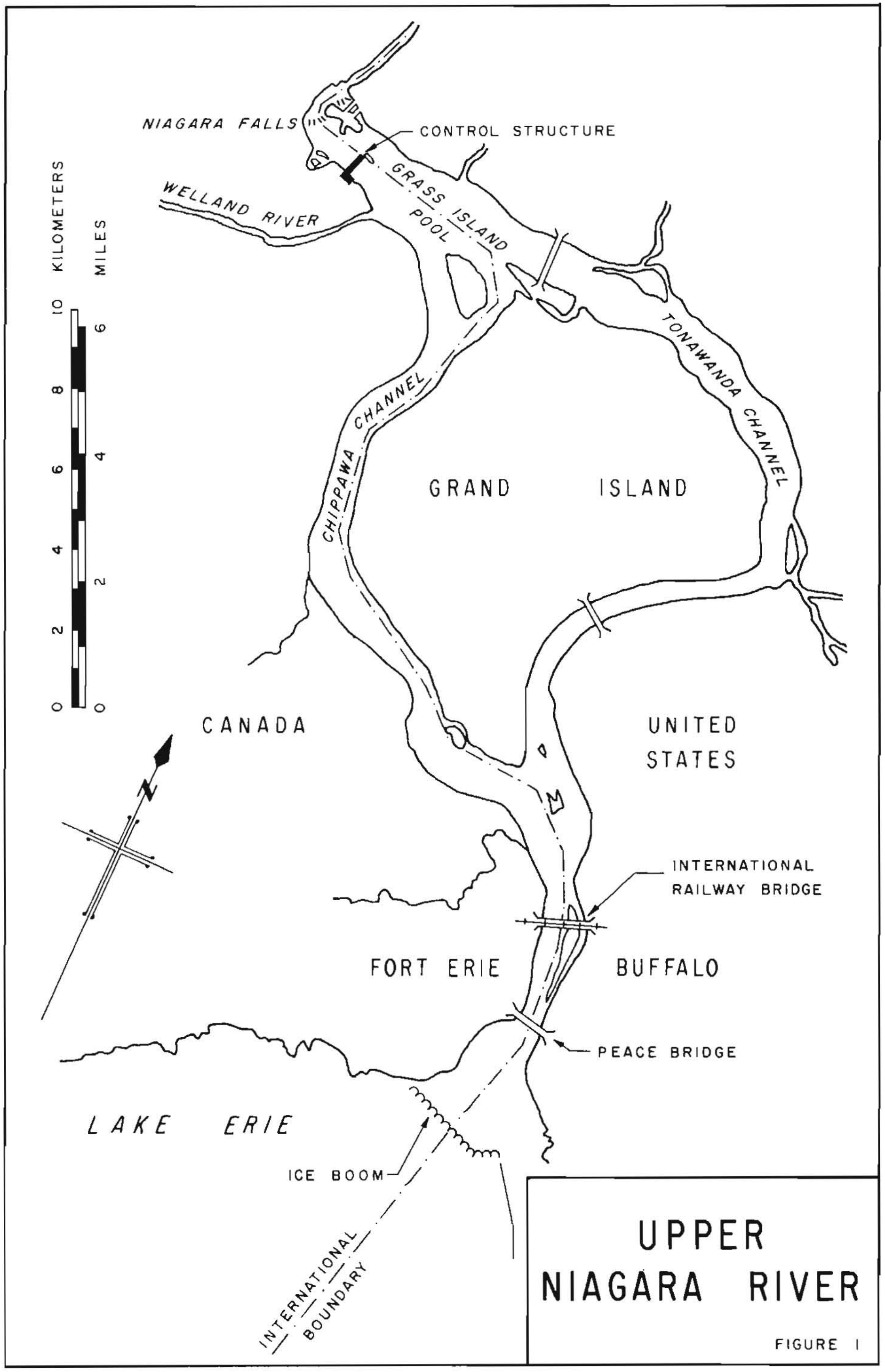
Frazil - As soon as the average water temperature reached 0°C, a unique form of ice crystal appeared, namely the frazil discoid, the concentration increasing as the water became supercooled. The analogy of a driving snowstorm below the water surface describes this phenomena exactly. The phenomenon was unmistakable once it started; the ice crystals were perfect discs 1 to 10 millimeters in diameter and about 0.1 to 0.5 millimeters in thickness (photograph 14). Many of these frazil crystals were observed on the river surface stacked together in an overlapping fashion forming small raft-like structures. Examining these conglomerates of frazil revealed that a minor transformation had taken place. The crystals were no longer disc-like, many of the crystals had become enlarged, the edges sometimes being sawtoothed in outline. This enlargement is thought to be the result of the natural growth of surface crystals. During the peak of the frazil storm, ice crystals began to collect on the rocks of the river bottom (Photograph 15). Certainly from these particular observations there was little doubt that the bottom buildup was caused by frazil and surface ice crystals (Photograph 16). However, subsequent examinations of the initial coating of ice on submerged objects has created doubt as to whether all subsurface ice formations are caused by these crystals in transit. Studies are still underway to answer this unsettled question.

CONCLUSION

These initial surveys and observations revealed some of the characteristics of the ice in the river, but more importantly they generated questions and ideas which set intermediate goals. Accordingly, programs have been attempted to measure the quantity of frazil in suspension during a variety of meteorological conditions, to measure the rate of growth of bottom ice and to determine whether anchor ice is fact or fiction. The term anchor ice is reserved to describe the particular sort of ice which forms in situ. It is planned to present the results of these programs sometime in the future. Preliminary studies are presently under way to correlate the meteorological parameters with the production of surface ice and river flow retardation.

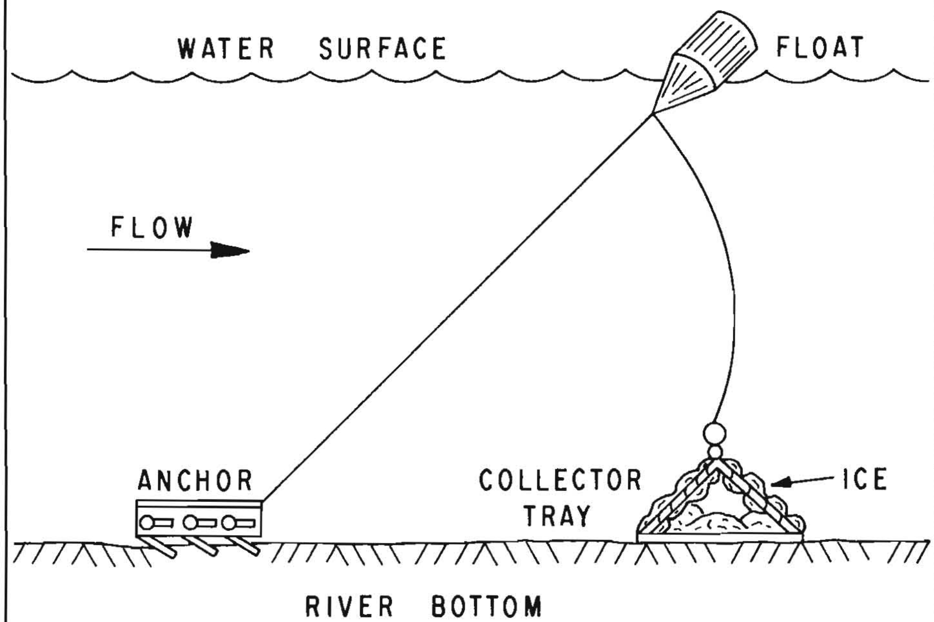
Reference

1. Arden R.S. - Instrumentation for Ice Investigations in the Niagara River, 1970



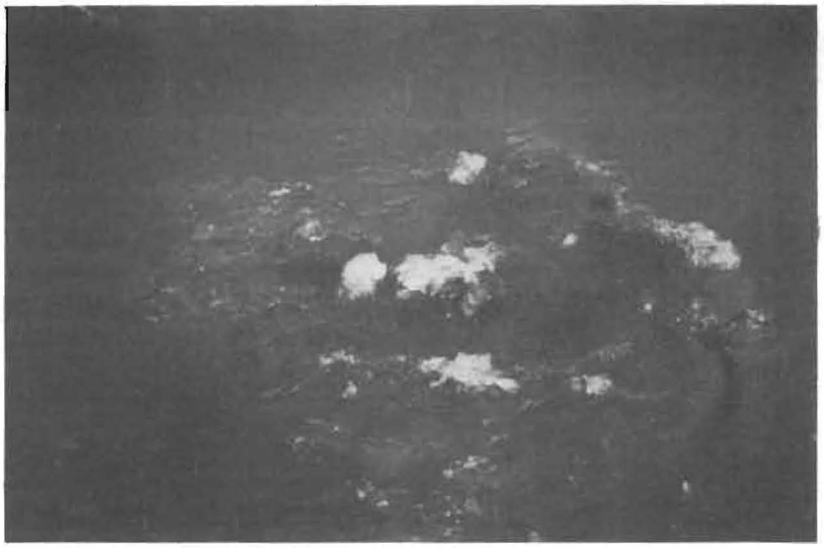
UPPER NIAGARA RIVER

FIGURE 1



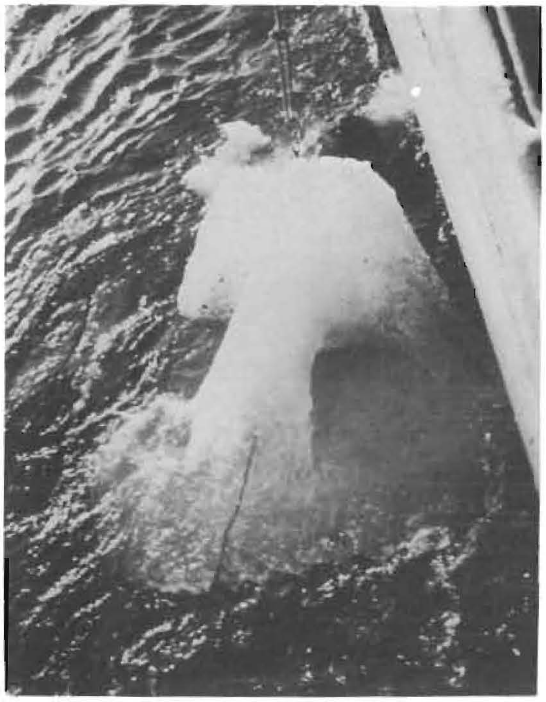
COLLECTOR TRAY

FIGURE 2



Photograph 1: A piece of bottom ice floating on the surface showing the load of gravel picked off the bottom.

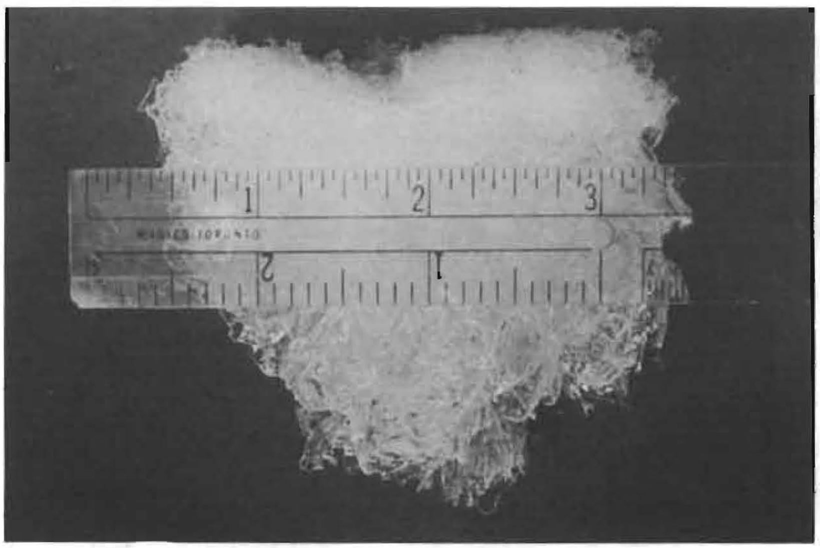
Photograph 2: The collector tray equipment used to detect the presence of bottom ice.



Photograph 3:
A mass of porous ice that formed on the collector tray placed on the river bottom overnight.

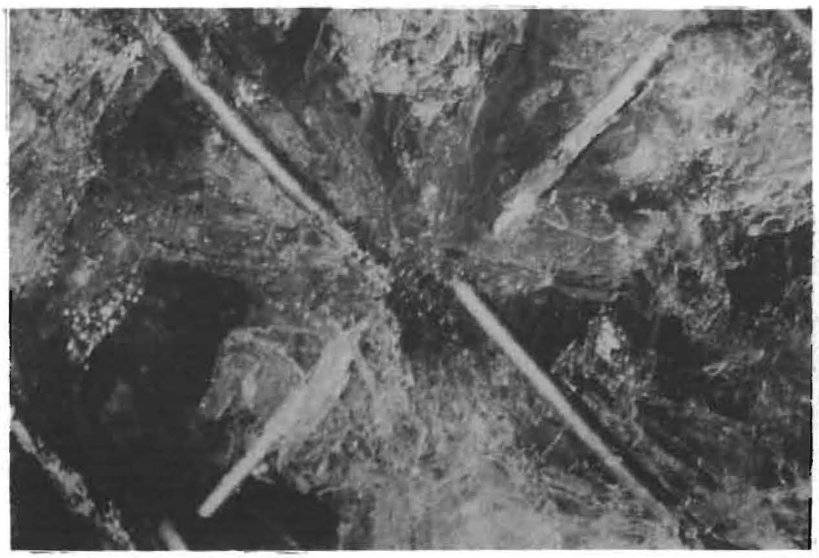
Photograph 4: This matt of bottom ice was observed rising to the surface after a cold clear night. The thickness is approximately 8 to 12 inches (20 to 30 cm).

Photograph 5: A view of the ice buildup on a polypropylene rope that was submerged for about 45 minutes in supercooled water. The formation is described as radial, the crystals being roughly perpendicular to the rope.

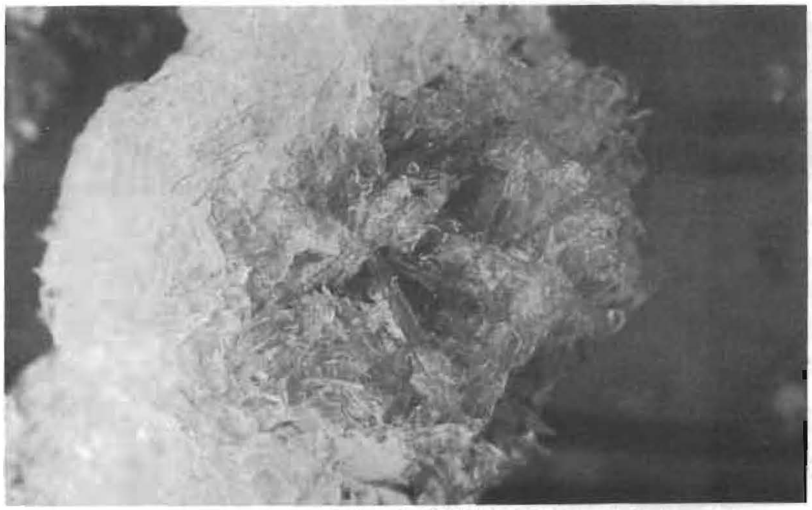


Photograph 6: A chunk of ice cut out of the mass that formed on the collector tray. Note the difference in gradation, the fine crystals formed last.

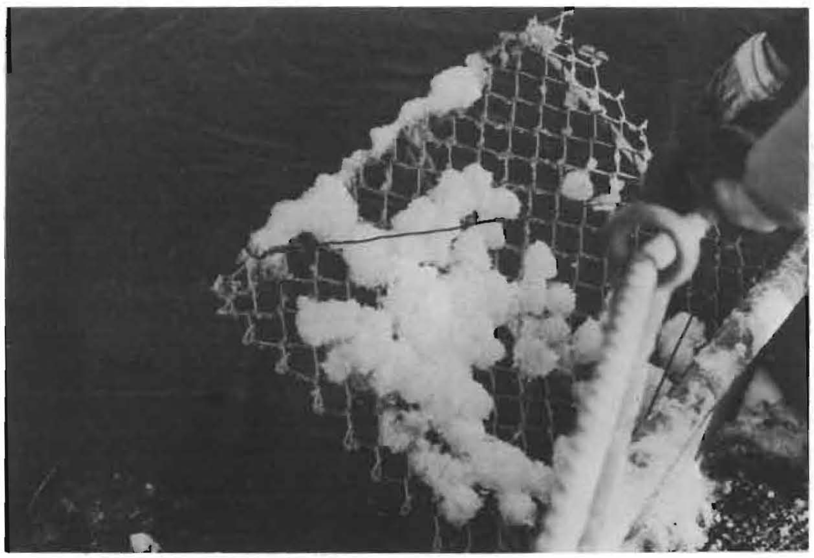
Photograph 7:
Ice on a collector tray showing the coarse crystal formation at the bottom and the fine, granular formation at the top.



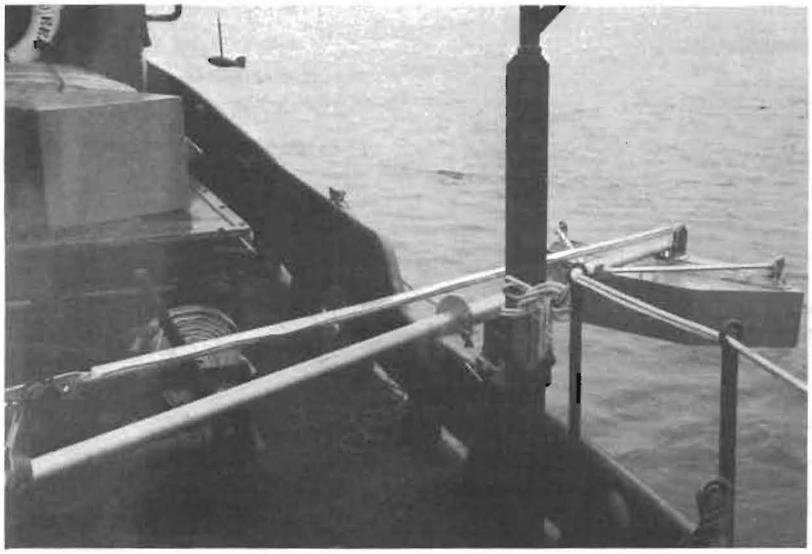
Photograph 8: The bottom view of a collector tray showing the "flower-like" formations that formed at the screen junction points.



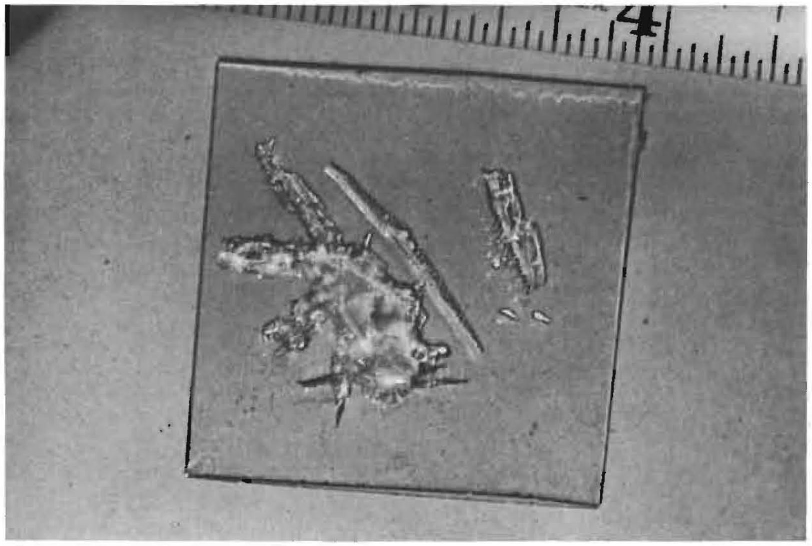
Photograph 9: **A view of the "flower-like" formation that formed on the collector tray screen. Note the frazil discoids throughout the formation.**



Photograph 10: **"Ice Balls" shown just as they formed on the collector tray screen while it was on the river bottom overnight.**

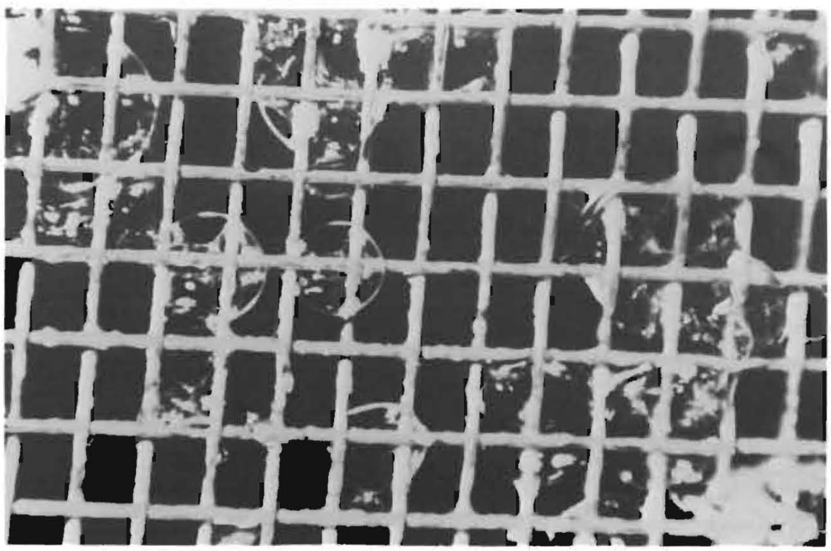


Photograph 11: The underwater light mounted on the side of the survey boat.



Photograph 12: Typical ice crystals strained from the surface of the river. The sawtoothed edges were very common.

Photograph 13:
A rear view of the strainer used to detect the presence of ice in suspension. A door at the front prevented the collection of unwanted ice.



Photograph 14: Frazil discoids shown on the strainer screen (screen spacing approximately 1 mm).

Photograph 15: An underwater view of the ice formations on the River bottom. Thickness is approximately 4 to 6 inches (10 to 15 cm).

Photograph 16: A stone retrieved from the river bottom showing ice crystals that have adhered to it.

Discussion by Mr. C.R. Neill, Research Council of Alberta.

1. Does the author consider that the so-called "anchor ice" usually originates from frazil ice that sticks to the bottom.

   Author's reply: It would appear that a large portion of the ice forming on the bottom generally originates by a process of adhesion of the transient ice crystals to the rocks. However ice has been seen to grow on rocks when there has been no apparent sign of frazil in the water. We have obtained photographs of ice formations that cannot readily be explained by the adhesion process. Obtaining a conclusive answer to this question is one of the primary goals for the remainder of our project.

2. Would the author like to comment on the "anchor ice" definition included in the proposed terminology presented by Dr. Kivisild on behalf of the Committee on Ice Problems?

   Author's reply: Until stronger evidence one way or another is presented, I believe the definition for "anchor ice" should stand as proposed.

ICE SYMPOSIUM 1970
REYKJAVIK

MAN-MADE ICE STRUCTURES FOR ARCTIC MARINE USE

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SYNOPSIS

The thermal aspects of man-made ice structures for arctic offshore use are considered. A two-dimensional ice model grounded in various depths of sea water is presented. Isotherm and time-temperature plots indicate that such a structure is thermally feasible for the geometry and climatic conditions considered. Possible areas of weakness are commented upon and recommendations for further areas of study are made.

RESUME

Cette communication présente les aspects thermaux d'une île artificielle construite en glace pour servir de platforme au large de l'Océan Arctique. On considère un modèle mathematique à deux dimensions de l'île échouie à des profondeurs variées. Les courbes isothermes et temps-temperature indiquent que telle platforme est possible dans un sens thermal pour la géométric et les conditions climatiques données. On discute les endroits faibles et l'on fait des recommendations pour les études futures.

INTRODUCTION

The recent discovery of petroleum deposits on the north coast of Alaska and in the Canadian Arctic has focused attention on possible marine transportation to markets in Europe, North America and Asia. The two successful trips of the MANHATTAN in the fall of 1969 and spring of 1970 have strongly pointed to the feasibility of such marine routes. However, for these routes to be a practical reality offshore structures in considerable depths of water and capable of withstanding the loads imposed on them by the ice will be required. Structures will also be required for the exploration and development of the potentially petroleum-rich arctic continental shelf.

The recent work carried out for the Marine Division of the Humble Oil Company by the University of Alaska involved two fragments of a tabular ice island (Figure 1) which grounded in 15 fathoms of water north of Prudhoe Bay, Alaska (McKay[1]). The larger of these fragments successfully withstood the sea ice action through the major portion of the ice year, thus suggesting that an offshore structure is capable of withstanding the ice loads imposed on it by virtue of weight and bulk of structure alone. A conventional structure utilizing steel and steel reinforced concrete could no doubt be designed to be equally successful but severe economic constraints would be placed on such a structure by the logistics involved, climatic conditions and the very short construction and navigation season.

The arctic region offers the following unique possibility: far more thermal energy can be lost during the cold winter period than could be gained during the warm summer. Thus, the concept of utilizing the natural heat sink to freeze a monolithic structure in which the cementing agent is the sea water itself deserves some consideration.

The use of suitably reinforced ice as a structural material is not a new concept. Perutz[2] describes an aircraft carrier or bergship constructed of a mixture of frozen water and wood pulp which was under consideration during World War II. Dykins[3] comments on the strengthening of an existing ice sheet through flooding of the upper surface with sea water. A sea-ice runway produced by this method is described by the Bureau of Yards and Docks[4]. More recently Behlke and McKay[5] consider the use of artificial sea-ice structures for Alaskan port facilities in regions where landfast ice exists.

GROUNDED ICE ISLANDS

FIGURE 1

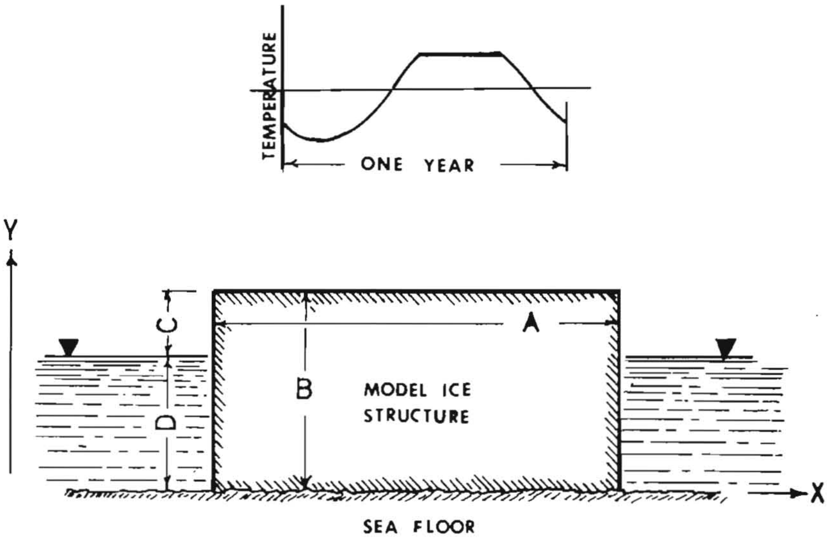


FIGURE 2

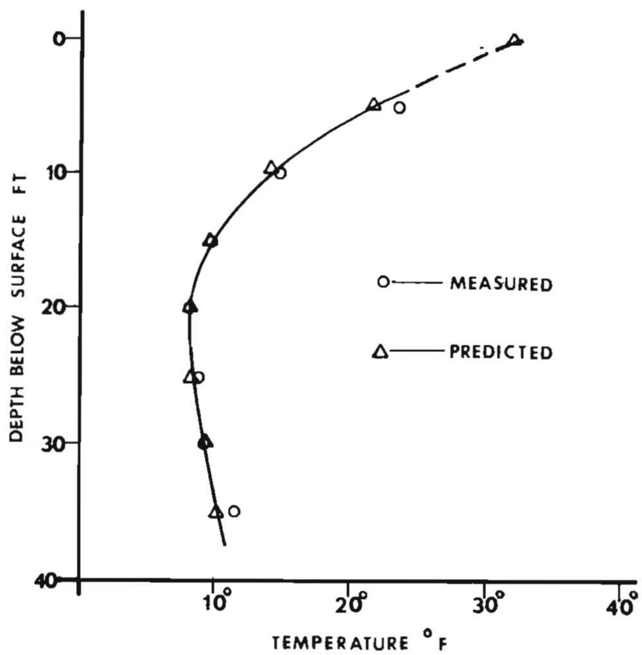


FIGURE 3

2.9

The immediate basic questions which have to be answered before serious consideration to the possible use of man-made ice structures for marine use are as follows:

1) Would such a structure grounded in sea water and subjected to annual arctic climatic variation be thermally stable and not melt away?
2) How could such a structure be built up and in what geographic areas could the natural heat sink be used to advantage?
3) What stresses can a man-made ice structure withstand?

This paper is addressed to the thermal stability of man-made ice structures in an arctic marine environment and an attempt is made to answer the first question posed here.

THERMAL CONSIDERATIONS

Ice Structure Model

Consider the model ice structure shown in Figure 2. The structure is assumed two dimensional with width A and total height B. The lower portion of the structure is in contact with the sea floor at depth D below the sea water surface. The structure protrudes from the water surface into the atmosphere to a height C. The surfaces either receive or lose thermal energy depending on the time of year under consideration.

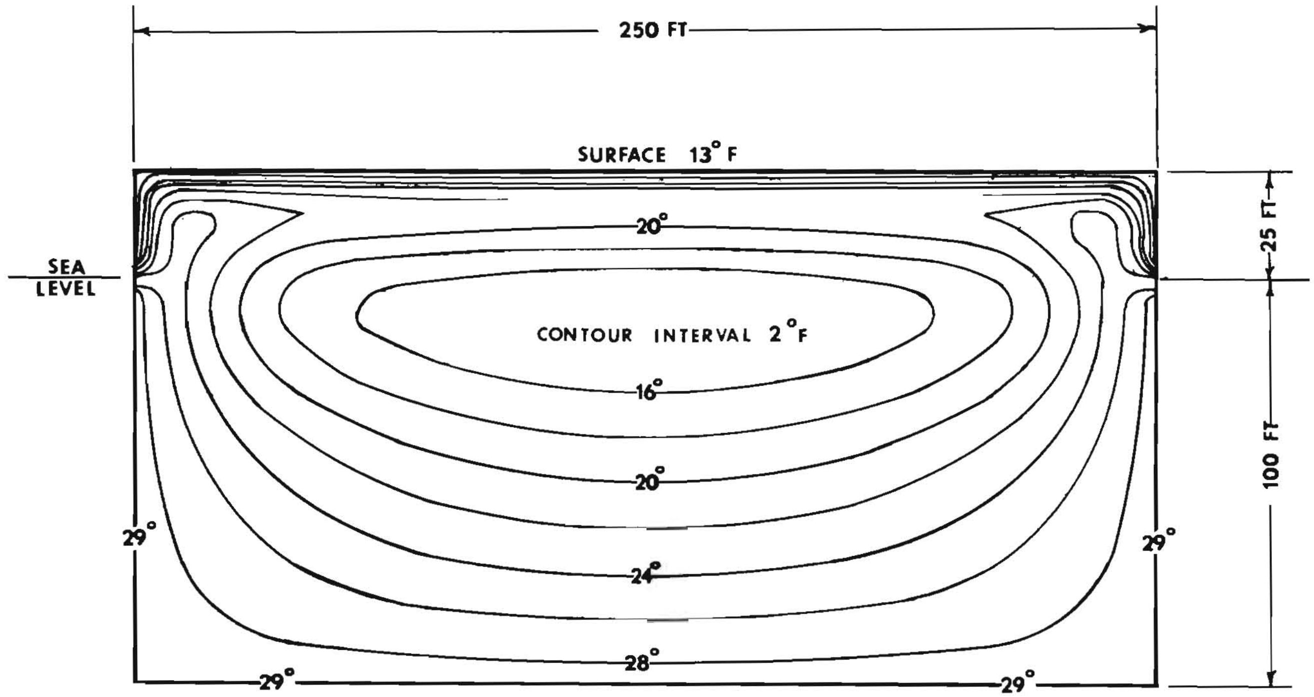
Difference Equation

A heat balance on a small volume element within the model may be expressed as follows:

$$\frac{\partial T}{\partial t} = \alpha \left[\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2}\right] \tag{1}$$

where T is the volume element temperature, t is the time, and α is the thermal diffusivity $\alpha = \frac{k}{\rho c_p}$ (the thermal conductivity divided by the product of the mass density and heat capacity), with all properties expressed in compatible units.

Eq. (1) may be expressed as a difference equation by use of a finite difference approximation. The difference equation may then be used to predict the future temperature of any node within the model in terms of discrete spatial and time steps. The size of the spatial and time steps is chosen to assure a stable solution for the equation. With suitable property values for the material making up the model and appropriate temperatures existing at the boundaries, the time-temperature history of any position within the model may be obtained.



ISOTHERM PLOT ICE STRUCTURE
OCTOBER 28

FIGURE 4

2.9

Boundary Conditions and Property Values

Temperatures of the atmosphere were assumed to be sinusoidal with time (Johnson and Hartman[6]), and the surface temperatures of the model exposed to the atmosphere were assumed to approach these values for that period of time when the temperatures were below 32°F When the sinusoidal variation indicated temperatures above 32°F the atmospheric surface temperature was held at 32°F. A typical time-temperature variation is shown on Figure 2. The values used in the solution of the difference equation were those of Barter Island, Alaska, with a mean annual air temperature of 10.4°F and amplitude 31.3°F.

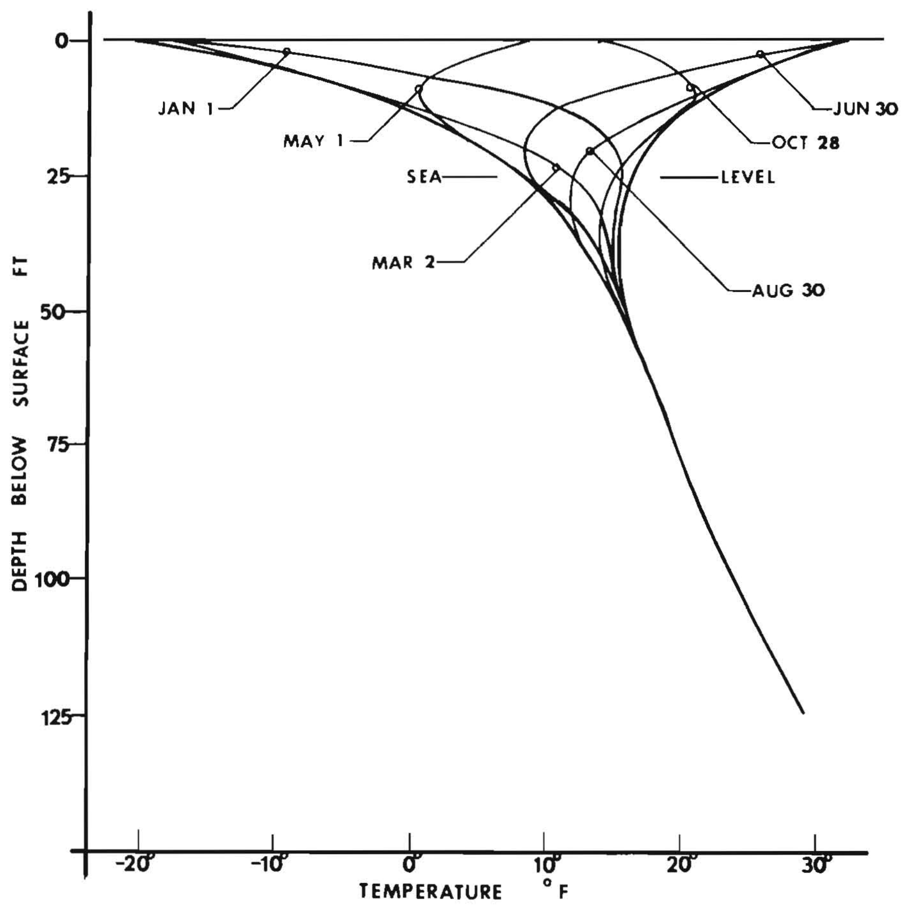
The sea water temperature was assumed constant at 29°F over the entire year and the temperature of the model at the sea-floor interface was assumed to approach this value.

The property values which determine the thermal diffusivity of the ice used for the model are all functions of temperature, and combine to cause a decrease in the diffusivity with an increase in temperature. Values used in the solution consider this temperature variation by use of the functional relationship established by James[7]

DISCUSSION

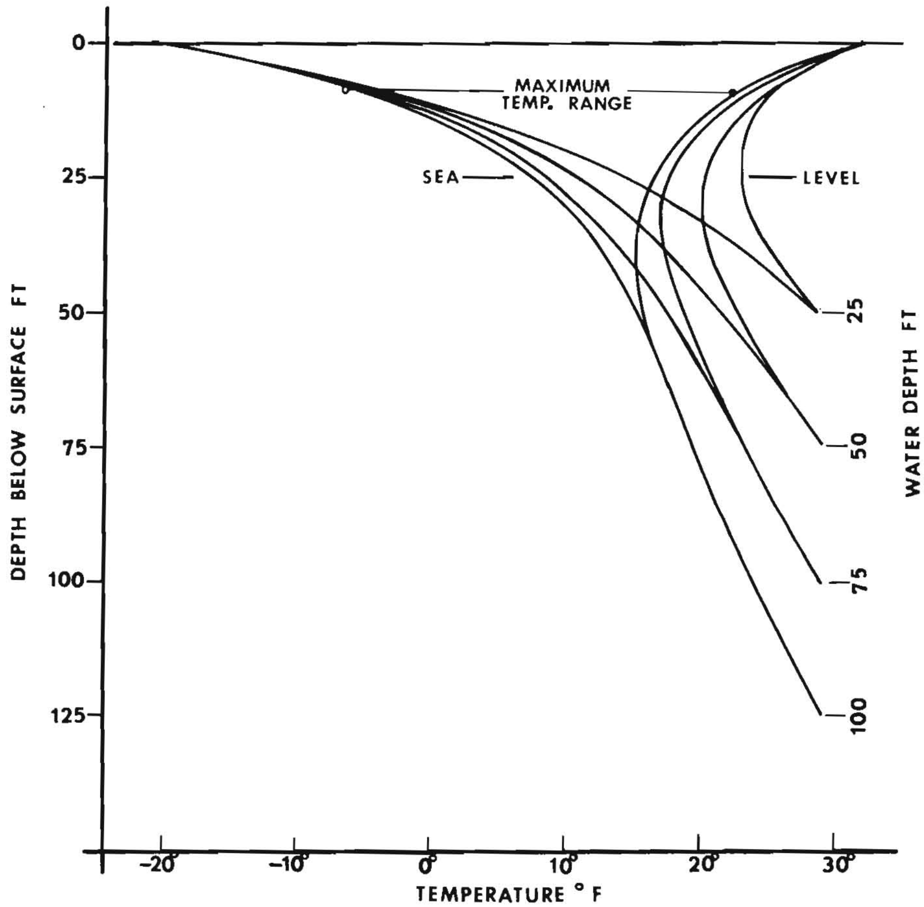
Though the boundary conditions applied to the model have been vastly simplified, they do place the model under more severe thermal conditions than a natural situation. A comparison between the temperatures predicted by equation (1) and those obtained in the field (McKay[1]) is shown in Figure 3. The close agreement strengthens the assumptions made.

Numerical results obtained after five years of temperature cycling are shown graphically in Figures 4, 5, and 6. Figure 4 represents the isotherm pattern of the model when the bulk of the model is at its highest temperature. The only region where the ice is near its equilibrium temperature with the sea water is in the lower corners. These corner regions may be considered to be the weakest regions of the model. Inspection of the isotherms within the model and consideration of the usual geothermal gradient suggest that the model ice structure could freeze into the sea floor. Consideration of the film conductance on the vertical submerged faces and the temperature pattern within the model suggests the possible freezing of additional ice on these faces. The shape of the isotherms also indicates that at no position within the model may the heat flow be considered as one dimensional as would be the case of a continuous ice sheet covering the sea surface.



TEMPERATURE WITH TIME AND DEPTH

FIGURE 5



TEMPERATURE ENVELOPES FOR DIFFERENT WATER DEPTHS

FIGURE 6

Temperature fluctuations at the vertical centerline for a structure with the same dimensions as Figure 4 are shown in Figure 5. The temperature profiles and the envelope containing them are shown for six equal time intervals during the year. It will be noted that the greatest temperature variation takes place in the uppermost portion of the model structure, while at slightly below 50 feet from the upper surface the temperature is effectively constant over the entire year.

Figure 6 shows the envelopes of the time-temperature profiles at the vertical centerline for four model structures grounded in 25, 50, 75, and 100 feet of water. In all four cases the horizontal dimension is 250 feet and 25 feet of structure protrudes into the atmosphere. The envelopes give the maximum temperature range through which the ice in the structure would fluctuate at a specified depth. As the depth of water is increased the temperature at which no perceptible fluctuations exist decreases and a maximum point for the depth at which this occurs is noted. This maximum suggests the dependency of the temperature profiles on the geometry of the structure and that a thermally optimized structure would be obtained with a unique geometry for a given depth of sea water.

CONCLUSIONS AND RECOMMENDATION

Based on this relatively simple model, man-made ice structures appear to be a possibility from the thermal point of view in sea water up to depths of 100 feet and in areas with annual atmospheric conditions similar to those at Barter Island, Alaska.

The model structures for the dimensions considered do not appear to have major regions where the ice comprising the structure would be excessively soft because of high internal temperatures. However, the warmest regions, in particular those in the lower corners, deserve attention. Heat loss from the structure to the atmosphere could be enhanced, thus cooling the bulk of the structure still further and minimizing the warm regions.

Reinforcing the structure will no doubt be necessary. Caution should be exercised in the selection and placement of the reinforcing to balance or even optimize the thermal and mechanical properties of the structure. Additives to the ice in the form of aggregates may also be used to change the thermal diffusivity and resulting temperatures such that stresses resulting from thermal effects are minimized.

The question of salt migration from the sea water into the structure and the possible buildup of ice on the water-exposed faces should be studied further as should the plastic flow of ice at the bottom and the possibility of the structure freezing to the sea floor. Though the answers to these questions and others may be obtained in part through analytical and laboratory model techniques, it is strongly recommended that serious consideration be given to the construction of a fully instrumented prototype structure.

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DISCUSSION BY: K.R. Croasdale

The following question was posed by Mr. Croasdale:

With reference to the statement that an ice island could be built artificially in up to 25 feet of water; what would be the time required to achieve this? Also what equipment would be needed?

RESPONSE BY: A.R. McKay

Assuming the same ambient temperature conditions as those used in the paper and an average wind speed of 10 mph, an island could be built up in approximately 120 days. Equipment required would essentially consist of high capacity pumps and a manifold system for flooding the surface uniformly.

DISSCUSSION BY: A. Assur

The following question was posed by Dr. A. Assur:

In order to evaluate the forces which can act on ice islands it would be very valuable to know the overpressure which was available to act between the ice island and the bottom, the area on which the overpressure acted, bottom conditions (friction coefficient), and the diameter of the island perpendicular to the direction of ice forces. In addition the thickness of the surrounding ice floes should be known. Are these data or least part of them available and what are the values?

RESPONSE BY: A.R. McKay

Data obtained at the time of the first motion of one of the grounded ice islands (July 2, 1969) due to ice floe action was as follows:

Overpressure: 140 lbs/ft^2

Bottom area: 56,00 ft^2

Bottom conditions: gravel 0.8% by weight, sand 8.8%, silt 29.5%, clay 60.9%. Coefficient of friction unknown.

Width of island perpendicular to ice forces: 190 ft.

Thickness of ice floe between 3.5 ft and 4.0 ft.

ICE SYMPOSIUM 1970
REYKJAVIK

EFFECTS OF ICE ON WATER INTAKES INCLUDING THE DESIGN OF ICE FREE CHANNELS

V.V.Balanin, Assistant Professor, Water Transport Institute, Leningrad, U.S.S.R.

This paper deals with some problems of preventing harmful effects of ice on water intakes by maintaining deiced water areas in front of them owing to lifting warm bottom waters and to outside heat supplying, as well as by intensification of frazil ice discharge by means of heating the grate and forming water craters.

Dans ce rapport on examine les problèmes de la prévention des influence nuisibles de la glace sur les prise d'eau au moyen du maintient des surface d'eliberé de glace à l'aide du levage des eaux chaudes de hautes profondeurs et du conduits de chaleur de l'exterieur et aussi l'intisification du passage de la frasil à l'aide du chaufage des grilles et formation des tourbillon.

1. General Description of the Problem

Discussing water intakes in general, we may point out two principal types of effects: 1) surface ice pressure on the gates and 2) filling up the cross section of the inlet by frazil ice. The former type of effects is characteritic for the surface orifices while the latter is a characteristic for the submerged ones.

As to the surface ice pressure on the water intakes and weir gates it is due to ice expansion caused by sharp temperature rise.

This problem will be dealt with at the next sittings of our symposium. That's why the problem set forth below considers only the way of preventing this pressure.

Maintaining of the free ice water area before the gate is the most expedient solution. But as the tackling of this task is of interest that comes out of water intakes work and is the subject of the second part of the problem discussed, we shall first of all dwell here on that of maintaining free ice water areas.

The second type of effect concerns the frazil ice formation, the problem that has partly been discussed on the present symposium. That's why we shall pay attention mainly to measures ensuring free passing of frazil ice through water intakes.

2. Methods of Ice Free channel. Maintenance.

In the water areas where there is a considerable temperature diff. between the bottom and the surface area (0,5°C), maintenance of ice free channels may be accomplished either by means of raising warm deep waters, or by supplying outside heat.

Raising of warm deep waters is accomplished by pneumatic installations (fig. 1), streamforming devices (fig. 2) and pneumatic streamforming devices (fig. 3).

The sphere of expedient application of each mentioned devices is mainly determined by the thermal character of the water area section and partly by the peculiarities of devices. Pneumatic installations are advisable with considerable stretch of the water area that is to be maintained ice free at considerable depths (h 6÷7 m) and full epura of water temperature distribution through the water reservoir depth (fig.4). The advantage of pneumatic installations is that there are no troubles for running ships, rafts etc. on the water surface.

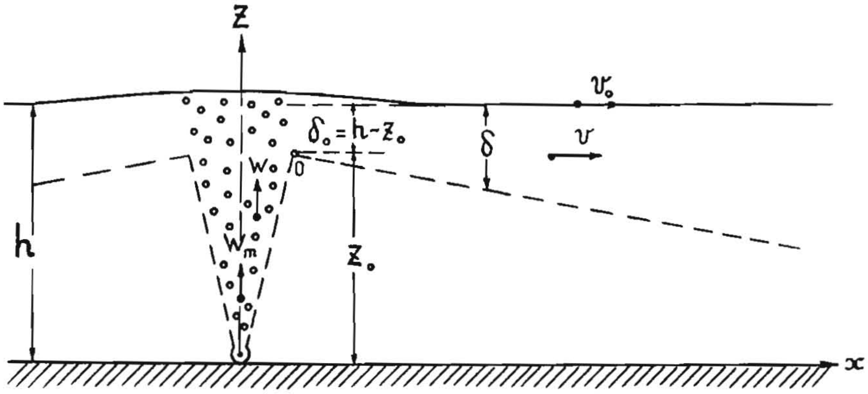

fig. 1

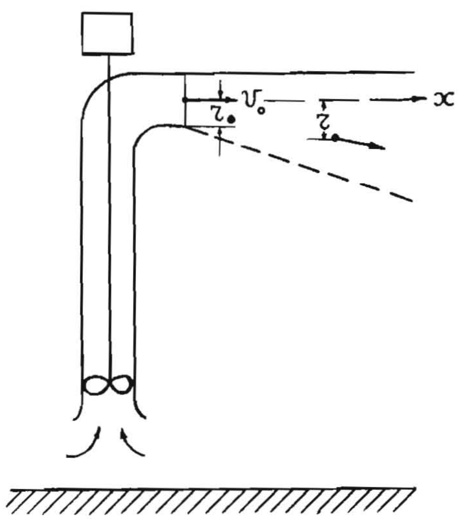

fig. 2

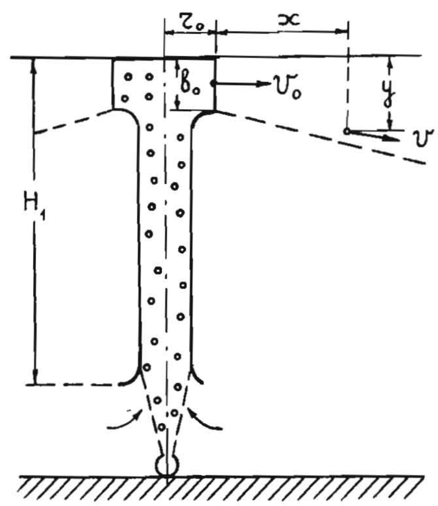

fig. 3

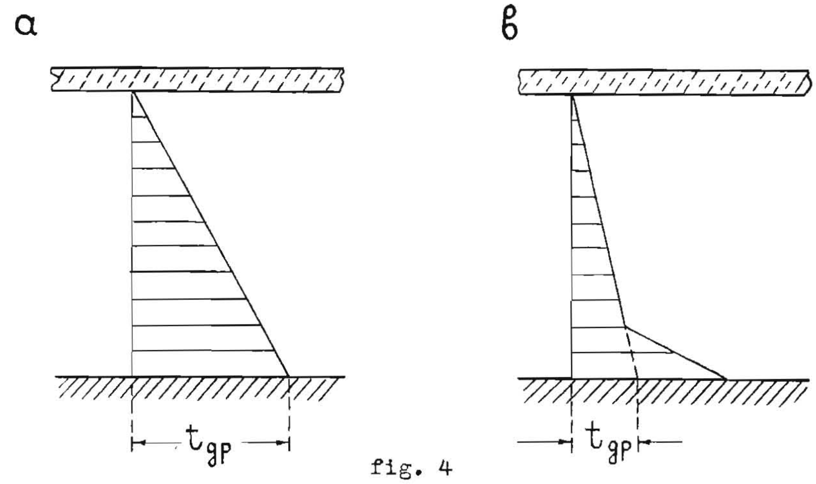

fig. 4

In these conditions, but with less extent of the water area the use of the radial-axial stream forming devices is advisable (fig. 5). The advantage of this scheme is the possibility of creating directional water stream.

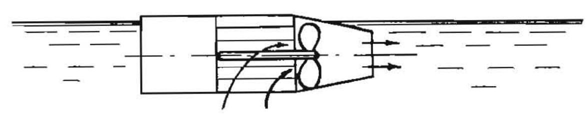
fig. 5

With sharp temperature differences of the bottom water layers and the rest of the water thickness (fig. 4), that is often in pools and other water reservoirs, having different velocities of streams at the surface and bottom, it is expendient to use pneumatic stream-forming devices or stream-forming devices with an elbow (fig. 2). In other cases considerable heat losses will take place with water transfer from bottom layers to the surface.

As studies have shown, to maintain free ice water areas, having no substantial temperature stratification over the depth it is most expedient to use heated water or steam - air mixture.

For heating water it is advisable to use a steam injector as a device having the most simple efficient design. The distribution of heated water over the bottom of the water reservoir, the surface of which is to be de-iced, may be carried out through pipelines. Experiments show, that the system of heated water distribution should be supplemented by perforated air outlet pipes ensuring the lifting of heated water to the surface.

Using stream-air mixture injector to which steam is supplied, is used, air being sucked and heated. Then steam and air escaping the orifices of the perforated pipelines r ise to the surface heating the surrounding and lifted water simultaneously. The advantage is that only one pipeline is used instead of two ones which were necessary in the previous variant. This is due to raising steam and water by means of lifting force of air bubbles. At the same time it is not yet clear how such a system will work at sea, where because of the salt saturation, water does not freeze even at considerable temperature fall below zero. It is not yet clear how heat transfer from the fresh steam to the salt water will happen and wether this steam is turned into ice crystals just as it gets into the water with negative temperature.

3. Calculation of Pneumatic Installations.

Calculation of pneumatic installations includes the determination of main kinematic and thermal characteristics of the stream provided by them, calculation of pipeline conveying compressed air and choice of installation performance conditions.

Kinematics of stream formed by pneumatic installation in the first approximation was studies by M.I.Jhydkykh in her report to the present symposium on the previous problem. [1]

Based on the theory of bottom water lifting by air bubbles developed by I.M.Konovalov the formulae for determining velocities in region of ascending stream (fig.1) were obtained:

for plane current:

$$W = W_m e^{-1{,}07 \frac{\sigma^2 x^2}{z^2}} \qquad (1)$$

$$W_m = 1{,}38 \sqrt[3]{\frac{q_o g (2P_o + \gamma h)}{P_o}}; \qquad \sigma = 6.55$$

for an axysemmetrical current:

$$W = W_m e^{-0{,}517 \frac{\sigma^2 x^2}{z^2}}$$

$$W_m = 2{,}27 \sqrt[3]{\frac{q_o g (2P_o + \gamma h)}{P_o h}}; \qquad \sigma = 12{,}2 \qquad (2)$$

where W – flow velocity at any point of a jet;

Wm – the velocity at the jet axis;

q_o – an air discharge at the initial cross-section;

P_o – pressure on the water surface (equal to the atmospheric pressure);

h – the depth of water reservoir;

σ – empirical constant jet structure.

In order to determine region limits there are 2 equations:
for plane current: $x_p = 0{,}26z$,
for axisymmetrical current: $x_p = 0{,}20z$.

The zone of spreading can be represented either as a turbulent plane or fan-shaped semi-jet . According to Prandtle – Guertle it is discharging from an imaginary source and limited in the plane of symmetry by free surface. 1*)

Formulae for flow velocities in this region are:

$$V = V_o \left(1 + th^2 \xi\right) \qquad (3)$$

where: V – velocity in any point of the region of spreading;

V_o – the velocity over a free surface;

ξ – the relative coordinate $\xi = \frac{\sigma(h-z)}{x}$.

For the plane flow the following ratio is true.

$$V_o = 0.88\, W_m \sqrt{\frac{h}{0{,}64h + x}} \qquad (4)$$

$$\sigma = 8{,}2 \;, \quad \delta_o = 0{,}22h \;, \quad \delta = \delta_o + 0{,}26x$$

For space flow:

$$V_o = 0.39\, W_m \sqrt{\frac{h}{0.97h + x}} \qquad (5)$$

$$\sigma = 14.2 \;, \quad \delta_o = 0{,}17h \;, \quad \delta = 0{,}15x$$

Owing to the peculiarities of the flows discussed, connected with the space surface available, the values of experimental constants σ differ from the values obtained by Gertler, resulting from specially made experiments these values as well as the values of initial cross-section δ_o , were determined.

1*) In A.Murota and K.Muraoka's reports at the XII Congress I.A.H.R. the same flow scheme was assumed. [2]

In the reverse flow region the character of motion is more calm than in the upper region. Here energy losses can be neglected and the motion is considered to be potential. The formulae obtained are rather unwildy and are not given here.

Comparing the designed data with the experimental ones it follows that the above-mentioned relations are true at a distance of about 4 h. Then the velocities decrease sharply, the horizontal jet decays and flow changes its direction.

Making the epura of velocities according to the depth for surface stream one can obtain the curve of water discharge changes in the stream along the axis X.

For approximate thermal calculation to define the water reservoir limits maintained free of ice the thermal balance equation for surface stream elements Δx meters long [3] can be given:

$$c\gamma_0 d(Qt) = [K(t_B - t) - E]\,dx + dQ\,t^* c\gamma_0 + (S_g + S_m)\,dx \qquad (6)$$

where C – the thermal capacity;

γ_0 – the specific water weight;

Q – water discharge at a given cross-section of a jet;

t – the average temperature of water stream: $t \approx (0.6 \div 0.8) t_{gp}$ (fig 4),

K and E – coefficients in the heat irradiation formula respectevly dependent or nondependent on temperature;

t_B – air temperature;

t^* – the temperature of added or separated water masses.

In the first approximation it is considered to be equal to the domestic thermal section at the depth of set border when discharge is added and to average temperature of the jet itself when separated

S_g – the heat irradiation from the reservoir bed ground;

S_m – heat developed due to dissipation of stream, energy.

One can neglect the term (being too small) expressing heat developed due to energy stream. As function $Q = f(x)$ is given graphycally we must transit to final increment and can write the formula (6) in the following form [3]:

$$t_{n+1} = t_n - (t_n - t^*)\frac{\Delta Q}{Q_{cp}} + \left[K(t_B - t_n) - E + S_g\right]\frac{\Delta x}{c\gamma_0 Q_{cp}} \qquad (7)$$

where ΔQ – the discharge change between the adjacent cross-sections n; $n+1$, and Q_{cp} – average discharge between above – mentioned sections.

Using the expression (7) one can draw the mean change temperature curve over surface stream length.

To determine reservoir limits maintained free of ice, temperature curve should be projected on the edge of ice cover t_k, coming from the emperical dependence confirmed by numerous observations

$$t_K = \frac{S}{\alpha_K} \qquad (8)$$

where S - heat irradiation - k (t_B - t) - E ,

α_K - the irradiation coefficient in the edge area which is based on experimental data α_K = 0,002 V_c t cal/m^2 °C per 24 hours.

V_c - the average velocity in the considered jet cross-section in m/24 hours.

Intersection of curves $t_K = f(x)$ and $t = f(x)$ will point out the water reservoir limit to be maintained under conditions free of ice.
The aim of calculation of pipe-line of pneumatic installation is to define the initial pressure for passing air discharge given and necessary diameters of perforation orifices providing the uniform distribution of air discharge over the pipe-line length.

To find out the necessary dependences one can use Bernoulli's equation:

$$\frac{v\,dv}{g} + \frac{dP}{\gamma} + dh = 0 \qquad (9)$$

conservation equation when D = Const.

$$\frac{d\gamma}{\gamma} + \frac{dv}{v} = 0 \qquad (10)$$

and equation of gaseous state

$$\frac{P}{\gamma^n} = Const \qquad (11)$$

Here v - overall mean velocity of gaseous stream;
γ - volumetrical weight of a gas;
h - specific energy losses;
n - polytrope index.

Assuming this process to be isothermal which best corresponds to the character of the thermal process during air pipe operation B.S.Borodkin and G.I.Melkonjan have obtained the following designed dependences [3].

$$P_o = P_н \sqrt{1 + \frac{1,41\, Q^2}{a_o^2\, \Omega^2}\, \zeta_{общ}} \qquad (12)$$

Here $P_н$ - air pressure corresponding to the air pipe submersion depth H that is : $P_н = 0,1 H + 1$ atm

Q - volumetrical air discharge referred to pressure;

Ω - cross-sectional area of air pipe;

ζ - total resistance coefficient of air pipe;

$$\omega_{к+1} = \frac{1}{1,19\, a_o\, \mu_{к+1} \sqrt{\ell n \frac{P_{к+1}}{P_н}}} \cdot \frac{Q}{n+1} \qquad (13)$$

$\omega_{к+1}$ - area of k + 1 orifice;

a_o - adiabatic sonic velocity equal to $\sqrt{1,41 g RT}$

R - gaseous constant;

T - absolute temperature;

g - acceleration of gravity;

$\mu_{к+1}$ - discharge coefficient;

$P_{к+1}$ - pressure in front of K + 1 orifice;

K - orifice number counting from the sealed end of air pipe;

n - the number of clearences between orifices.

The value of $\mu_{к+1}$ is defined by the formula:

$$\mu_{к+1} = \frac{\varepsilon_{к+1}}{\sqrt{1 + \zeta_{к+1} - \left[\varepsilon_{к+1}(K+1)\frac{\omega_{к+1}}{\Omega} \cdot \frac{P_н}{P_{к+1}}\right]^2}} \qquad (14)$$

Where $\varepsilon_{к+1}$ - contraction jet coefficient in orifice;

$\zeta_{к+1}$ - resistance coefficient of orifice.

Economical indices are defined to a great extent by the right choice of installation operating conditions. Rational relationship between the period of idleness in the operation of installation Tp and that of its active operation conditions in installation Ta may be set up, if ice freezing on the water reservoir surface and the preceding melting ice process by using this considered installation can be investigated. This investigation must be carred out at different durations of $\frac{T_p}{T_a}$; at different air temperatures and temperature of water, lifted to the surface by means of this installation.

If the influence of heat stream from the bottom and heat developed due to energy dissipation can be neglected, that goes for period of ice formation of small thickness at considerable negative air temperatures, the differential equation of heat balance will be (3) :

$$Q\,dT = \gamma_л \beta\, dh_л \qquad (15)$$

where Q - heat stream through ice equal to heat irradiation;
T - time;
$\gamma_л$ - volumetrical ice weight;
β - latent ice formation heat;
$h_л$ - ice cover thickness.

Having integrated the equation (15) we get:

$$T = -\frac{\gamma_л \beta}{2\lambda_л t_в}\left[\left(h_2 + \frac{\lambda_л}{\alpha} + \frac{h_c \lambda_л}{\lambda_c}\right)^2 - \left(h_1 + \frac{\lambda_л}{\alpha} + \frac{h_c \lambda_л}{\lambda_a}\right)^2\right] \qquad (16)$$

Where $\lambda_л$, λ_c - coefficients of thermal conductivity of ice and snow respectively;
α - coefficient of heat irradiation from snow cover to the atmosphere.
h_1, h_2 - thickness of ice at the beginning and at the end of considered period of time;
$t_в$ - air temperature.

Heat balance equation for the ice melting process during pneumatic installation operation will be :

$$\frac{-t_в\, dT}{\frac{1}{\alpha} + \frac{h_л}{\lambda_л} + \frac{h_c}{\lambda_c}} = t\alpha_1\, dT - \beta\gamma_л(-dh_л) \qquad (17)$$

Where t - average temperature of surface steam formed by pneumatic installation ;
α_1 - heat irradiation coefficient of running water to the bottom ice surface.

Approximately:

$$\alpha_1 = 300 + 1800\sqrt{v}\ \frac{\text{k.cal.}}{\text{M}^2\text{hour }^\circ\text{C}}$$

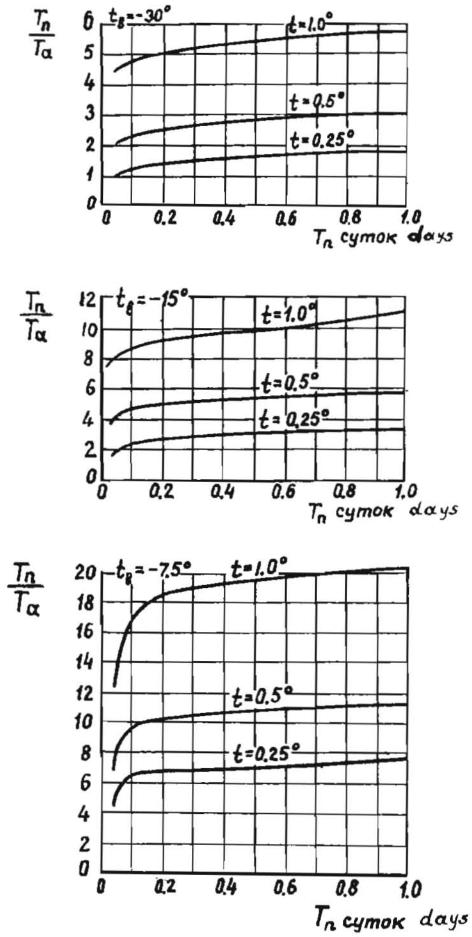
$\frac{T_n}{T_\alpha}$
$t_B = -30°$
t=1.0°
t=0.5°
t=0.25°
T_n сутoк days
$t_B = -15°$
t=1.0°
t=0.5°
t=0.25°
T_n сутoк days
$t_B = -7.5°$
t=1.0°
t=0.5°
t=0.25°
T_n сутoк days

Fig. 6

$\mathcal{V}$ - water flow velocity under ice in meters per second. Having integrated the equation (17) we derive the formula for Ta that is a period of time necessary for ice melting:

$$T = \frac{\gamma_{\text{л}} \lambda_{\text{л}} \beta}{(\alpha_1 t)^2} \left\{ \frac{\alpha_1 t h_1}{\lambda_{\text{л}}} + t_{\text{в}} \ln \frac{t_{\text{в}} + \frac{\alpha_1}{\alpha} t}{t_{\text{в}} + \left(\frac{1}{\alpha} + \frac{h_1}{\lambda_{\text{л}}}\right) \alpha_1 t} \right\} \qquad (18)$$

Figure 6 shows the results of calculations for different values $\frac{T_n}{T_a}$; $T_n ; t ; t_{\text{в}}$. According to figure 6 the usage of condition with interval some 1,5 -2,5 hours is considered to be the most expedient. The usage of short intervals is not effective. Prototype investigations show that unsteady heat condition near the pneumatic installation is maintained about 0,5 - 1,5 hours after it has stopped acting. Thus during those intervals ice covering would not be formed even in severe condition.

Figure 6 shows that in ordinary conditions the ratio $\frac{T_n}{T_a}$ approximates 3 - 4 that is to make installation operate in several stages. (Fig.7). This allows either to provide high efficient durable operation of the compressor and to reduce capital investments or to enlarge the width of reservoir maintained free of ice by laying several parallel pipe lines operating in certain sequence. Reservoir maximum width B_2 to be maintained in some conditions is defined from the formula :

$$B_2 = \left(1 + \frac{T_n}{T_a}\right) B_1 \eta \qquad (19)$$

Where B_1 - iceless reservoir width formed by an air pipe operating separately;

η - coefficient taking into account the change of reservoir width per an air pipe when passing to parallel operation of several air pipes, on the base of experimental data this coefficient can be assumed to be equal 1,5 - 2, 0.

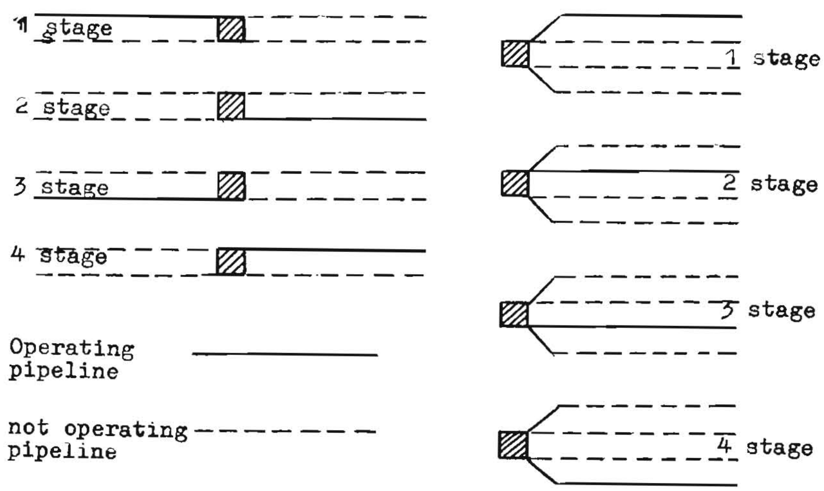
1 stage
2 stage
3 stage
4 stage
Operating pipeline
not operating pipeline
1 stage
2 stage
3 stage
4 stage

fig. 7

4. Calculation of stream forming devices.

Kinematics of fluid movement in the area of stream forming device is due to the availability of a jet discharged from the nozzle which should be considered as a free turbulent jet and to suction fluid stream. The studies carried out showed that the latter's influence on the distribution of velocities in turbulent jet can be neglected.

The velocity at any point of axisymetrical turbulent jet at a distance of "r" from its axis can be found according to the formula [3] :

$$V = \frac{r_o V_o}{2ax} e^{-\frac{r^2}{8a^2x^2}} \qquad (20)$$

Where r_o - radius of nozzle outlet section;
V_o - velocity in nozzle outlet section;
x - coordinate along jet axis counted off the nozzle end;
a - experimental coefficient which can be considered to be equal 0,04.

Accounting the influence of limiting surfaces parallel to jet axis on its velocity field and in particular the influence of fluid surfaces availability can be made by interposing epure of energy $\frac{V^2}{2g}$ from several fictitious properly placed stream forming devices. Hence the resulting velocity will be $V = \sqrt{V_1^2 + V_2^2}$ as the equation used is linear in relation to the square of velocity.

Thermal calculation can be carried out using the equation (7) with the only difference that in this case the energy irradiation term should not be neglected especially near the orifice since the velocities of water discharging from the stream-forming device are considerable.

The value of the term Sm may be defined as the amount of heat given off by the stream due to its kinetic energy change betwеen cross-sections "n" and "n + 1" used for overcoming the turbulent friction forces.

$$S_m = \frac{\sum_{i=-m}^{i=+K} \sum_{i=-p}^{i=+u} \frac{\rho V_{n_i}^3}{2} \Delta y \Delta z - \sum_{i=-m}^{i=+K} \sum_{i=-p}^{i=+u} \frac{\rho V_{(n+1)_i}^3}{2} \Delta y \Delta z}{427} \qquad (21)$$

The values "m" and "k" correspond to the number of intervals over the vertical axis "y", counted off the orifice axis up to the point where velocity value will approximate zero, otherwise the intervals will not reach the fluid surface. As well "p" and "u" correspond to the number of strips this section is divided into, towards the axis "z".

The limit of the water area maintained icefree is defined in the same ways as it was when calculating pneumatic installations.

To define time necessary for ice melting up to the thickness required or for deicing the water area of given limits is possible by solving for the first problem the equation integral (17) as follows:

$$T = -\frac{\gamma_{\text{л}} \lambda_{\text{л}} \beta}{(\alpha_1 t)^2}\left[\frac{\alpha_1 t}{\lambda_{\text{л}}}(h_2 - h_1) - t_{\text{в}} \ln \frac{t_{\text{в}} + \left(\frac{1}{\alpha} + \frac{h_c}{\lambda_c} + \frac{h_2}{\lambda_{\text{л}}}\right)\alpha_1 t}{t_{\text{в}} + \left(\frac{1}{\alpha} + \frac{h_c}{\lambda_c} + \frac{h_1}{\lambda_{\text{л}}}\right)\alpha_1 t}\right] \qquad (22)$$

Where h_1 and h_2 - the initial and final thickness of the ice cover respectively.

To solve the second problem the equation (18) can be used, if the calculation is carried out for separate parts.

It should be noted that the above mentioned method of calculating gives, the minimum deiced area of reservoir during some period of time, as the inhomogeneous structure of the ice cover, consisting of crystals and inter-crystal filling is not taken into account. The latter is broken faster as a result of thermal and especially mechanical effect of the stream-forming device jet. Therefore, after weakening or complete destruction of inter-crystal bonds, the spontaneous breaking of ice cover carrying unmelted crystals may occur. In this case to melt 1 kg. of ice less thermal energy than "β" will be required.

5. Calculation of pneumatic stream-forming devices.

Discharge of water supplied by a pneumatic stream-forming device can be defined by the formula [3]:

$$Q = \mu\omega\sqrt{2gH_1(\gamma-\gamma_1)}\ \frac{\gamma_1}{\gamma} \qquad (23)$$

Where γ_1 - the specific weight of water-air emulsion;

H_1 - the depth of lowering of the bottom pipe end from water surface;

ω - pipe sectional area;

μ - coefficient of discharge.

Air volume change being neglected, air discharge will be:

$$Q_в = Q\ \frac{\gamma-\gamma_1}{\gamma_1} \qquad (24)$$

Comparative data of water discharges supplied to the surface by means of pneumatic installation or pneumatic stream-forming device, air discharge being the same and corresponding to the optimum saturation of water-air mixture with air in the pneumatic stream-forming device, are given in table 1.

As the depth becomes lesser, the difference between water discharge supplied by the pneumatic installation and pneumatic stream-forming, device is reduced. If the t° of water supplied by the pneumatic stream-forming device is equal to the temperature of near bottom layers, and that of the pneumatic installation is lower, then it comes clear that the less the depth, the more efficient the use of pneumatic stream forming devices.

Table 1.

Depth m.	Air discharge per one pneumatic stream-forming device or per one measure metre of pneumatic installation $m^3/sec.$	The discharge of water lifted by pneumatic stream-forming device when $\mu\omega=0,1\ m^2$ $Q\ m^3/sec$	The discharge of water lifted by one measure metre of pneumatic installation. $Q'\ m^3/sec.$	Discharge ratio $\frac{Q'}{Q}$
3	0,13	0,30	0,65	2,16
5	0,25	0,38	1,40	3,70

10	0,32	0,55	3,70	6,70
15	0,35	0,66	5,20	7,9
20	0,36	0,77	6,95	9,0

The velocity condition of the stream, pouring out over the upper edge of the pneumatic stream-forming device can be calculated by using the following dependence for the velocity in the fanshaped turbulent jet [4].

$$V = \frac{V_o}{\sqrt{2}} \cdot \frac{r_o}{r_o + x} \sqrt{\Psi\left(\frac{b_o - y}{2ax}\right) + \Psi\left(\frac{b_o + y}{2ax}\right)} \qquad (25)$$

Where: r_o - radius of the pneumatic stream-forming device;

x - distance of the point, where the velocity is defined by the radius from the edge of the pneumatic stream-forming device;

V_o - horizontal jet velocity on leaving the pipe is approximately equal to: $\frac{Q}{2\pi r_o b_o}$

y - coordinate over the vertical counting from water surface;

Ψ - Cramp's function;

b_o - distance from water surface to the upper end of the pneumatic stream-forming device;

a - experimental coefficient $\cong 0{,}04$.

The calculation of the deiced water area limit can be carried out using the recommendations referring to the calculation of pneumatic installations.

6. Maintenance of non-running and poor running water areas iceless and their deicing by artificial heating.

When solving the first problem the main equation can be expressed as follows:

$$S = kqtc\gamma \qquad (26)$$

Where: S - heat irradiation from 1 sq.m. of free surface per hour;

k – the coefficient of supplied heat utilization;
q – water discharge per 1 sq.m. of the surface;
t – temperature of supplied water;
γ – volumetrical water weight;
c – thermal heat capacity.

To solve the problem with minimum expense it is necessary to supply water at ~ 20°C correspondingly increasing the discharge in order to get the necessary product value "qt" at minimum thermal irradiation. The same stipulates the requirement of maximum uniform distribution of supplied warm water over the water area surface, The system of perforated pipe lines laid on the reservoir bottom meets the requirements best of all.

As physical and convective heat conductivity in stagnaut or poor running water reservoirs, where artificial heating is most expedient, are small, it is necessary to provide intensification of stream turbulization not to utilize much heat for initial water. The best solution of this problem is to supply compressed air by the system of perforated pipe lines located amidst adjacent pipe lines supplying hot water. Necessary air discharge in accordance with necessary water volume which is to be lifted to the surface is defined by the following formula [3]:

$$Q = 0{,}146H\sqrt[3]{\frac{q_o(P_o+\gamma H)g}{0{,}044\gamma H}\ln\frac{P_o+\gamma H}{P_o}} \qquad (27)$$

Where: H – depth;
q_o – air discharge per 1 measure metre of air pipe line;
P_o – atmospheric pressure;
γ – specific water weight;
g – acceleration of gravity.

The specific feature of calculation of horizontal perforated pipe lines for supplying warm water [5] is in the necessity of keeping constant product of discharge by the temperature of water discharging from each orifice, that is:

$$c\gamma q_1 t_1 = c\gamma q_2 t_2 = c\gamma q_n t_n = \frac{c\gamma Q_o t_o - S_1}{n} \qquad (28)$$

Where: Q_o - water discharge at initial cross section;
t_o - water temperature at initial cross section;
n - the number of outlets;
S_1 - thermoirradiation through the walls of the pipe-line;
C - thermal heat capacity of the water;
γ - volumetric water weight;
c, γ - assumed as non - dependent on temperature.

Heat balance equation for the portion of the pipe-line dx long can be written:

$$d(Qtc\gamma) = dS_1 + dS_2 + dS_3 \qquad (29)$$

Where: dS_1 - thermoirradiation through the walls;
dS_2 - heat discharging through outlets together with water.
dS_3 - heat developed owing to the forces of internal friction;
dS_3 - may be neglected as being very small a value.

Suppose, that heat charge over the pipe-line length takes place according to the linear law. Then, to calculate water temperature change over the pipe-line length, we obtain:

$$t = \frac{\theta t_o \exp \theta B \ln \frac{\mathcal{L}}{\mathcal{L}-x}}{\theta - t_o\left[1 - \exp \theta B \ln \frac{\mathcal{L}}{\mathcal{L}-x}\right]} \qquad (30)$$

Where: θ - the temperature of surrounding pipe-line medium - water in the reservoir;
$\mathcal{L}$ - the length of the perforated portion of the pipe-line;

$$B = \frac{\pi \mathcal{D} K \mathcal{L}}{Q_o t_o c \gamma}$$

$\mathcal{D}$ - the diameter of the pipe-line;
K - heat transfer coefficient;

Supposing, as usually, $\theta \simeq 0$ we derive:

$$t = \frac{t_o}{1 - A t_o \ln\left(1 - \frac{x}{\mathcal{L}}\right)} \tag{31}$$

Where: $$A = \frac{\pi \mathcal{D} K \mathcal{L}}{Q_o c \gamma}$$

If the linear change of the discharge over the pipe-line length is supposed to be near the really exhisting one, as the experiments show, the equation (31) will be:

$$t = \theta + (t_o - \theta) \exp A \ln\left(1 - \frac{x}{\mathcal{L}}\right) \tag{32}$$

It is proved by the experiments to be satisfactory.

The area of the outlets can be determined by means of the formulae derived both from the equation (28), with (32) taken into account and the value of heatirradiation through the walls all the pipe- line length $S_1 = K\pi\mathcal{D}(t_o - \theta)\frac{\mathcal{L}}{A+1}$

$$\omega_n = \frac{Q_o t_o c\gamma - K\pi\mathcal{D}(t_o - \theta)\frac{\mathcal{L}}{A+1}}{c\gamma n \mu_n \sqrt{2g\frac{P_o}{\gamma}}\left[\theta + (t_o - \theta)\exp A \ln\left(1 - \frac{x}{\mathcal{L}}\right)\right]} \tag{33}$$

Where: $\frac{P_n}{\gamma}$ - piezometric pressure in front of the outlet;
μ_n - outlet discharge coefficient.

When the pipe-lines are laid on the inclined plane, i.e. at the building-berths and slips, it is necessary to provide not the uniform heat distribution, but the equality of axial temperatures at the surface in all the jets discharged from perforated pipe-line.

According to Tailor's theory the following temperature dependence in a submerged, axisymmetrical turbulent jet [6]:is derived

$$\frac{\Delta t_T}{\Delta t_o} = \frac{0.7}{\frac{\alpha y}{R_o} + 0,29} \tag{34}$$

Where: Δt_T - the difference between t_T over the jet axis at the distance y from the outlet and the temperature in the surrounding space;

Δt_o - the difference between t° at the initial cross-section of the jet and the temperature; Q in the surrounding space;

a - experimental coefficient $\sim$ 0,1;

y - the distance from the outlet up to the point on the jet axis, we are interested in ;

R_o - outlet radius.

Due to relatively high temperature of the water discharging from the outlet, one can take $\theta \cong 0$ besides, it should be taken into account that $\frac{ay}{R} \gg 0,29$ and hence:

$$\frac{t_{01} R_{01}}{t_{02} R_{02}} = \frac{y_1}{y_2} \qquad (35)$$

Where : y_1 and y_2 - coordinates over vertical.

To satisfy the conditions (35) non-linear change of the discharge over the length is required. In the first approximation it may be expressed by the formula:

$$Q = Q_o \left(\frac{\mathcal{L} - x}{\mathcal{L}}\right)^m \qquad (36)$$

Q - water discharge in the cross-section of the perforated pipe-line considered;

Q_o - water discharge in the initial cross-section of the pipe- line;

m - degree index determined experimentally.

If the heat developed owing to the internal friction forces is neglected, the heat balance equation for the pipe-line portion dx long will be:

$$Q c \gamma dt = -\pi \mathfrak{D} \kappa (t - \theta)\, dx \qquad (37)$$

Having integrated it (36) taken into account, the following expression for temperature is obtained:

$$t = \theta + (t_o - \theta) \exp \frac{A_1}{1-m} \left[(\mathcal{L} - x)^{1-m} - \mathcal{L}^{1-m} \right] \tag{38}$$

Where: $A_1 = \dfrac{\pi \mathcal{D} K \mathcal{L}^m}{Q_o c \gamma}$

Taking into consideration (35) the following expression for the area of any K shaped outlet at the distance x from the beginning of the perforated pipe-line portion is obtained:

$$\omega_K = \omega_o \left(\frac{t_o \cdot y_K}{t_K \cdot y_o} \right)^2 \tag{39}$$

The value of ω_o - the first outlet should be determined from additional condition the equality of the sums of discharges passing through separate outlets to the initial discharge Q_o.

To tackle the second task i.e. to deicing the area such heat balance equation may be written:

$$K q t c \gamma dT = \beta \gamma_л dh_л - \frac{t_в \, dT}{\frac{1}{\alpha} + \frac{h_л}{\lambda_л} + \frac{h_c}{\lambda_c}} \tag{40}$$

The integrating of this equation results in the formula:

$$T = \frac{-\beta \gamma_л \lambda_л}{(K q t c \gamma)^2} \left[\frac{K q t c \gamma}{\lambda_л} (h_2 - h_1) - t_в \ln \frac{t_в + K q c \gamma t \left(\frac{1}{\alpha} + \frac{h_c}{\lambda_c} + \frac{h_2}{\lambda_л} \right)}{t_в + K q c \gamma t \left(\frac{1}{\alpha} + \frac{h_c}{\lambda_c} + \frac{h_1}{\lambda_л} \right)} \right] \tag{41}$$

From this expression we can see that the second term of the equation in brackets characterizing the time rise for deicing water area by heat irradiation into atmosphere is inversely proportional to the second power product of the supplied water discharge by the temperature.

That's why for the fastest and most economical deicing the water area at low air temperatures one should rise the temperature and the supplied water discharges as much as possible.

If for heating instead of warm water steam-air mixture is used, the calculation of the temperature change over the pipe line length as studies show [7], may be carried out by the formula (32), the expression for "A" being given as follows:

$$A = \frac{\pi k \mathcal{D} \mathcal{L}}{G_o c}$$

Where G_o - weight discharge of steam-air mixture in the initial cross section.

For the area of the outlets the following expression is derived:

$$\omega_n = \frac{G_o t_o c - k\pi\mathcal{D}(t_o - \theta)\frac{\mathcal{L}}{A+1}}{n \mu_n B t_n P_n \sqrt{\frac{T_o}{T_o - t_n}}} \qquad (42)$$

Where: $B = C\sqrt{2g\frac{\gamma_k}{P_k}}$

Here γ_k and Pk - volumetric weight and pressure
To - absolute gas temperature.

As to other calculations they may be carried out by using formulae for pneumatic installations given in 3 of the paper.

7. Maintenance of reservoirs of running water in iceless conditions by artificial heating.

Maintenance of navigable channels or rivers deiced may be carried out by using exhaust water of enterprises and first of all powerful condensed thermal stations with the rational disposition over the waterway.

For preliminary solution of the problem, the task of stream motion having value of heat irradiation from its surface due to stream temperature change should be considered here. The temperature and meteorological conditions in the regions surrounding the stream, its depth and width as well as the coming in discharge are supposed constant.

Heat balance equation for the elementary stream part "dx" long will be written [4]:

$$c\gamma Q(t-dt)-c\gamma Qt=(Mt+N)\,dx \tag{43}$$

Q - stream discharge;

M and N - coefficients in the heat irradiation formula.

The solution of this differential equation under boundary conditions: $t = t_0$ when $x = 0$ gives

$$t=\left(t_0+\frac{N}{M}\right)e^{-\frac{Mx}{c\gamma Q}}-\frac{N}{M} \tag{44}$$

Hence, it is easy to find out the distance at which temperature changes from "t_0" to "t", i.e., the distance from one condensed station to the next one.

$$x=\frac{c\gamma Q}{M}\ln\frac{Mt_0+N}{Mt+N} \tag{45}$$

For most economical utilization of heat supplies available especially in the conditions of non canalizated river, it is expedient to separate in some way the region of ship running where the deiced channel is to be maintained. Nowadays, it may be achieved by guarding of the channel, e.g., by the film of synthetic materials (fig.8)

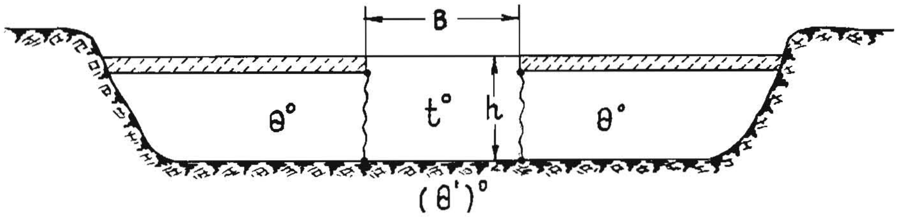

fig.8

According to the conditions considered, the value of temperatures, surrounding the main stream of regions taken into account, coefficient values "M" and "N" may be written:

$$M = \kappa B + 2h\kappa_1 + B\kappa_2$$

$$N = \kappa t_\theta B + EB + 2h\kappa_1\theta + B\kappa_2\theta'$$

where κ, E, t_θ see the equation (6):

B - stream width of warm water;

h - the depth of the same stream;

κ_1 - total heat transfer coefficient from stream reservoir water through the film;

κ_2 - total heat transfer coefficient from stream water to reservoir bottom;

θ - reservoir temperature, considered constant;

θ' - reservoir bottom temperature, considered constant.

More detailed studies of the problem of the temperature change over the channel length and depth may be carried out by using the equation suggested by A.I.Pekhovich, S.M.Aleinikov and V.M.Jhidkikh. The derivation of these equations is set forth in the paper to be delivered at the present sitting. Therefore, I shall give now only the final expression to ascertain stream temperature [8]:

$$\theta = \frac{t - t_0}{\vartheta_э - t_0} = \cos\left[\sqrt{B_i}\,(1-\eta)\right]\exp(-M_i) \qquad (46)$$

where:

$$B_i = \frac{\alpha h}{\lambda z}\ ; \quad \eta = \frac{z}{h}\ ; \quad M_i = \frac{\alpha x}{c\gamma h v}\ ;$$

Here: x - coordinate over the stream;
h - channel depth;
t - water temperature;
t_0 - initial temperature;
λ - coefficient of heat conductivity;
α - coefficient of heat irradiation;
ϑ - effective air temperature;
v - flow velocity;
c - thermal heat capacity water;
γ - specific water weight.

Using the equation (45) one can find the distribution of temperature both over the length and depth of the stream in any section.

Criterian for choosing final temperature "t" where the next station is to be situated is the requirement of the power station to water temperature of the pond, i.e. water cooler. It should be noted that in warm seasons when more intensive cooling is required it may be carried out in accordance with the scheme considered by means of removal guarding facilities of running and including the whole water area of the reservoir into the cooling surface.

8. Rational Construction of Water Intake and Means of Intensification of Frazil Ice Discharge through Water Receiver.

Fraizil ice, having lesser volumetric weight than water usually tends to concentrate on the fluid surface or near it. In accordance with this the configuration of water intakes is of vital importance for improving their operation. The most favourable conditions for diversion of near bottom water usually saturated least of all with frazil ice are created when placing water inlet according to the scheme (fig. 9-a), when water shed fills near the bottom. Then it may be expressed:

$$Z_B + \frac{P_B}{\gamma} = Z_A + \frac{P_A}{\gamma} + \frac{1}{g}\int_B^A \frac{v^2}{\rho}\, dS \qquad (47)$$

Where z – geodesic height;
p – pressure;
γ – volumetric weight;
g – acceleration of gravity;
v – velocity;
ρ – radius of jet curvature;

Hence
$$z_B + \frac{p_B}{\gamma} \gg z_A + \frac{p_A}{\gamma}$$

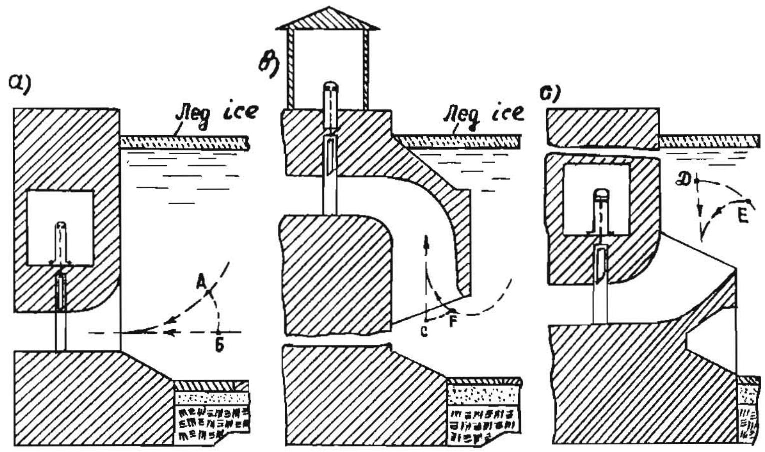

fig. 9

If such a position of water intake is not convenient for some reason and the water intake mouth raises at higher marks it should choose water intake according to the scheeme (fig.9-b)

Least successful is the scheme (fig.9-c) which should be avoided.

Rather often some reason that makes the operation of water intake more difficult or impossible is icing of grate rods.

To prevent this electric heating of grates is usually used. Investigations carried out by A.I.Pekhovich and S.M.Aleinikov [9] showed that value of heat irradiation coefficient on the surface of the grate rod is not constant but varies depending on the length of the stream line way. From criterial equation derived by M.A.Mikheev.

$$Nu = 0{,}0356\, Re^{0.8}\, Pr^{0.4}$$

A.I.Pekhovich obtained the following dependence heat irradiation coefficient from iron to water when having frazil ice.

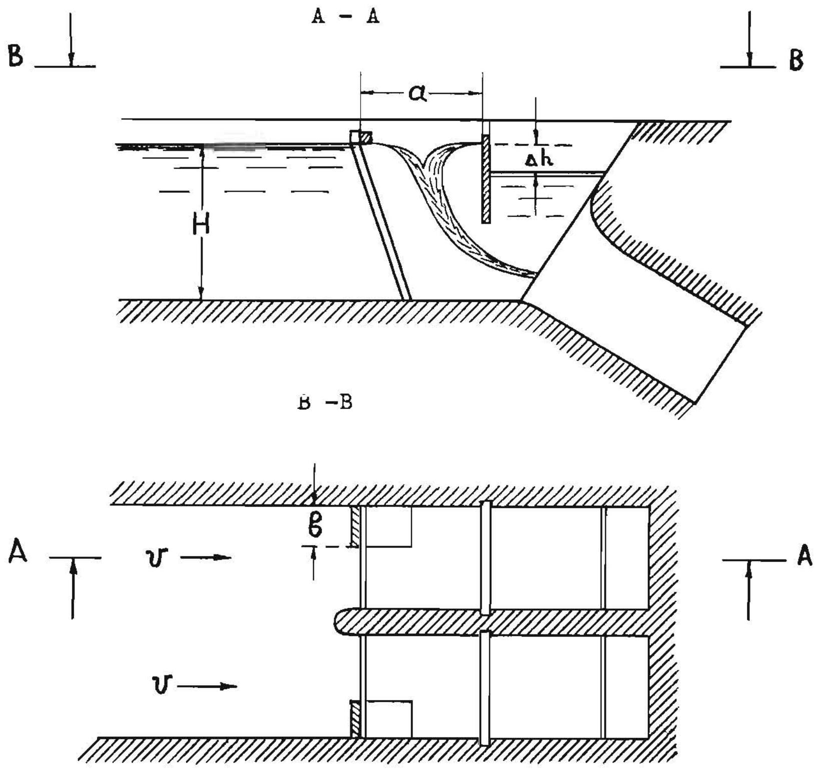

fig. 10

$$\alpha_c = 1672 \frac{v^{0.8}}{x^{0.2}} \quad \text{k.cal./m}^2 \text{ hours °C} \qquad (48)$$

Where v - water velocity in a grate in m/sec;
x - the coordinate, being counted from the frontal surface of the rod.

Instability of heat irradiation coefficient over the rod surface involves instability of temperature on the surface of the same rod, the beginning of the rod being less heated. That's why calculation of power capacity is to be made for it, but not for mean coefficient of heat irradiation or for average temperature of the rod. Only then it is possible to prevent coverring rods with ice. With a view of economy of electrical energy it is advisable to make heating making of the frontal part of the rod by eccentric disposition of heaters or by partial heat insulation of heated rods. The following formulae were derived for calculating power capacity of heating of right-angled rods.

$$P = 7{,}7\, v^{0.8} (0{,}01 - t_1) \quad \text{kvt/m}^2 \qquad (49)$$

for round rods

$$P = 2 \frac{v^{0.6}}{d^{0.4}} (0.01 - t_1) \quad \text{kvt/m}^2 \qquad (50)$$

Here t_1 - temperature of super cooled water $\sim$ - 0,09°
d - diameter of the rod in m.

Experiments and field investigations are confirmed by formulae (49) and (50). It should be taken into account here that devices for heating should protect grates from sticking crystals of ice to rods. As well as from crystalization of saper cooled water upon them rather then the task of melting passing masses of frazil ice that is unpracticable. The character of surface of the grate rods is strongly influenced by freezing crystals to in frazil ice.

Covering rods with rabber, asphalt, oil, bitumen, silicon organic combinations creating hydrophobic films on the surface of metals improves the functioning of the grates.

Determination of correct distance between rods is of great importance for proper function of grates. Eddy water craters are means of intensification of frazil ice discharge through water receivers. The essence of such a method of frazil ice passing is the following (fig.10) [10, 11].

In pressure cell thrust chamber in front of the intel of the water intake a wooden shield reaching the bottom of the basin is installed at the angle of $\sim 18°$ to the vertical line. The second shield don't reaching the bottom but creating a fall Δh is put between them and the grates of water receiver. A water crater is formed be between these shields. Because of considerable velocities of circumference of water rater bodies, floating on the surface are carried down and fürther to the pipe-line of a turbine. Dimensions of seperate elements of complex of measures bringing to water crater formation in the first approximation may be found from the following relations:

$$\frac{a}{b} \cong 1{,}2 \div 1{,}6 \qquad b = B\left(\frac{\Delta h}{H}\right)^{K} F$$

where a - distance between shields N 1 and 2 (fig. 10);
b - width of the shield (fig.10);
Δ h - value of fall (step);
H - depth of stream in front of cell (chamber);
K - coefficient varying approximately linearly within the limits of 0,2, when V = 0,5 till 0,4 , when V = 1,2;
F - coefficient varying approximately linearly within the limits of 0,8 when V = 0,5 till 0,3 when V = 1,2.
V - velocity water approach to the shield.

Value of a fall depends upon the percentage of frazil ice in the stream and must not exeed 1,2 m.

It is necessary to protect the pipe line from cold air penetration through a water crater, shaping the inlet part in such a way as to insure air outlet from the water crater to the surface.

It should be noted that large velocities in a grate, created by a water-crater, increase the value of the coefficient (α) and, hence, temperature gradient being ($t_2 - t_1$) and showing absence of crystalization on grates, it increases necessary heating capacity. That's why the usage of method of intensification of frazil ice passing through the turbines of hydro-power stations by means of eddy water craters is more expedient with watercourse having comparatively great flow velocities (mountain rivers) and in case of unsteady cold snap. Then in front of the water intake frazil ice mainly brought from above is accumlated and there may be on super cooling of water in the site of water intake i.e. the danger of grate freezing may lack. A water crater will promote (help) breakage of ice sheets covering the surface of water in front of the water intake and the free passing through grates.

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HEAT EXCHANGES AND FRAZIL FORMATION

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SYNOPSIS

The most important terms in the heat balance of a body of water are radiation, convection and evaporation. Examples of empirical approximations to the more exact expressions are given.

The heat lost by the water results in a change of state after the water has become sufficiently supercooled.

When the surface turbulence is strong enough, an ice cover cannot form, and heat losses then produce frazil ice. This process and the subsequent history of the frazils are discussed.

THE HEAT FLUXES

Introduction

Between a body of water at rest and its environment heat is exchanged, with the atmosphere through the free water surface and with the earth through the boundaries. These heat fluxes are of three kinds: radiation, convection/conduction and evaporation/condensation.

The heat balance for flowing water includes one more term, namely frictional heat. Friction converts mechanical energy into heat by a one-way process that can only add heat to the water.

The heat flux which is a consequence of mass transport by precipitation, surface runoff or ground-water flow, is not included in this discussion. Neither is the terrestrial heat flux, from the inner of the earth through its mantle, taken into account, as this term is small compared with the other terms.

The various heat fluxes out of and into a water body are listed as loss or gain in table 1.

Table 1 Heat fluxes

Loss		Gain	
long-wave radiation	(S_1)	short-wave radiation	(S_1)
evaporation	(S_2)	long-wave back radiation	(S_1)
convection	(S_3)	condensation	(S_2)
		friction	(S_4)

The net heat loss results in cooling when $T > 0^oC$ and in supercooling and ice formation when $T < 0^oC$.

A review of the terms in table 1 is given below followed by a discussion of the conditions under which frazil ice forms. A good summary of the heat balance in rivers is given by FREYSTEINSSON 1968.

Radiation

The radiation flux is for our purpose conveniently split into short-wave and long-wave components:

The short-wave radiation is made up of those heat rays that penetrate water to considerable depths, such as sunshine and visible light.

The long-wave radiation is due to heat rays that are stopped by a few mm of water, such as infrared light.

The dividing line between short and long wavelengths is at a few microns (fig. 1). Instead of short-wave radiation we can just as well say solar radiation, because the solar spectrum has 99% of its energy in the wave range 0,15 - 4μ .

Similarly, long-wave radiation is also called terrestrial radiation.

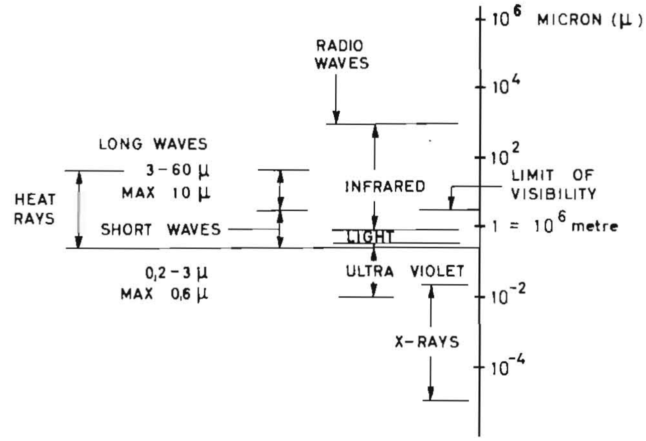

Fig. 1 Wave length ranges

The reason for splitting the radiation flux in these two components, is that short-wave radiation shows up in the heat budget only as a gain, like frictional heat. Long-wave radiation, on the other hand, causes heat gains as well as heat losses. This is demonstrated in Fig. 2.

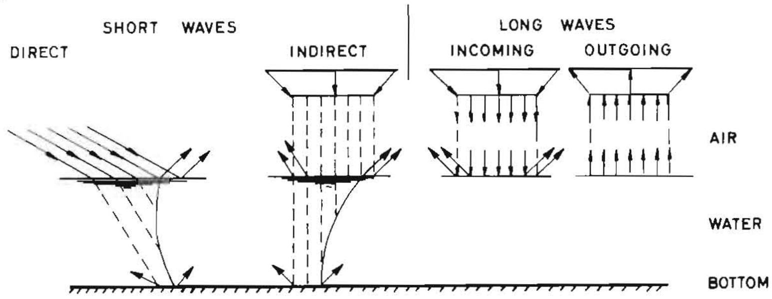

Fig. 2 Schematic of radiation fluxes

Short-wave radiation. The most common formula for the heat flux by short-wave radiation is

$$G = G_o(a-bN^m) \quad (1)$$

G_o is the heat flux with a cloudless sky (insolation), and a, b and m are local coefficients depending on latitude and topography. N is an index for the cloud cover with range 0-8 or 0-10, commonly used by meteorologists.

Of the incoming radiation G a fraction α G is reflected by the water surface, while $(1-\alpha)G$ is absorbed. The coefficient of reflection α is called the albedo and depends partly on the roughness and texture of the surface, partly on the sun's declination.

There exists so many observations of solar radiation that we can determine fairly accurately the short-wave radiation flux. The solar constant, or the energy flux at the upper limit of the atmosphere, is approximately 2 cal/min cm^2 (in more familiar units, 1,4 kW/m^2).

Most of the solar energy is absorbed or reflected by the atmosphere. That part of the short-wave radiation reaching the earth as sunrays is called direct radiation. Rays with other directions, reflected from the air or from clouds, is called diffuse radiation. The insolation G_o includes both direct and diffuse radiation. Table 1 gives some characteristic daily values of G_o, averaged over 24 hours, and derived from the records of the Norwegian Meteorological Institute, 1901-1930.

Table 1 Insolation, kcal/daa,s

Met. station	N. lat.	Dec	Jan	Feb	Mar
Lillehammer	66,1	3	4	12	32
Mo i Rana	66,3	3	1	6	25
Alta	70,0	0	0	4	20

The albedo of a water surface is 8-10%.

DEVIK (1931) proposed b = 0,09, m = 1 in (1).

Long-wave radiation. Long-wave radiation obeys BOLTZMANN's law, which states that the radiation flux from a body is proportional to the fourth power of the body temperature:

$$\text{heat flux} = \zeta\sigma T_b^{\,4} \quad (2)$$

where ζ is the emissivity of the body surface, σ is the Stefan-Boltzmann constant and T_b is the body temperature in ^oK.

The competing fluxes of long-wave energy from the water surface to the atmosphere and the other way around gives a net heat loss according to the old DEVIK (1931) formula

$$S_L = 1{,}05\ (T_w - T_a) \quad {}^{kcal}/_{daa,s} \tag{3}$$

where T_w is the water temperature and T_a the air temperature in ^oC. This formula is based on the following assumptions: 1) the water surface is surrounded by an extensive earth surface with the same temperature as that of the air immediately above, 2) the atmospheric lapse rate is $0{,}5^oC$ per 100 m and 3) the water surface acts as a black body.

The latter assumption is permissible, as the emissivity of the water surface is $\zeta = 0{,}95\text{-}0{,}97$. The two other assumptions are restrictive. If there is an inversion, (3) overrates the radiation loss.

The total radiation loss is

$$S_1 = S_L - G$$

where S_L is given by (3) or some other approximation to (2), and G is given by (1).

Evaporation and Convection

Evaporation and convection are analogous processes of diffusion, and the heat loss losses by these mechanisms may be written

$$S_2 = -K_e \frac{de}{dz} \gamma L \tag{4}$$

$$S_3 = -K_h \frac{dT}{dz} \gamma c \tag{5}$$

K_e and K_h are turbulent diffusion coefficients for vapour and heat, and $\frac{de}{dz}$ and $\frac{dT}{dz}$ are gradients of humidity and temperature, respectively. L is the latent heat of evaporation, c is the specific heat and γ the specific weight of water.

In contrast to radiation, which is not greatly influenced by wind, the diffusion fluxes depend strongly on the wind speed and the wind profile. The wind field actually generates the turbulence that carries out the transport. However, near the water surface where the turbulence is damped out, molecular diffusion takes over. In this region K_e and K_h in (4) and (5) should be replaced by their molecular counterparts D_e and D_h. The resulting fluxes still depend on the turbulence in the air flow because the gradients $^{de}/_{dz}$ and $^{dT}/_{dz}$ at the water surface are functions of the Reynolds number of the air flow.

Many empirical formulae exist for the heat losses S_2 and S_3. As an example we give those of DEVIK 1931

$$S_2 = 3,0\ (v+0,3)^{0,5}\ (e_w-e_a) \qquad (6)$$

$$S_3 = 1,4\ (v+0,3)^{0,5}\ (T_w-T_a) \qquad (7)$$

where v is the wind velocity in m/s, and e_w and e_a the vapour pressure in mm Hg at the water temperature T_w and air temperature T_a, respectively.

Frictional heat S_4

Joule's constant gives the relation between thermal and mechanical energy:

1 kcal = 427 kpm

It takes 427 kpm of mechanical energy to heat 1 kg of water 1^{o}C. This means that a head loss of 4,27 m (or 4,27 kpm/kg) raises the water temperature $\frac{1}{100}$ ^{o}C.

The heat gain due to friction is therefore important only in steep rivers.

Fig. 3 Stable ice cover with small open area due to frictional heat

Fig. 3 shows a reach of the river Glomma with just the right bed slope to maintain a small open area, through which the frictional heat can escape, throughout the winter.

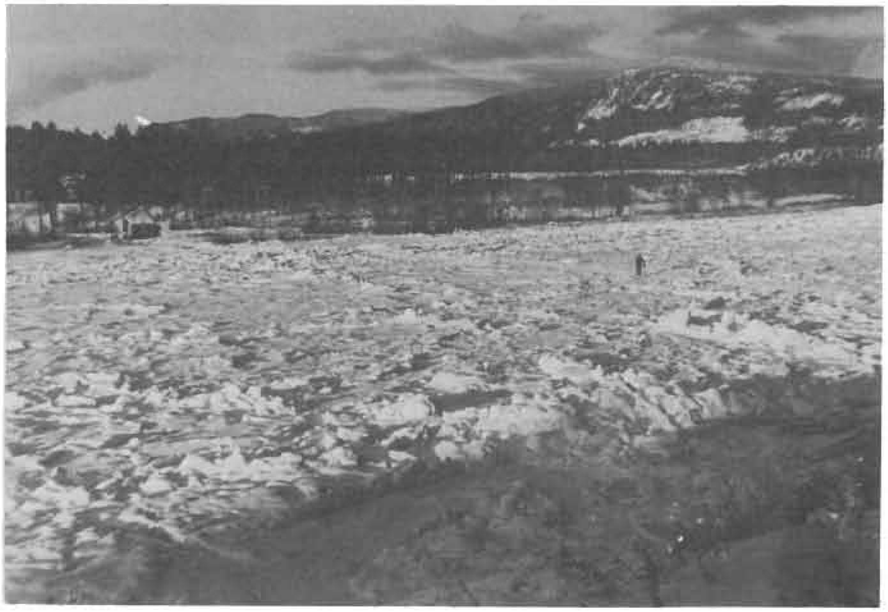

Fig. 4 Stable ice cover at low flow velocities

Fig. 4 shows another reach of the same river with a milder slope. The heat loss is the same, but the frictional heat gain is here too small to maintain an open water surface.

One of the prime concerns of hydro power engineers is to avoid the conversion of mechanical energy into heat. Therefore, one rarely finds conveyance systems where frictional heat needs to be taken into account.

Heat loss and ice production

By summing the heat losses and gains discussed above the total heat loss S_o is obtained

$$S_o = S_1+S_2+S_3-S_4 \tag{8}$$

giving an ice production

$$M_i = \frac{1}{L} S_o A \quad \text{tons/s} \tag{9}$$

where L is the latent heat of fusion, equal to 80 kcal/kg, and A is the free water surface. If A is measured in km^2, (9) gives

$$M_i = 1080\ S_oA \text{ tons/day km}^2$$

FRAZIL ICE FORMATION

Experiments

Experimental demonstrations of frazil ice formation have been made in open air by MICHEL 1963 and in cold rooms by CARSTENS 1966.

Time history Fig. 5 is taken from the latter publication and shows a typical time history of the water temperature in a recirculating flume. The water mass in the flume is giving off heat at a constant rate while a propeller generates sufficient turbulence to prevent thermal stratification.

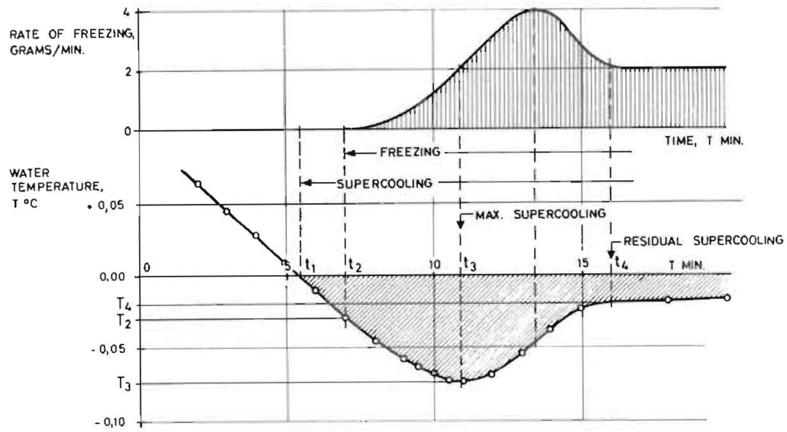

Fig. 5 Typical time history of supercooling

The lower curve in Fig. 5 shows first a linear temperature decrease with time. At time t_1 the water temperature reaches the freezing point, and beyond time t_1 there is supercooling. At time t_2 the curve starts to deviate from the straight line. Since the heat loss is constant, this means that the first ice crystals have formed and released the liquid's latent heat of fusion.

Once started, the growth of ice crystals accelerates. At time t_3 the water temperature reaches a minimum, indicating a balance between released heat and heat loss. Between time t_3 and t_4 the water temperature rises, indicating an excess of released latent heat over the heat loss, until at time t_4 most of the supercooling has disappeared.

From time t_4 on the water temperature T_4 remains essentially constant, and this means that the released latent heat equals the heat loss.

Thus the characteristic T-t curve, after crossing the freezing point as a straight line, levels off to reach a minimum temperature, or a maximum supercooling, of T_3 °C. The curve then rises to a constant temperature T_4 which is lower than the freezing point and represents the residual supercooling T_4 °C.

The upper curve in Fig. 5 shows the rate of freezing, $\frac{dM_i}{dt}$, which is derived from the lower curve T(t) by the formula

$$\frac{dM_i}{dt} = \frac{1}{L} \left(cM_w \frac{dT}{dt} + S_o\right)$$

where M_w is the mass of water in the flume.

<u>Visual observations</u> The first ice crystals, the frazils, became visible at time t_2. These disk-shaped crystals had diameters up to 2-3 mm, and they showed up anywhere in the liquid. The frazils increased rapidly in numbers, but not in size, until shortly after time t_3 the formerly clear water became more or less opaque. Then, probably through collisions, flocs formed that were large enough to gravitate towards the surface, and the water soon became clear again.

The flocs formed clusters that were still to a large extent suspended in the flow, but now with a high concentration near the surface and very small amounts near the bottom. From time t_4 on, only a gradual thickening of the slush layer near the surface was observed.

<u>Rate of cooling</u> The influence of the rate of cooling on the water temperature is seen in Fig. 6. As the cooling rate increases, corresponding increases were observed of i) the maximum supercooling ii) the residual supercooling and iii) the rate of temperature rise.

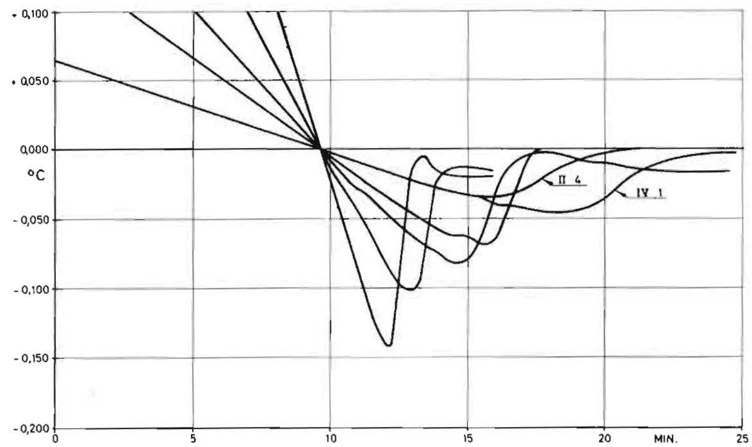

Fig. 6 Effect of rate of cooling on supercooling

<u>Turbulence</u> A qualitative change of the turbulence in the flow was made, and the resulting curves are shown in Fig. 7. Curve A was obtained while the water in the flume was driven by the propeller, at a speed of about 0,5 m/s. A minimum temperature of -0,06°C was observed after 4 minutes of supercooling, followed by a rapid rise to a few thousandths of a centigrade residual supercooling.

The next test started out exactly as the one giving curve A, but this time the propeller was stopped after one minute of supercooling. The flow therefore gradually lost its speed, while the turbulence decayed.

The time history of the water temperature for this decaying flow is shown as curve B in Fig. 7. A minimum of - 0,18°C was observed after about 30 minutes of supercooling. After another 30 minutes the temperature had climbed to - 0,15°C. Now the propeller was started, and within 3 minutes the temperature jumped almost to the freezing point.

During this experiment the water surface gradually became covered with ice, and for the last 15 minutes a strongly supercooled flow prevailed under a 100 % ice cover.

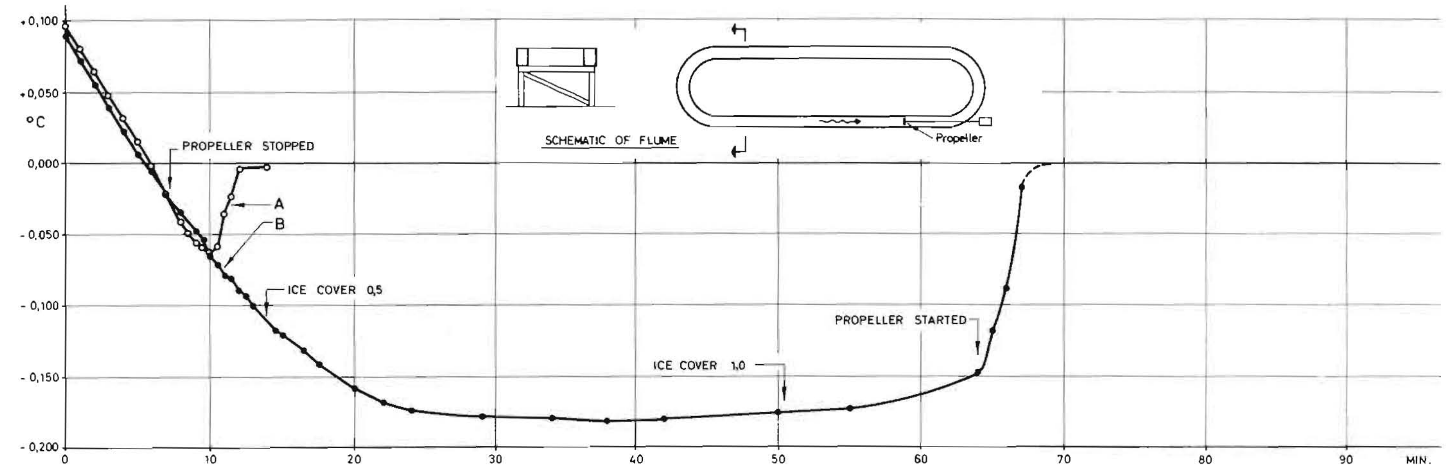

Fig. 7 Effect of turbulence on supercooling

Experiments of the type described above have shown that when a body of water is subjected to a heat loss, the water becomes supercooled. The supercooling is a function of the heat loss and the turbulence of the flow. When the heat loss is reduced or when the capacity for heat transfer, ie the turbulence of the flow, is increased, the supercooling is reduced, and vice versa.

Field observations

It is difficult to observe the same sequence of events in a river as described above for a recirculating flume, however, the same modes of ice do appear in nature as in the experiments. A metamorphasis from individual frazils through frazil slush to more solid ice forms such as pancake ice, takes place in the downstream direction of a river reach (MICHEL 1965).

Each reach of a river is thus characterized by a typical ice regime. In addition to its dependence on the local heat flux and river discharge, the ice regime of any particular river reach is usually heavily influenced by upstream conditions.

Frazil formation The Norwegian experience from rivers is that surface velocities exceeding 0,6 m/s are required to prevent the formation of a solid ice cover (FLATJORD 1964). Consequently, frazil forms continually in river reaches with higher surface velocities than 0,6 m/s, corresponding to bed slopes steeper than about $1{,}5 \cdot 10^{-3}$. However, it is not the mean surface velocity but rather the surface turbulence, which determines whether freezing results in a solid ice cover or in frazil. With a strong wind frazil forms in lakes even with zero mean velocity. As soon as the wind ceases and the turbulence decays, the frazil rises to the surface and forms a continuous cover, preventing further production of frazil.

Frazil slush The cluster or cloud mode of ice is called frazil slush. Referring once more to the Norwegian experience, frazil slush is observed to accumulate at the free surface of rivers where the surface velocity is in the range 0,6 - 1,2 m/s.

For velocities exceeding about 1,2 m/s the turbulence becomes strong enough to suspend the frazil slush and maintain a water surface almost uncovered by ice. The transported ice is now more or less uniformly distributed throughout the whole depth of flow, and anchor ice grows from the bottom.

Frazil Ice Accumulation

A simplified picture of the frazil deposits can now be drawn, based on the information outlined above.

Any case is assumed to fit into one of the three regimes shown in Fig. 8.

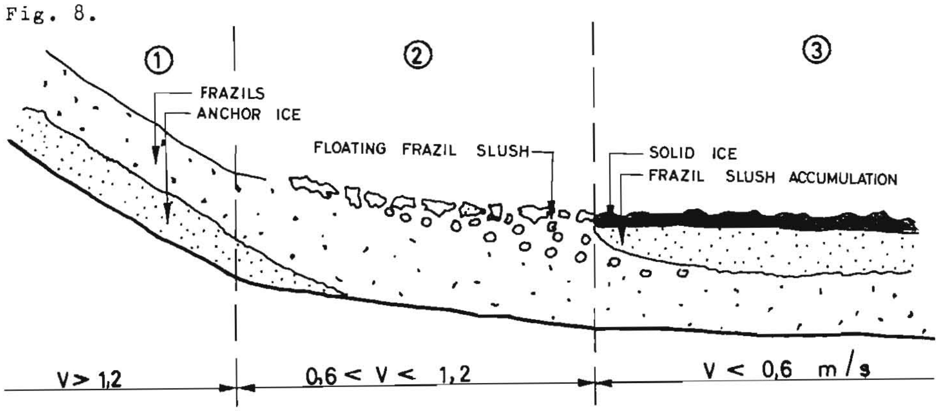

Fig. 8 Regimes of river ice

1. High flow velocities (v>1,2 m/s). Free water surface, strong cooling, supercooling and local ice formation. Some of the locally produced ice, as well as some of the ice from upstream, is accumulated as anchor ice, however, most of the ice moves downstream.

2. Medium flow velocities (0,6 m/s<v<1,2 m/s). The water surface is more or less covered with moving frazil slush, cutting down on the heat loss and the subsequent ice production. The water temperature is at or close to the freezing point. There is little anchor ice and a general tendency for the ice to move on.

3. Low flow velocities (v<0,6 m/s). The solid ice cover prevents large heat losses, so the local ice production is small. Frazil slush from upstream is deposited underneath the ice cover, and there is a general tendency for the ice to accumulate.

Examples of these three modes of ice formation and/or accumulation are shown in Figs. 9, 10 and 4, respectively.

Fig. 9 Free surface and anchor ice buildup at high flow velocities

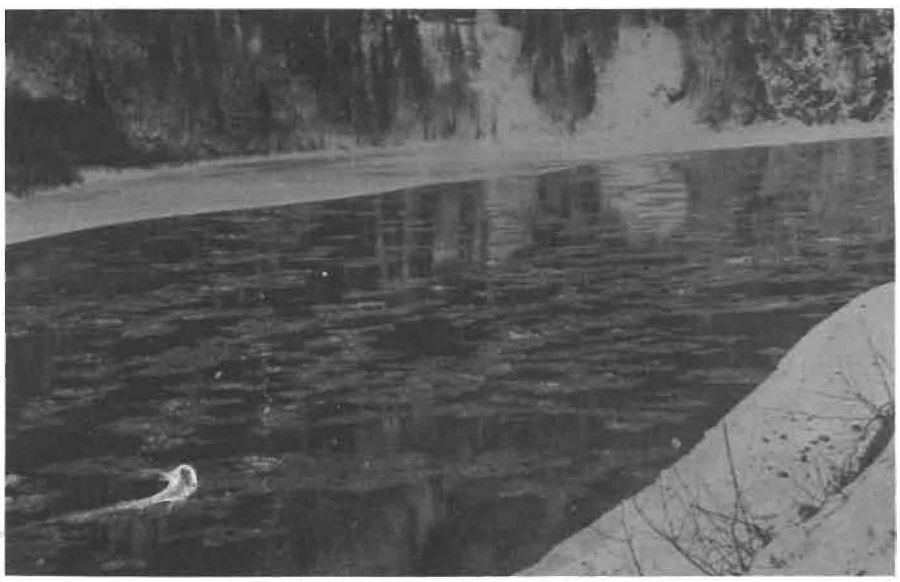

Fig. 10 Floating frazil slush at medium flow velocities

CONCLUSIONS

Estimates of ice production must be preceded by estimates of the net heat loss from the water to the atmosphere. The various heat gains and heat losses through the free water surface have been studied by meteorologists and others, and successful prediction formulae are developed in many countries.

The heat lost by the water results in a change of state after the water has become supercooled. The degree of supercooling is a function of the rate at which heat is lost from the free water surface, and of the intensity of the flow turbulence that transports this heat to the water surface.

When the surface turbulence becomes strong enough, an ice cover cannot form, and the heat loss results in the formation of frazil ice. Field experience indicates a limiting surface velocity of 0,6 m/s for ice covers to form.

The subsequent history of frazils depends on the flow velocities. In swift rivers, with velocities exceeding about 1,2 m/s, some of the frazils are trapped at the bed and builds up anchor ice. In calm rivers, the frazils accumulate underneath the ice cover.

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DISCUSSION

R.S. ARDEN:

Question: With reference to a diagram which indicated that where v<0,6 m/s, no frazil ice occurs, does one infer that anchor ice will also not appear?

Answer: The suggested limiting velocity of 0,6 m/s is of course no absolute limit, it is merely serving as a rough measure. The formation of anchor ice may take place at any velocity, however, it is unlikely to occur at low velocities and very likely to occur at high velocities.

L.S. TSANG:

Question: Dr. CARSTENS' lecture is a good summary of the existing basic knowledge in heat transfer and ice-forming. I venture to suggest that whether Dr. CARSTENS may like to use the Richardson's number approach to define the frazil ice regime instead of simply using flow velocity.

Answer: In order to determine the Richardson's number you must know both the velocity and density gradients, and so it is a cumbersome parameter. It would be very difficult to determine it accurately because of the extremely small density gradients.

B.M. MICHEL:

The author has given a most clear, original well organized review of actual knowledge on frazil formation. The only point I would like to discuss is the fact that he was not able to conclude on the origin of anchor ice as to whether it actually nucleated at the bottom or at the surface of the flow. To me it seems quite clear that the only place that anchor ice could origin indirectly is at the surface of the flow. This is the only place where you would have enough supercooling (many degrees C^{o} below zero), in the thin boundary layer at the surface, to get the best known ice nucelators to start the ice crystals. These nucleus carried down by turbulence would easily stick to protruding rocks in their early stage of growth, from which anchor ice of different forms would build up or grow up in supercooled water. I have expressed that view a few years ago and I am pleased to see from a paper presented here by Ontario Hydro that they have seen frazil nucleating at the surface even when the average water temperature of the flow was above freezing point. This suggest only one origin of ice nucleus.

<u>AUTHOR'S CLOSURE:</u> While preparing the final manuscript, I looked up the original data for the determination of the limiting velocity v = 0,6 m/s of a stable ice front.

The constant limiting velocity was obtained with little spread in depths ranging from 1 to about 5 m. For larger depths the limiting velocity tends to decrease. Thus the Froude number does not appear to be the obvious similarity variable.

Several of the observations refer to moving ice fronts. In those cases the limiting velocity is the relative surface velocity.

The essence of Arden's question and Michel's comments is the nucleation process, which takes place whenever the temperature gradient at the nucleating surface is large enough. While the temperature difference ΔT (equal to the supercooling) surely is largest at the free surface as Michel states, it does not necessarily follow that the temperature gradient $\delta T/\delta n$ is also largest there.

The gradient depends on the Nusselt number $Nu = \frac{l}{\Delta T} \frac{\delta T}{\delta n}$ where l is a measure of the boundary layer thickness on the nucleator. Nucleation at the bottom should occur when i) supercooled surface water is gently carried downwards by large eddies with low vorticity and ii) smaller eddies of high vorticity are generated at the nucleating surface as a result of the no-slip condition on a fixed boundary.

ICE SYMPOSIUM 1970
REYKJAVIK

ICE COVER FORMATION AND ASSOCIATED HYDRO-DYNAMIC EFFECTS IN THE LOWER PART OF THE RIVER RHINE.

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The Hague
Netherlands

Synopsis
In the Netherlands studies have been made on phenomena related to the formation of an ice cover on the various branches of the river Rhine. This was done with special regard to the future situation when, after the completion of the Deltaproject, the watermovement in the area concerned will be changed considerably. In order to study the possibility of discharging ice to the sea the movement of drift ice in the estuaries was observed by marking ice floes with dye.
The influence of a fast ice cover on the discharge distribution and the waterlevels was determined by means of gauge readings and discharge measurements. Conclusions could be drawn about the roughness of the river with a fixed ice cover.
The formation of ice dams in the tidal area was investigated and related to flow conditions.
With the aid of the investigations it was possible to conclude on the operational procedure to be followed after the completion of the Deltaproject.

Résumé
Aux Pays-Bas des études ont été faites sur les phénomènes que se déroulent lors de la formation de la couche de glace sur les différents bras du fleuve du Rhin. Les études ont été orientées vers la situation future où, après l'achèvement du Projet du Delta, le mouvement de l'eau dans le région prise en considération, sera considérablement modifiée.
Afin d'étudier les possibilités d'une évacuation des glaces vers la mer, le mouvement des glaçons dans les estuaires a été observé par marquage des glaçons avec de la peinture.
Pour déterminer l'influence de la couche fixe de glace sur la répartition des débits et les hauteur d'eau, des observations des niveaux aux échelles et des mésures de débits furent employées. Ceci a permis de tirer des conclusions quant à la rugosité du fleuve avec une couche de glace solide.
La formation d'embacles dans la région des cours d'eau avec un régime à marée a été examinée et mise en rapport avec les conditions de l'écoulement.
Les investigations ont permis d'établir la procédure de gestion à suivre après l'achèvement du Projet du Delta.

The presence of ice in the rivers of the Netherlands is not a very common occurence. In the central area of the country, where the main rivers Rhine and Meuse find their way towards the sea, on an average only about 10 days per year the maximum day temperature remains below zero. As a result in the lower reach of the rivers navigation is hampered on an average for small craft only during 8-10 days per year and for more powerful ships this happens during 4-7 days per year. The winters being rather mild on an average during one out of five winters the ice cover in the rivers become land-fast. Then backwater effects due to the ice-cover formation are occuring and the flow distribution over the various tributaries is changed considerably.

The rivers are bordered by districts which lie below the waterlevels prevailing in the rivers. These districts are protected against inundations by dikes along the rivers.

Especially at the entrance of the tidal region conditions are favourable for the start of the formation of a landfast ice cover. This solid mass of ice may grow very thick indeed and usually several ice-dams are formed. When thaw sets in rapidly, or thaw starts in the (more southern) upper reaches of the Rhine and Meuse these dams may cause a dangerous rise in the waterlevel, threatening the dikes and the land behind. In history numerous dike breaches due to these backwater effects have caused serious inundations.

To reduce the danger of the floods and to open up the river for shipping as soon as possible ice-breakers start to break up the fast ice cover from the down stream end before thaw sets in. In the winter of 1860-61 an ice breaker was used for the first time, but not until 1890 were ice breakers used on a larger scale. To carry out the ice breaking programme on the river Rhine and its branches during the winter 1962-63 24 ice breakers were in action.

Since 1850 important regulation works were carried out in the delta of the Rhine and Meuse. It is due to these regulation works and to the ice breaking programme that no dike breaches, resulting from ice dams, have occured since 1861.

When the delta project, which is under construction now, will come into operation it has to be expected that the discharge of drift ice will meet more obstructions. This project provides for the closure of three large sea-arms situated between the Western-Scheldt and the Rotterdam Waterway. Furthermore the waters of the tidal delta will be devided into two separate basins by means of the Volkerakdam. The southern basin will be entirely cut off from the sea and become an almost stagnant fresh water lake. The northern basin which comprises the mouth of the Rhine and Meuse

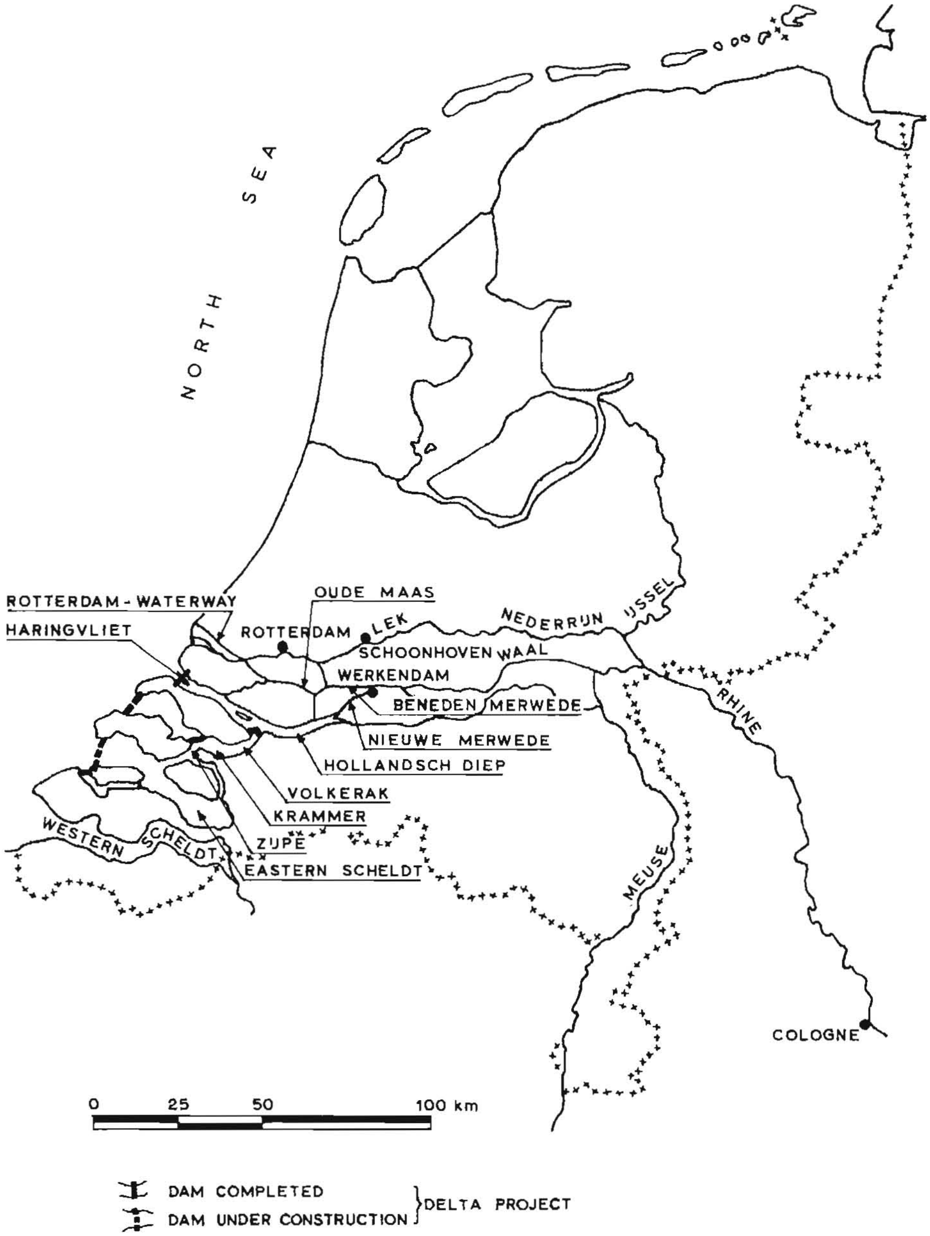

NORTH SEA
ROTTERDAM - WATERWAY
HARINGVLIET
OUDE MAAS
ROTTERDAM
LEK
NEDERRIJN
IJSSEL
SCHOONHOVEN
WAAL
WERKENDAM
BENEDEN MERWEDE
NIEUWE MERWEDE
HOLLANDSCH DIEP
VOLKERAK
KRAMMER
ZIJPE
EASTERN SCHELDT
WESTERN SCHELDT
RHINE
MEUSE
COLOGNE
0 25 50 100 km
DAM COMPLETED
DAM UNDER CONSTRUCTION
DELTA PROJECT

Figure 1

rivers, will continue to be in communication with the sea. In the mouth of the Haringvliet estuary a discharge sluice has been built. Through 17 gates of 56,5 x 5,5 m^2 surplus water of the Rhine and the Meuse will be discharged into the sea. The Rotterdam Waterway connecting Rotterdam with the North Sea will remain open for shipping. Consequently tidal waves will still be able to penetrate inland via this inlet. This scheme will upset completely the ice formation and its discharge. This is the reason that, in recent years, whenever ice conditions occurred studies have been made about the ice phenomena in the Rhine delta especially with regard to the changes in the future.

Soon after entering Dutch territory the river Rhine is splitting up into various branches (see fig.1). The southern-most tributary, called Waal, is the main carrier of water and, if available, of ice. On a part of the northern branches called Nederrijn-Lek and IJssel a fast ice cover is formed in most cases already during early stages of a severe winter period. The IJssel debouches into the almost stagnant fresh water basin "the IJssel Lake" which freezes early even during moderate winters (average about 25 days per year). During such circumstances the ice discharge of the river Waal is of the order of 2.10^6 m^3 or 6.10^6 m^2 per day. This quantity of ice is formed downstream of Cologne because upstream of this city nearly no ice is observed due to the narrows at the Lorelei. It is to be expected that the ice production of the Rhine and especially in the German part of the river will decrease due to the discharge of considerable quantities of warm cooling and waste water coming from the German industrial area bordering the river and its tributaries.

An investigation has been made on the ice production on the Netherlands part of the river. Fig.2 and fig.3 give the increase of the ice discharge coefficient (C) over a stretch of 100 km as a function of the air temperature for two branches of the Rhine, namely the Waal and Nederrijn-Lek. The ice discharge coefficient has been defined by Santema and Valken [1] as the ratio of the ice discharge in m^2/sec and the discharge of water in m^3/sec.

The graphs are based on numerous, but rather rough, estimations of the ice cover coefficient during the winters of 1939-40 and 1953-54. Obviously the Nederrijn-Lek has a greater ice production than the Waal. The slope of the waterlevels is nearly the same for both rivers while the ratio of the depths was 1 : 1,3 and that of the discharge 1 : 3. Over the whole depth the temperature of the water during the period of floating ice was zero centigrade or a little lower.

The ice formed in the stretch downstream of Cologne is discharged to the tidal zone of the delta. As can be seen on fig. 1 the branch Waal

Figure 2

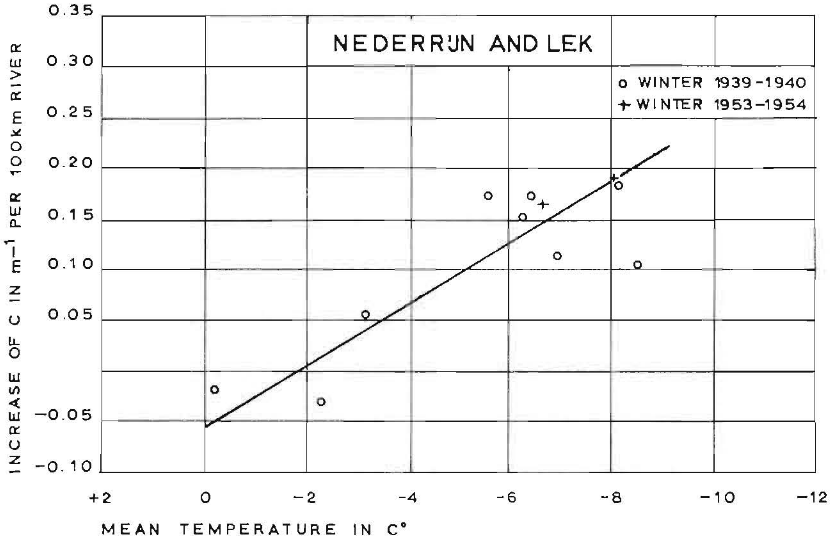
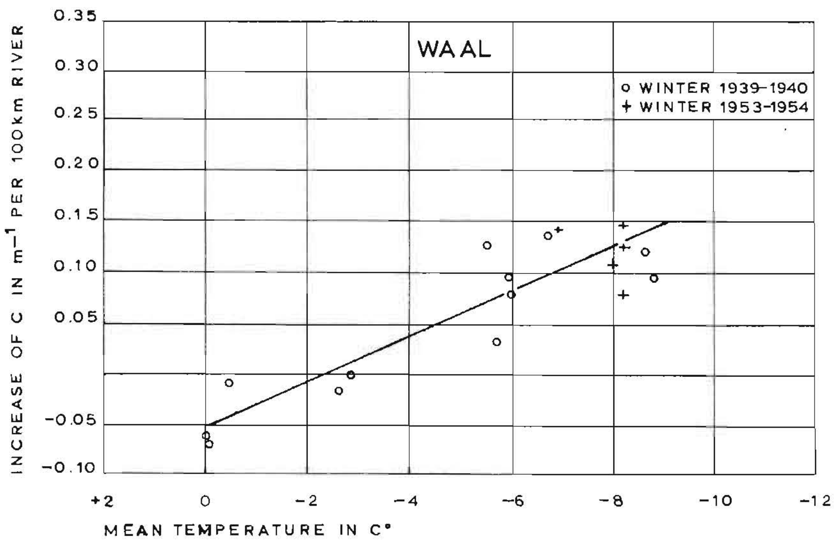


Figure 3



continues in some wide estuaries called Beneden Merwede, Hollandsch Diep and Haringvliet. The river Meuse is also debouching in the Hollandsch Diep. The rivers and estuaries mentioned above contain predominantly fresh water, but there is a considerable tidal movement. The tidal range varies from about 1,75 m at the seaface to about 1,1 m at Werkendam. In this tidal area huge quantities of ice are formed. The total area of the water surface of Nieuwe Merwede, Hollandsch Diep and Haringvliet is about 130 km^2. As a comparision it can be mentioned that the Rhine between Werkendam and Cologne has an area of water of about 80 km^2.

The ice production per unit area in the fresh water tidal zone is even bigger than in the river itself. In the estuaries there are extensive shallow zones which are covered during long periods of the tidal cycle with only a thin layer of almost stagnant water. At HW the ice produced here is moved by the tidal currents and wind to the main channels and the southern (predominant down-wind) shore. Therefore on part of the shallows, especially those along the northern shore of the estuaries, every tide new ice is being formed.

This is the reason why in general on the Haringvliet-Hollandsch Diep stretch ice is observed one or two days before drift ice is occurring on the river Rhine itself. The ice formed in this area together with the ice originating from the river is moved to and fro by the tidal currents and drifted away by the, during severe winter conditions predominant, northern or northeasterly winds. Studies have been made about the ice drift in the tidal areas and the discharge of this ice to the North Sea. Ice floes in the Hollandsch Diep and Haringvliet have been marked from helicopters with dyes. The marked floes were traced back by air reconnaissance. The result of this investigation is shown on fig. 4.

It appears that the majority of the floes marked in the Hollandsch Diep were observed to follow the Volkerak-Krammer-Zijpe-Eastern-Scheldt route to the sea. Only a few floes were found in the eastern part of the Haringvliet and not one floe was observed to find its way to the sea through this estuary. The average ice drift per tidal cycle on the Hollandsch Diep varied between 1 and 6 km. In the Volkerak-Krammer stretch this drift was about 13-23 km per tidal cycle. In the eastern part of the Haringvliet the drift was observed to be 1-3 km per tidal cycle and in the western N.W. orientated part hardly any drift at all was found during severe winter conditions. However when weather changed and with thaw S.W. winds occurred, within one tidal cycle, the marked floes of the western Haringvliet were observed scattered along the adjoining Northsea shores. From this investigation it followed clearly that in addition to the water movement the wind force and direction are the main factors influencing

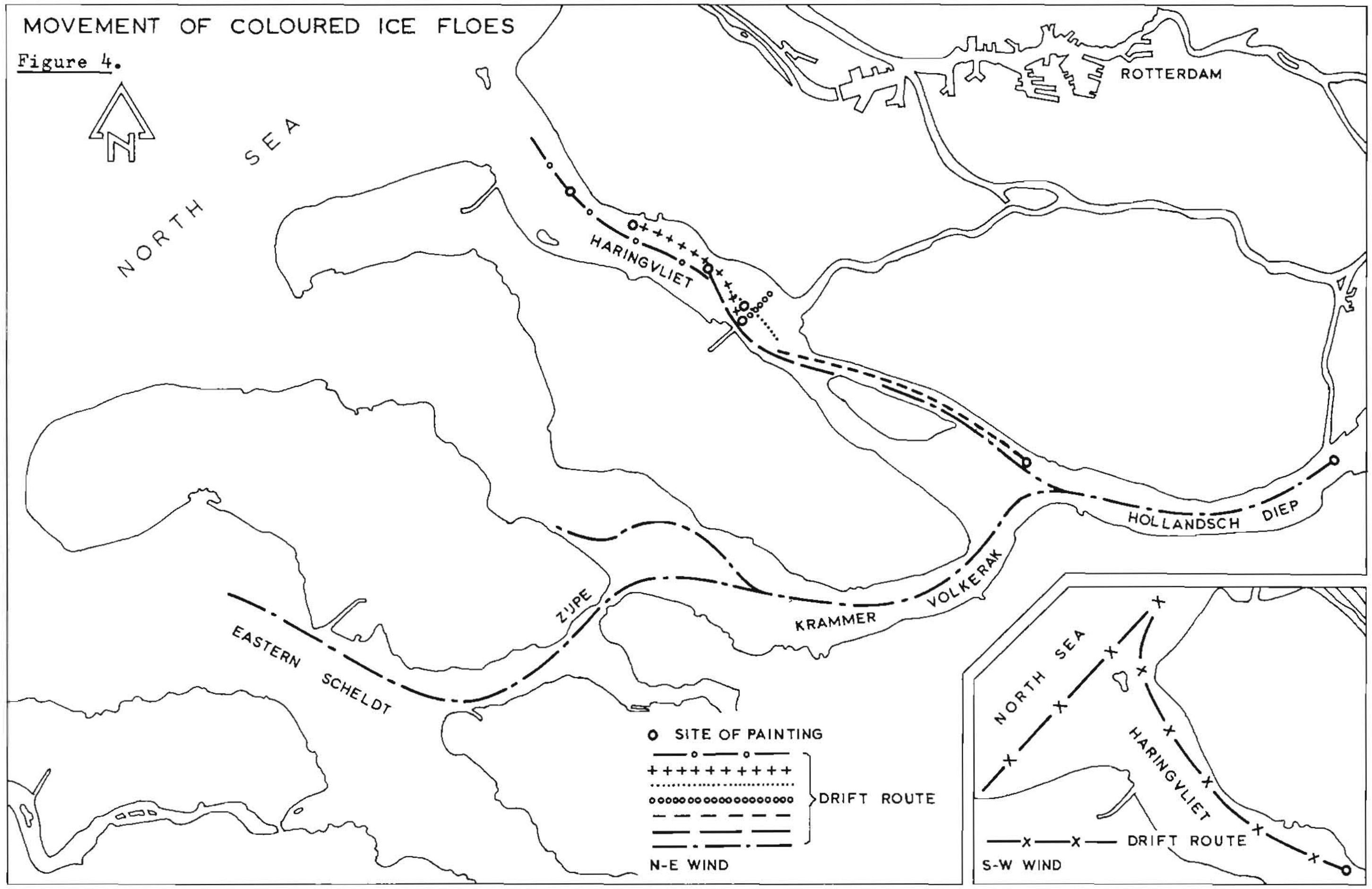

MOVEMENT OF COLOURED ICE FLOES
Figure 4.
N
NORTH SEA
ROTTERDAM
HARINGVLIET
HOLLANDSCH DIEP
VOLKERAK
KRAMMER
ZIJPE
EASTERN SCHELDT
SITE OF PAINTING
DRIFT ROUTE
N-E WIND
NORTH SEA
HARINGVLIET
DRIFT ROUTE
S-W WIND

Figure 4

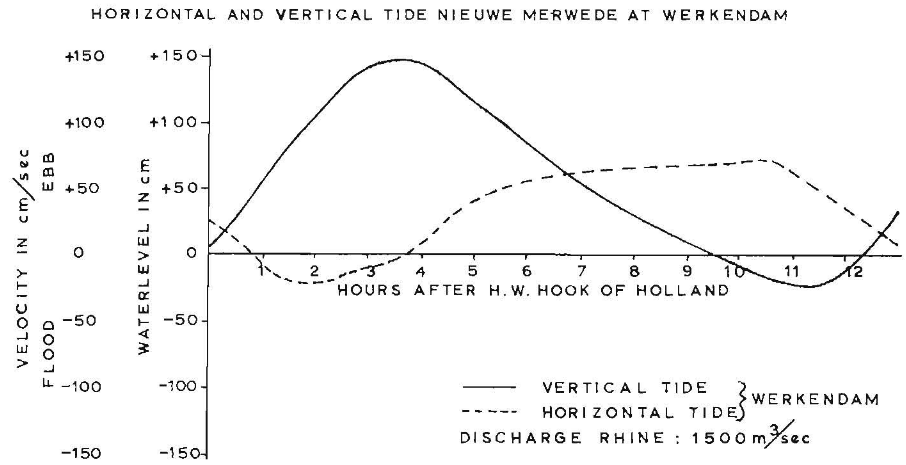


Figure 5.

the drift of ice in the wide tidal estuaries.

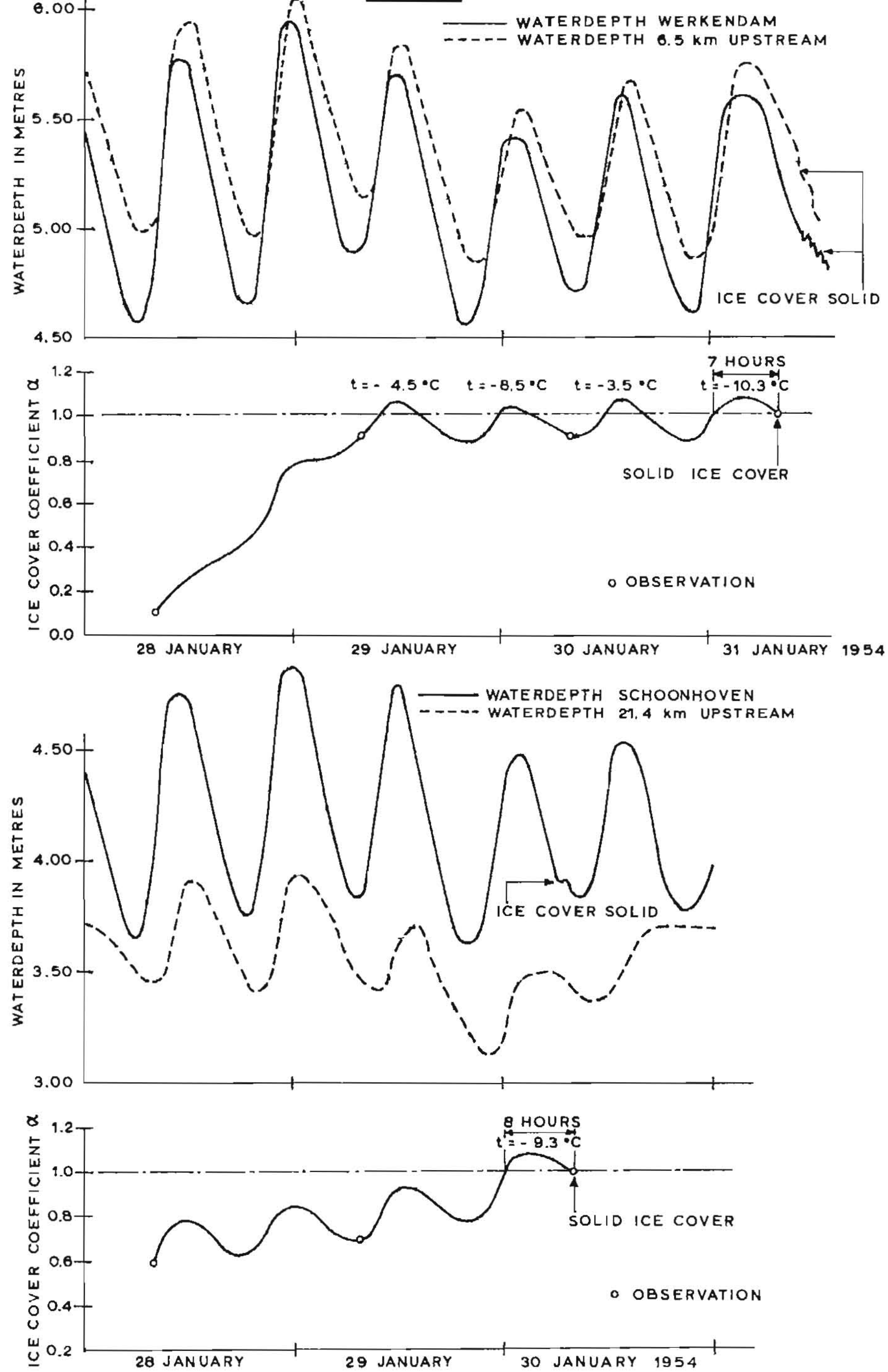
In the transition zone between the tidal area with twodirectional flow and the river with a constant unidirectional flow there is a river section in which during a relatively long period of the tidal cycle the flow velocities are very low. A typical example of a velocity curve in the Nieuwe Merwede-Werkendam area is given in fig. 5. During the flood period of the tidal cycle the velocities are low and they tend to pack the ice together in the relatively narrow river section, thus increasing the ice cover coefficient. This weak flood period is followed after HW by a long period of slack water during which,if the conditions are favourable, the ice may freeze together to a solid sheet.

From a number of observations in the area concerned it appeared that the following combination of conditions is critical for the formation of a landfast ice cover in the upper reaches of the tidal zone of the delta (also in the Nederrijn-Lek area):

1) The ice cover coefficient has a value of 1 (or somewhat greater due to packing) during a period of 5 or 8 hours.
2) the average temperature during that period is lower than -9° C.
3) the water velocity during the period concerned is not more than 50 cm/sec

In fig. 6 an example is given of the associated effects leading to the formation of a landfast ice cover near Werkendam and in the Lek during the 1954 winter.

Figure 6

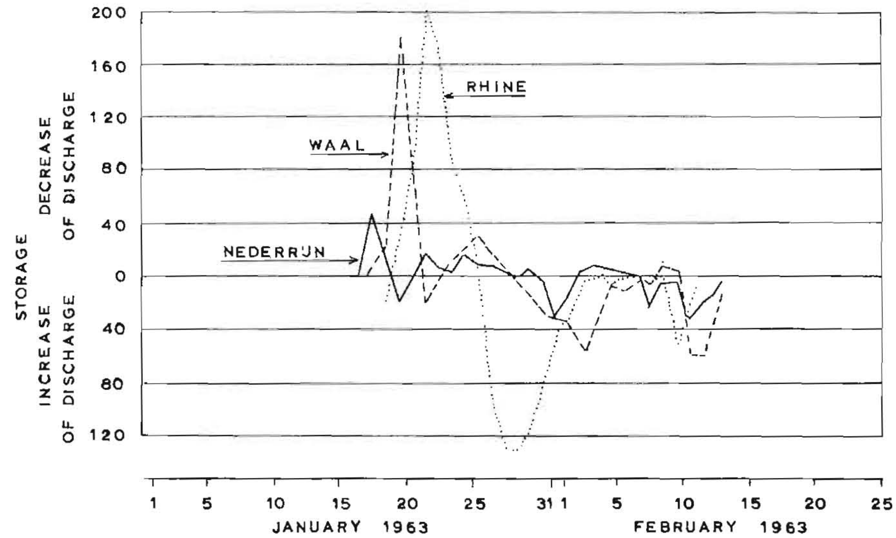

WATERDEPTH WERKENDAM
WATERDEPTH 6.5 km UPSTREAM
WATERDEPTH IN METRES
6.00
5.50
5.00
4.50
ICE COVER SOLID
ICE COVER COEFFICIENT α
1.2
1.0
0.8
0.6
0.4
0.2
0.0
7 HOURS
t = - 4.5 °C
t = -8.5 °C
t = -3.5 °C
t = -10.3 °C
SOLID ICE COVER
o OBSERVATION
28 JANUARY
29 JANUARY
30 JANUARY
31 JANUARY 1954
WATERDEPTH SCHOONHOVEN
WATERDEPTH 21.4 km UPSTREAM
WATERDEPTH IN METRES
4.50
4.00
3.50
3.00
ICE COVER SOLID
ICE COVER COEFFICIENT α
1.2
1.0
0.8
0.6
0.4
0.2
8 HOURS
t = - 9.3 °C
SOLID ICE COVER
o OBSERVATION
28 JANUARY
29 JANUARY
30 JANUARY 1954

Once a fast ice cover is formed in the Werkendam-Nieuwe Merwede region the fast cover grows upstream due to juxtaposition of floes on an average at a rate of about 25 km per day on the Waal. On the river Meuse an average upstream movement of the edge of the fast ice cover of about 18 km per day is observed.
As soon as the ice become landfast ice breakers start to proceed from the downstream edge of the ice cover, in order to remove ice-dams and to clear the river for the free discharge of water. Ice breaking is only done during the ebb tide since during flood the danger exists that the ice-breakers will become trapped. If a progress is made of 3½ km per tidal cycle 300.000 m^3 (about 1000.000 m^2) free ice is produced.

A careful analyses has been made of the factors incluencing the upland discharge and its distribution over the various branches of the river system during the 1962-63 winterseason.
As the normal stage discharge relationship of the Dutch gauging stations was disturbed due to the presence of an ice cover all possible methods were used to determine the actual situation. The Ruhrort gauge on the German Rhine was not disturbed by the ice cover and the discharge at that station could be used as a known boundary condition.
The discharge observed in the uninfluenced part of the German Rhine however could not be used without correction as given quantity at the bifurcation points in the Netherlands. When a landfast ice cover is being formed in a relatively short period the hydraulic radius of the river section concerned is roughly halved and the ice cover is exerting an extra resistance. Consequently the water levels will rise. This increase of waterlevels can develop in such a way that during a few days a substantial part of the discharge is stored in the river section. Due to this storage, downstream of the section in which the ice cover is being formed, the discharge may temporarily decrease considerably. During break-up the reversed will happen.

In fig. 7 the effect of this storage is given for river sections of the upper Rhine, Waal and Nederrijn. From this figure it follows that considerable quantities can be involved in these processes. The formation of the ice cover in the period January 17 - January 22 caused a sudden drop in the discharge of the lower sections of the river of about 150-200 m^3/sec. This phenomena resulted in a unprecedented serious intrusion of salt sea water in the tidal area of the delta. At the intake of the Rotterdam municipal drinking water supply system even at L.W. a chlorinity of 3000 - 4000 p.p.m was observed and actually this water had to be fed into the distribution system. The chlorinity of the drinking water in the city of Rotterdam increased during this period to a maximum of over

Figure 7



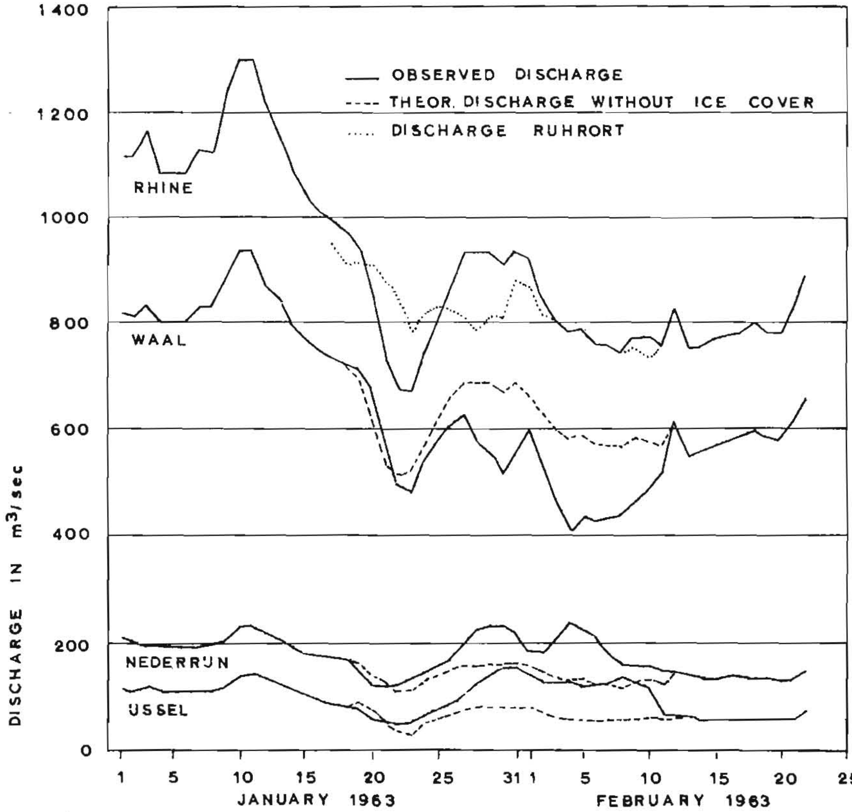


Figure 8

3000 p.p.m.

The graphs of Fig.7 were deduced from the waterlevels observed along the sections and the known storage area in the river itself and in the adjecent storage area, as harbours etc. Various checks were made with the aid of discharge measurements under the ice cover. A good agreement was found. With the aid of these storage calculations the discharge at the site of the reference gauging station for the Netherlands at Lobith could be reconstructed. With this discharge its distribution over the various branches under normal conditions is defined.

The deviations from this normal discharge distribution have been determined by means of measurements carried out in holes in the ice and just upstream of the edge of the solid ice cover.

Other information could be collected about the discharge of the various branches, when after the end of January the ice breakers had cleared the lower part of the rivers and the stage discharge relation curve at the stations along these sections was reliable again.
With the aid of these observations for a number of gauging stations new "ice conditions" stage discharge relationships were determined which of course varied with the changes in the ice condition. Using the observed waterlevels and the "ice stage discharge relationship" it was possible to interpolate between the observed discharges in the various branches. When keeping in mind that at the bifurcation points the condition of continuity should hold a good estimate could be made of the distribution of the discharge over the various branches.

In fig. 8 the results of these investigations are given. As a comparison also the discharge distribution under normal conditions is indicated.

From this figure it appears that, due to the ice cover, the discharge along the Waal was reduced considerably. A minimum of 400 m^3/sec was observed. The minimum discharge of this branch observed under normal conditions is 490 m^3/sec. The deficit of the Waal discharge was diverted along the two northern branches IJssel and Nederrijn.
These changes in the discharge distribution and its associated effects, as the observed waterlevels, could not be fully explained by the decrease of the hydraulic radius due to the ice cover. Therefore a study was made of the variations of roughness coefficient of de Chézy (C) of the river section concerned.

With the aid of the observed waterlevels and the observed and reconstructed discharge for a number of riversections, during the ice conditions, C_i values were determined using the formula

$$Q = b.h.\ C \sqrt{R\ I}$$

in which b = width of the river bed h = waterdepth
I = slope of water level R = hydraulic radius

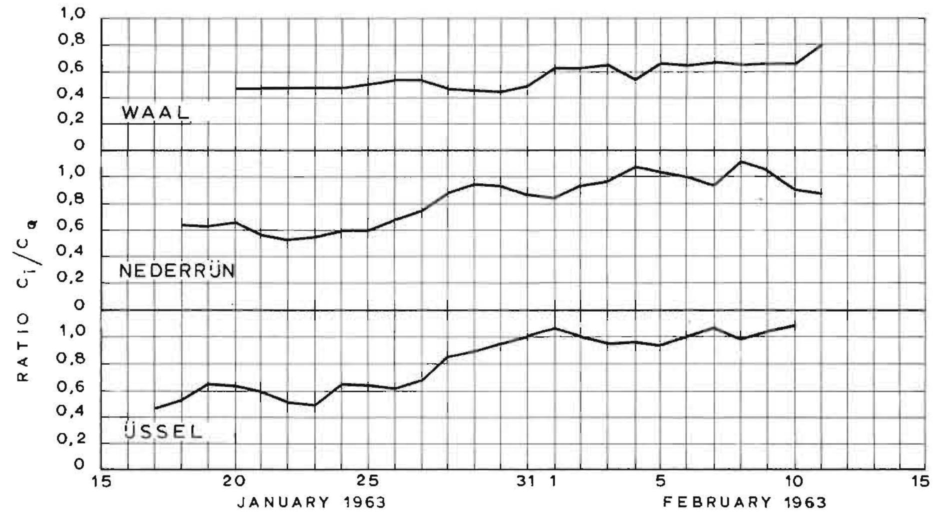


Figure 9

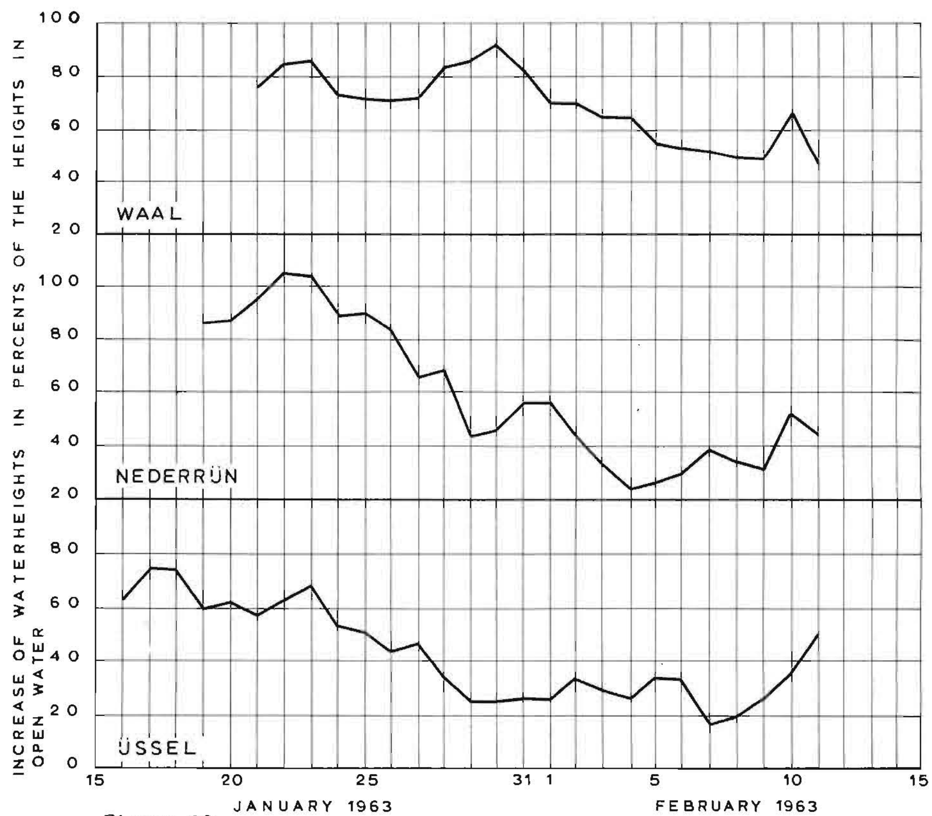


Figure 10

Under ice conditions

$$R = \frac{bh}{2b + 2h} \qquad \frac{h}{2}$$

If Q, h, b an I have been observed

$$C_i = \frac{Q}{bh\sqrt{\frac{h}{2}\ I}}$$

In order to compare these C_i values with the roughness of the river sections under normal conditions the coefficient of de Chézy is determined for the situation with the same discharge in each section without an ice cover (C_q).

In fig. 9 the variation of the ratio $\frac{C_i}{C_q}$ for some river sections is given. From this figure it can be concluded that

1) the C_i values are considerably lower than the corresponding C_q values
2) the C_i values are lowest just after the solid ice cover has been formed.
3) with the continuation of the ice period the C_i values rise gradually.

Furthermore it followed from more detailed investigations that there is a tendency for C_i to raise steeper during a period of thaw with relatively higher water temperatures. If after such an intermediate period of thaw, lower temperatures are reoccurring C_i has not the tendency to fall again to lower values.

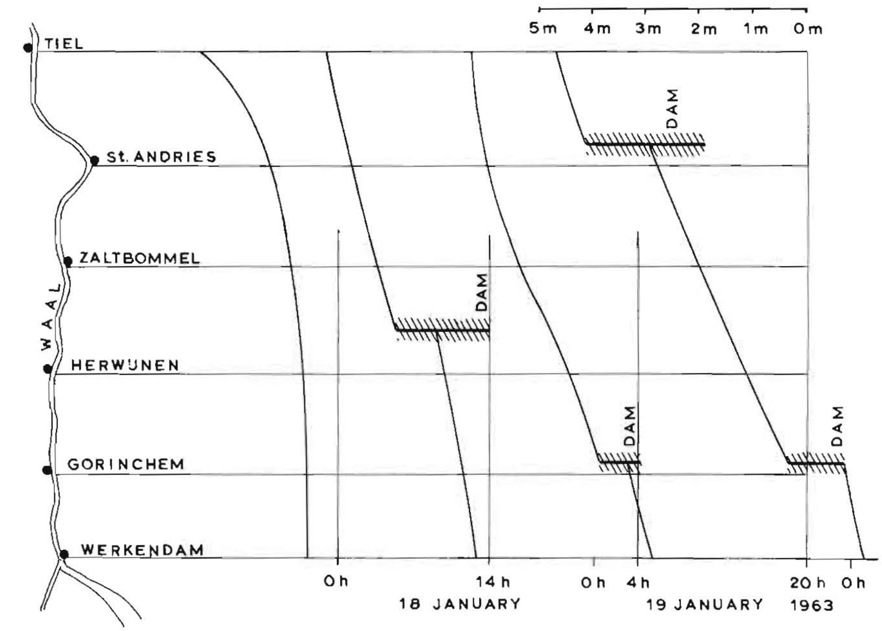
From the investigations it appeared that on the Waal relatively the lowest C_i values were observed and these low values persisted during a longer period than in the other branches. This is the explanation for the change in the distribution of the upland discharge as shown in fig. 8.

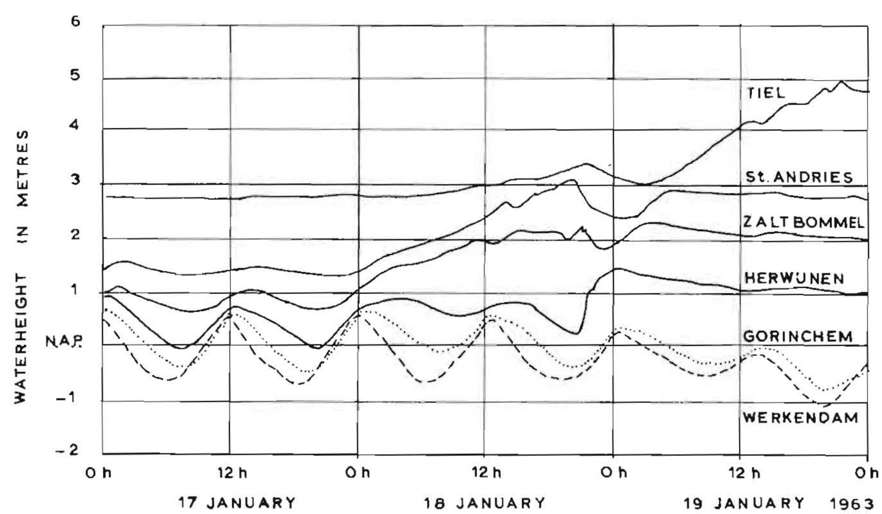
Parallel with the variations of C_i is the variation of the water levels. According to the decrease of the hydraulic radius an increase of the waterlevels with 30% could be expected. Fig.10 shows the development of the increase of the waterlevels during ice conditions (h_i) in per cents of the waterlevels under normal conditions (h_q) during the continuation of the ice winter. From this figure also the departure of the behaviour of the Waal branch is clear.

In addition to the consequences of a landfast ice cover the formation of ice dams and its associated effects on the water movement has been a subject of special interest.

Figure 11

HISTORY OF INFORMATION OF ICE DAMS IN 1963





The investigations concerning ice dams are concentrated especially to the tidal area since, due to the delta project, the conditions are changing considerably in that area. The aim of the investigations is to determine the hydraulic conditions under which ice dams are being formed and to find out which river sections will be vulnerable in the future for the formation of these dams.

On fig. 11 an example is given of the history of the formation of some ice dams during the 1962-1963 winter. In the lower part of this figure the recorded waterlevels at the gauges along the Nieuwe Merwede - Waal section are given while in the upper part are shown the locations of the gauges and, at four instants, the longitudinal profiles of the water-level with the location of the ice dams.

On January 17th the tidal movements was not yet disturbed by the presence of a fast ice cover. After the midnight H.W. on the 17th a land-fast ice cover was formed in the Werkendam area, which resulted in a general divergency of the waterlevels as compared to the normal tidal movement in the entire river section. At about 7 a.m. on the 18th an ice dam was formed between Herwijnen and Zaltbommel causing a deviation of the waterlevels up and downstream of this dam. The maximum differential head between the Herwijnen and Zaltbommel gauges was about 2 m at about 21 hours on the 18th. Then it was LW at the downstream side.
Apparently the dam could not withstand this head and it collapsed. This resulted in a sudden rise of the downstream waterlevels and a fall up-stream. Due to the high velocities in the downstream section and probably the ample supply of ice a new dam was formed, around H.W., a few hours later in the section between Herwijnen and Gorinchem. This dam damped out almost all tidal movement upstream.
At about noon of January 19th another dam was formed between St. Andries and Tiel.
For a number of years the history of the formation of ice dams has been reconstructed. Using the gauge readings and the reconstruction of the dis-charge distribution it was possible to determine the hydrodynamic condit-ions occurring during the formation of the ice dams. Using the work of Kivisild published at the eighth I.A.H.R. Congress held in 1959 in Montreal [2] , [3] the Froude number was chosen to describe the hydrodynamic situation. The results of this investigation are shown on figure 12. A good agreement is found with the findings of Kivisild. Also for the river Waal an average critical Fround number of

$$Fr = \frac{V}{\sqrt{g\,h}} = 0.08$$

is found for the formation of an ice dam. The single observations vary

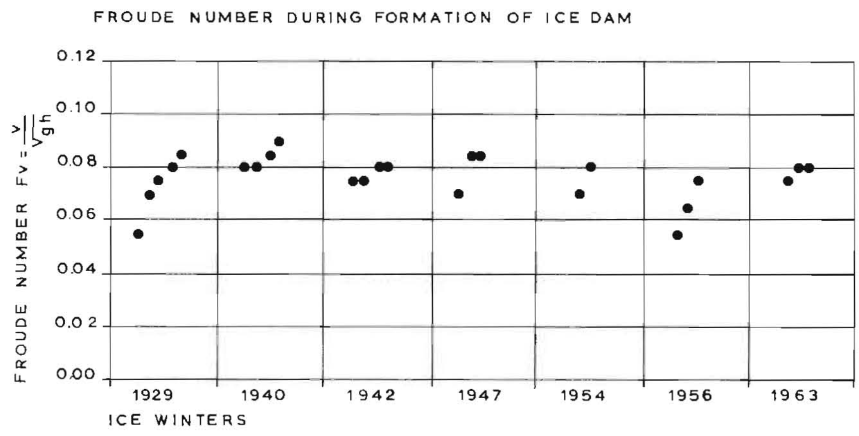


Figure 12.

mainly between $F_r = 0.06$ and $F_r = 0.09$
As of the 1956 winter good detailed information was available about the hydrodynamic situation the formation of ice dams during that winter was reproduced in an analogue computer. This computer is specially built for the computation of the tidal movement of the Delta area. An ice dam could be inserted as a special section with a variable resistance. It appeared that the methods used for the determination of the hydrodynamic situation in the other winters were giving reliable results.

With the aid of the analogue computer a study was made of the hydrodynamic conditions occurring in the riversections concerned after the completion of the Delta project.
During low upland discharges, which prevail during ice conditions, the discharge sluices in the Haringvliet estuary will be closed and the Froude number in the Werkendam-Zaltbommel area will be lower than the critical value. During higher upland discharges the Froude number will rise, but, within certain limits, it can be controlled with the aid of the Haringvlietsluices and other means in the river system. The investigations on this subject are not yet completed.
From the studies described in this paper the following general conclusions could be drawn:
- The discharge route of the ice to the sea through the southern Delta will be blocked due to the construction of a dam in the Volkerak
- The possibility of discharging ice through the Haringvliet and the discharge sluice in the mouth of this estuary is highly problematic during continuing freezing periods with prevailing NE to E winds.

- If in the Haringvliet and Hollandsch Diep the ice cover should be removed in a systematic way the ice production of these wide estuaries should be considerably bigger than the production of the rivers.
- With the aid of gauge readings and the observed roughness of the river a good estimate can be made of the distribution of the upland discharge over the various branches and its consequences for the hydrodynamic situation
- The formation of ice dams is related to the hydrodynamic situation in the river section concerned ($0,06 < Fr < 0,09$)

From these conclusions it was possible to determine a general operational procedure which can be followed during ice periods after the completion of the Delta project. The main points are:
- During the freezing period no efforts should be made to break and discharge the ice in the Hollandsch Diep and Haringvliet.
- Ice breaking activities for safeguarding the free discharge of upland water should be confined to the narrow (and deep) rivers of the Oude Maas, Beneden Merwede - Waal route.
- The hydrodynamic situation should be followed carefully and all possible means should be used to avoid a critical situation for the formation of ice dams.
- During break-up and S or S.W. winds as soon as possible the ice of Hollandsch Diep and Haringvliet should be discharged to the North Sea.

litterature

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[2] H.R. Kivisild. Hanging Ice Dams. Eighth IAHR Congres Montreal 1959. Proceedings Vol III. Seminar I Ice Problems in Hydraulic Structures page 1-51-1/3

[3] H.R. Kivisild. Hydrodynamical Analysis of ice floods. Eighth IAHR Congress Montreal 1959 Vol 2 Subjects C and D and Free papers page 23-F-1/30

Discussion by R.O. Ramseier on paper 3.0 by H.M. Oudshoorn

Question:
In your paper you suggested that after a few decades the water temperature of the river Rhine is high enough to eliminate the ice problems. Would it be possible to accelerate the thermal conditioning process by use of strategically placed nuclear power plants an other installation producing hot waste water.

Reply:
The best location of power plants depends on quite a number of factors such as planological aspects, biological aspects, climatological aspects, transport of energy to the user, circulation system and cooling capacity of the river under summer (high temperature) conditions, etc. etc.
The influence of the thermal processes on ice conditions is just one, and even not the most important, point of consideration in the planning of the power system in the overpopulated area boundering the Lower Rhine.

ICE SYMPOSIUM 1970
REYKJAVIK

FORMATION OF PRIMARY ICE LAYERS

René O. Ramseier*	Laboratoire de Mécanique des Glaces	Québec
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ABSTRACT

An understanding of the formation of primary ice is essential to the prediction of the mechanical properties of ice covers. Certain meteorological and hydrodynamic parameters can be correlated with the texture of primary ice. A review of laboratory results is given indicating the effects of water temperature and current on primary ice crystal formation. Its texture is correlated to air temperature at time of formation.

RESUME

La compréhension de la formation de la glace primaire est essentielle pour prédire les propriétés mécaniques des champs de glace. Certains paramètres météorologiques et hydrodynamiques sont en corrélation avec la texture de la glace primaire. Une revue des résultats obtenus en laboratoire est donnée pour illustrer l'effet de la température de l'eau et des courants sur la formation des cristaux de glace primaire. Sa texture dépend aussi de la température de l'air au moment de la formation.

* This work was done while on Educational Leave from the Department of Energy, Mines and Resources. Permanent address: Department of Energy, Mines and Resources, Inland Waters Branch, 562 Booth Street, Room 24, Ottawa, Canada.

The primary ice layer has been defined as the initial ice skim forming on a body of water. On a calm surface its growth is predominantly in the horizontal plane forming a very thin ice layer. Where there is a current the primary layer begins with the formation of border ice and also frequently with the freezing of frazil slush which has accumulated on the surface.

An understanding of the formation of primary ice is essential to the prediction of the mechanical properties of ice covers since the primary layer imposes its texture on the secondary ice which grows parallel to the heat flow; usually perpendicular to the water surface. In addition, certain meteorological and hydrodynamic parameters can be correlated with the texture of primary ice. The most important of these parameters are air temperature, wind velocity and current. Impurities in the water also have an effect on the texture of the ice but their role is not yet well understood.

Shumskii (1964) has given an excellent description of the formation of primary ice which needs no further elaboration here. A knowledge of the amount of supercooling and the thickness of the supercooled water layer is needed for quantitative analysis. Early work by Altberg (1963) and Devik (1944) indicate that the amount of supercooling varies between a few hundredths of a degree centigrade for disturbed water to 1C for calm water. Different amounts of supercooling cause different growth velocities of the c-axis and a-axes as well as different shapes of the crystals forming in the supercooled surface layer of an undisturbed body of water. Table 1 gives the free growth velocity for the c- and a-axes, with the ratio V_a/V_c* added (Pruppacher, 1967; Knight, 1968). In nature, the small amounts of supercooling cause a rapid growth in the a direction, i.e., parallel to the basal plane. If impurities such as NaCl are added to the water the growth velocity in the c direction is greatly reduced while it is unaffected in the a direction (Sperry, 1965).

Table 2 gives the shape of the crystals formed at the various degrees of supercooling for ice grown from pure water. The table was compiled from work by Macklin and Ryan (1966) and Williamson and Chalmers (1966). Although these data were collected under laboratory conditions and on particular crystals, they should reflect similar occurrences in nature.

At nucleation a certain number of minute spherical ice crystals form (Michel, 1967) which will grow to disks (Table 2). These crystals grow rapidly in the supercooled layer until they come in contact to form a continuous thin ice sheet. Since the crystals tend to float, the ice layer will have a preferred vertical global orientation of the c-axis. The various possible orientations of the primary ice layer have been classified by Ramseier (1970) and Michel

*Growth velocity in the a direction divided by growth velocity in the c direction.

and Ramseier (1970) by means of a Wulff net to determine the angular relationships between grains. During growth crystals will interfere with each other causing some to have their c-axes inclined at some angle to the vertical. As the amount of supercooling increases, both the growth velocity and number of nucleation centers will increase giving rise to further interference, reducing the crystals tendency to float. This will result in a random distribution of the c-axis on a Wulff net plot.

Two types of tests were performed to document the above conclusions. Ice was permitted to form in a tank filled with tap water at air temperatures of -10 and -30C (Ramseier, 1970). Still and motion pictures were taken using polarized light during the process of ice crystal growth. Figure 1 shows a series of pictures taken during the course of growth at an air temperature of -10C. Needles first appear, their c-axes tending to be at some angle to the water surface. As the skim becomes slightly thicker the dendrites become visible and cover most of the areas in view. This type of primary ice will grow on a calm surface with a small air-water temperature gradient. The crystals tend to be several centimeters in length and contain a large amount of substructure caused by interference of secondary and tertiary dendrite arms. A Wulff net plot reveals a preferred vertical global c-axis orientation.

Figure 2 shows the growth of ice at an air temperature of -30C. Growth is rapid with many needles initially visible. Areas between the needles are then filled in with dendrites at some angle to the water surface. The crystals tend to be smaller with less visible substructure. The c-axis orientation is random. This type of ice forms on a calm surface with a large air-water temperature gradient.

Other forms of primary ice formation are due to the presence of snow or frazil which cause the growth of small grains with random c-axes. Wind will modify the thermal regime causing a greater heat loss from the water surface and thus enhancing growth. With a strong enough wind there will be mechanical action among the free-growing crystals or with the water, reducing the crystal size and favouring a random orientation of the c-axes in the primary layer.

Pariset and Hausser(1961) and Michel (1966) have given a general description of the formation and evolution of ice covers in rivers. They treat the evolution of the solid ice cover from the thermo and hydrodynamic viewpoint, but do not cover the evolution of the primary layer or its crystallographic properties.

A further factor influencing the formation of the primary layer is current. Miksch (1969) studied the solidification of ice dendrites in flowing supercooled water. He found that the primary and secondary dendrite axes are deflected toward the current. This can be explained by the heat flow. The supercooled water is warmed by the growing ice causing the tip of the downstream dendrite to be exposed to warmer water and therefore have a smaller growth velocity than the upstream side. Dendrite growth rate increases with higher flow velocities. At a temperature of -0.3C the growth rate at a flow velocity of 20 cm s^{-1} is 10 times greater than at 0.37 cm s^{-1} (Fernandez and Bardulan, 1967; Miksch, 1969). A preferred crystallographic orientation of the ice cover may develop due to the preferred growth direction. Some evidence of this has been found in nature (Ramseier, 1970; Michel and Ramseier, 1970). This results in columnar ice having a preferred aligned horizontal c-axis orientation. More work needs to be done to definitely establish that the aforementioned conditions cause this ice type.

The formation of frazil has been well documented in the literature (Michel, 1966; Carstens, 1966; Bukina, 1967; Chalmers and Williamson, 1965; Ramseier, 1970) and will not be further elaborated upon here. In terms of primary ice, it will have a random crystallographic orientation and will be fine-grained.

In order to predict the presence of a possible ice type, one would have to know the air temperature, wind velocity and the current. These factors have to be correlated with the texture of the ice (grain size, crystallographic orientation, grain shape) in order to determine the mechanical properties of the ice (Ramseier, 1970). The prediction is not as simple a matter as may at first be assumed. Attempts have previously been made to estimate lake ice strength (Andrews, 1963). It is hoped that the approach taken here will eventually aid the forecaster and the engineer to determine the mechanical properties of an ice cover.

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TABLE 1

Free Ice Growth Velocity Component of c-axis and a-axes

T (C)	α (deg)	V_c ($m\ s^{-1}$)	V_a ($m\ s^{-1}$)	V_a/V_c
-1	1.5	0.0004	0.015	38
-2	4	0.004	0.060	15
-3	7	0.023	0.185	8
-4	9.5	0.063	0.379	6
-5	12	0.136	0.634	4.7
-6	14.5	0.242	0.935	3.9

TABLE 2

Characteristics of Ice Grown in Pure Water as a Function of Supercooling

-0.2 C	Disk-shaped crystals ≈ 0.01 cm thick. Disks have a diameter of 0.2 cm and grow to 1 cm in ϕ after one to two minutes.
-0.4 C	Small protuberances (no preferred growth direction) appear on the edge of the disks, some may start to branch.
> -0.6 C	Dendritic shape having preferred growth direction in the $<11\bar{2}0>$.
> -1.0 C	Dendritic growth dominates; secondary and tertiary branches appear.
0.9 C - 2.5 C	Plane stellar dendrites or dendrite sheets.
2.5 C - 5.5 C	Simple double pyramids (pyramidal caps).
5.5 C -	Complex double pyramids showing secondary and higher order nonrational growth.

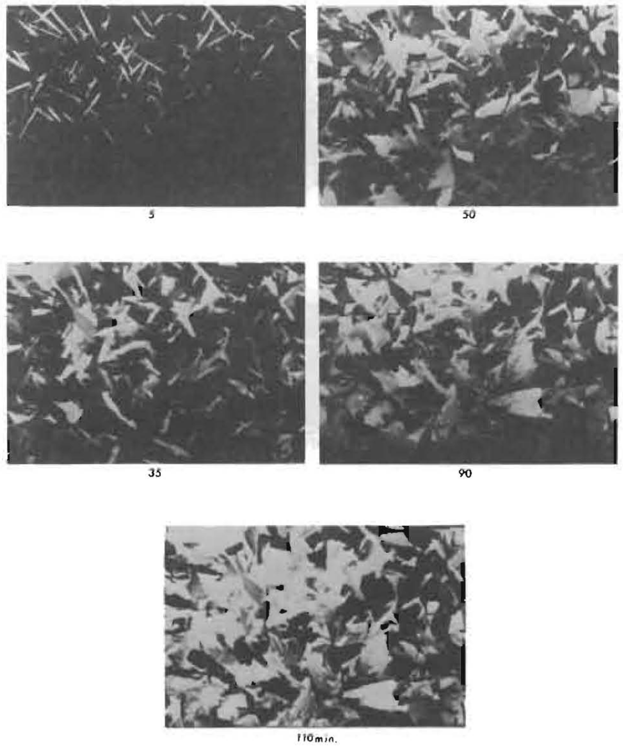

Figure 2 - Still picture sequence of primary ice forming at an air temperature of -30C in calm water. The number underneath the individual pictures indicate the time at which the picture was taken after nucleation was observed.

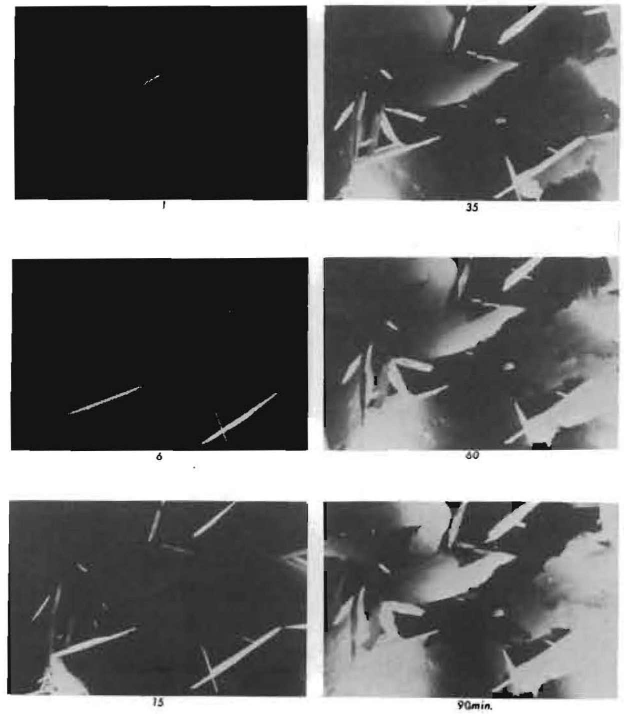

Figure 1 - Still picture sequence of primary ice forming at an air temperature of -10C in calm water. The number underneath the individual pictures indicate the time at which the picture was taken after nucleation was observed.

ICE SYMPOSIUM 1970
REYKJAVIK

CHANGE OF VELOCITY DISTRIBUTION IN A CROSS-SECTION OF A FREEZING RIVER AND THE EFFECT OF FRAZIL ICE LOADING ON VELOCITY DISTRIBUTION.

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SYNOPSIS

The change of velocity distribution in the cross-section of a freezing river is discussed. Field data showing such a change in velocity distribution are shown. The loading of frazil ice in an ice-covered stream significantly increases the boundary shear and greatly affects the magnitude and distribution of the velocity of the flowing water. Field data showed that the presence of an ice cover and a high loading of frazil ice can easily double the maximum velocity of an open channel flow for the same rate of discharge. Velocity distribution equations for water flow under an ice cover with a high, or a low, frazil ice loading are developed and were qualitatively verified by field data.

SYNOPSIS

Le sujet traite du changement de la répartition de vitesse dans la coupe en travers d'une riviere gelante. On y montre les résultats d'une expérience faite sur place qui met en évidence ce changement dans la répartition de la vitesse. La charge totale de frazil dans un cours d'eau gelé augmente considerablement le cisaillement sur les bords et la surface gelee, et affecte le grandeur et la répartition de la vitesse du courant d'eau. Des expériences sur place ont montré que la présence d'une couche de glace et une charge élevée du frazil peut facilement doubler la vitesse maximum d'une voie d'eau pour le meme débit. Les équations de la répartition de vitesse pour un courant d'eau sous une couche de frazil, y sont développées et ont été qualitativement vérifiées par des expériences sur place.

INTRODUCTION

The freezing of the surface of a river introduces a new boundary condition. The effective flow area of the river is reduced by a displacement boundary layer thickness at the top. This causes an increase in flow velocity and a higher velocity gradient in the bottom boundary layer.

As ice cover forms at the banks, the extra resistance causes a reduction in flow in the ice covered part and an increase in flow in the open channel part of the river. The increase in flow in the central portion of the river produces a higher velocity gradient at the bottom. Once a complete surface ice cover is formed, the resistance in the central portion is also increased. This in turn causes a reduction in flow in the central part and an increase in flow at the sides. Thus, as the surface of a river is being frozen, there is a gradual change in velocity distribution in the cross-section of the river. Until today, there has been little knowledge about the above problem, it is considered beneficial to collect some field data at this point. Some limited field data have been obtained in the Winter of 1969-70 and will be shown.

Once the surface freeze of a river is completed, the river becomes a closed conduit. If the river is not too narrow, the water flow under ice may be approximated by the flow of water between two parallel boundaries. This offers a possibility of theoretical approach to determine the distribution of velocity as shown in the following section.

VELOCITY DISTRIBUTION OF FLOW UNDER ICE AND THE EFFECT OF FRAZIL ICE LOADING

In considering flow under ice cover, the effect of frazil ice must be considered if it is present. The density of flowing water with a frazil ice concentration is non-homogeneous, which modifies the distribution of velocity. In the discussion here, a loading of frazil ice is assumed. The situation of frazil ice free flow is then the special case of water flow with zero frazil ice concentration.

The suspension of frazil ice is similar to the suspension of sandy sediment in the reversed way. There is, however, a significant difference between them, which is that the density of frazil ice is not much different from that of water and its concentration in water can be quite high during cold spells. This means the effect of frazil ice suspension on velocity distribution and bed shear can be very pronounced.

Viewing the problem qualitatively, when water flows turbulently under an ice cover with little frazil ice loading, the velocity distribution is fairly uniform except in the boundary layers. The uniformity of the velocity distribution is the result of turbulent mixing, which transports fluid particles of different velocities from one layer to the other. When the rate of discharge remains the same, but the loading of frazil ice is substantially increased, the above flow situation is changed. Because of buoyancy, frazil ice particles tend to float to the top. However, because of the turbulence generated by the flow, part of the frazil ice particles is transported back to the lower layers. Thus, when the state of equilibrium is established, there is a density gradient in the flow, with the density increasing from top to bottom. The existence of the density gradient discourages further turbulent transport of fluid particles because a large turbulent energy is required to overcome the density gradient. The reduction of turbulent mixing thus produces a non-uniform velocity distribution for the upper part of the flow. Accompanying the non-uniformity of velocity distribution is the reduction in discharge by the upper part. For the same rate of discharge to be carried by the stream, the maximum velocity has to increase to compensate the reduction in carrying capacity of the upper part of the stream. The increase in maximum velocity in turn increases the velocity gradient in the bottom boundary layer and hence bed shear. The change in velocity distribution due to frazil ice loading is shown in Figure 1.

To study the problem quantitatively, a xoy coordinate may be placed in the stream, with its x axis coincident with the maximum velocity as shown in Figure 1. For elementary fluid body dxdy, the forces acting on it are related by

$$-\frac{dp}{dx}\,dxdy + \rho\, gsdxdy + \frac{d\tau}{dy}\,dxdy = 0 \qquad (1)$$

Where $\frac{dp}{dx}$ is the pressure gradient and is a constant for a long ice-covered section, s is the bed slope, ρ is the density, g is the gravitational acceleration and τ is the shear stress. The integration of the above equation, with the boundary condition of $\tau = 0$, at $y = 0$, gives

$$\tau = \frac{dp}{dx}\,y - gs \int_o^y \rho\, dy \qquad (2)$$

In the upper part of the flow, the density ρ is a function of y, depending on the concentration of frazil ice. In the lower part of the flow, the density is approximately the same as that of the water ρ_w if the concentration of sediment is neglected. According to equation (2), at the upper boundary the shear stress is

$$\tau_{o_1} = \frac{dp}{dx}\,y_1 - gs \int_o^{y_1} \rho\, dy = \frac{dp}{dx}\,y_1 - gs\bar{\rho}\cdot y_1 \qquad (3)$$

where y_1 is the thickness of the upper part of the flow and $\bar{\rho}$ is the average density in the upper part. At the lower boundary, the shear stress is

$$\tau_{o_2} = \frac{dp}{dx}\,y_2 - gs\,\rho_w y_2 \qquad (4)$$

It is seen from equation (3) and (4) that if the point of maximum velocity and the average density can be determined, the shear stress at the top and at the bottom can be calculated.

The shear stress at a point is composed of two parts

$$\tau = \mu\frac{du}{dy} + \rho\,\overline{u'v'} \qquad (5)$$

where μ is the molecular viscosity and u' and v' are the velocity fluctuation in the x and y axis respectively. The bar above the second term means the time

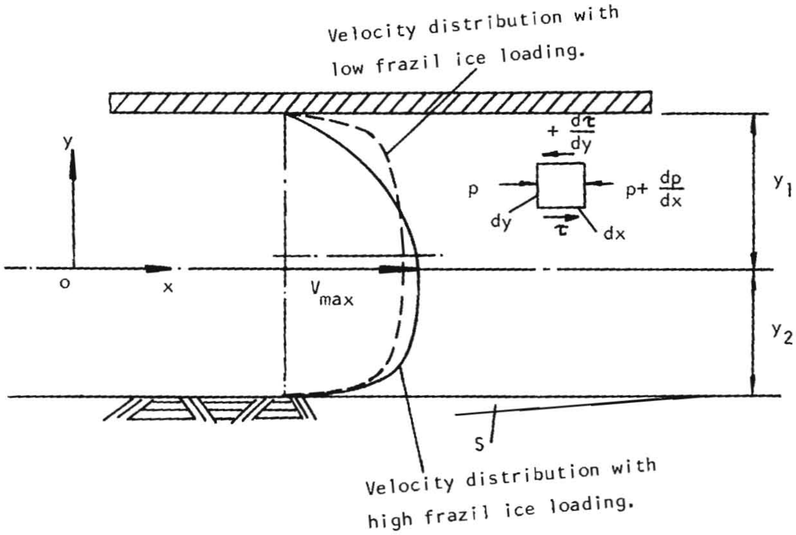


Fig. 1 - Change in Velocity Distribution due to Frazil Ice Loading.

average. The first term is the shear due to molecular friction and the second term is the shear due to turbulent momentum transport. When the concentration of frazil ice is high, as discussed earlier, the momentum transport of fluid particles is prohibited and the molecular shear is predominant. The substitution of equation (5) under this condition in equation (2) and the subsequent mathematical operations lead to

$$\frac{u}{u_{*1}} = \frac{N_{r_{*1}}}{2} (1 - y_{*1}^2) \tag{6}$$

where $u_{*1} = (-\tau_{o1}/\rho)^{\frac{1}{2}}$ is the friction velocity at the upper boundary, $N_{r_{*1}}$ is the Raynolds number defined as $N_{r_{*1}} = \frac{\rho y_1 u_{*1}}{\mu}$ and y_{*1} is y/y_1. The above equation is obtained assuming a constant molecular viscosity. This, in fact, may not be true since the viscosity of the mixture may vary with the concentration of frazil ice. As information relating the molecular viscosity and frazil ice concentration is not yet available, μ is considered to be a constant as an approximation here. Once additional information about viscosity and frazil ice concentration is known, appropriate velocity distribution equation may be obtained by the same approach.

When the concentration of frazil ice is low, the shear contributed by momentum transport will be the predominant part. The combination of equations (2) and (5) under this condition and the use of the mixing length theory lead to

$$\frac{du}{dy} = -\left(\frac{gs\rho_w - \frac{dp}{dx}}{\rho_w}\right)^{\frac{1}{2}} y^{\frac{1}{2}} L^{-1} \tag{7}$$

where L is the mixing length. If, similar to the approach in boundary layer studies, the mixing length is assumed to be proportional to the distance from the boundary, the above equation can be integrated and gives

$$\frac{u}{u_*} = 1.25 \left(\ln \frac{2}{1-y_{*o}^{\frac{1}{2}}} - \ln \frac{1+y_*^{\frac{1}{2}}}{1-y_*^{\frac{1}{2}}} \right) - 2.5 \,(1-y_*^{\frac{1}{2}}) \tag{8}$$

In the above equation the subscript "1" is dropped because the theoretical approach is equally valid for the lower part of the flow. Therefore, if it is borne in mind that for the lower part of the flow $y_* = y/y_2$, where y_2 is the thickness of the lower part of the flow, the above equation can be applied for the whole flow. y_{*o} in the above equation gives the point at which the velocity is zero. y_{*o} can be obtained from the velocity distribution equation in a turbulent boundary layer

$$\frac{u}{u_*} = 2.5 \ln (1-y_*) N_{r_*} + 5.5 \tag{9}$$

which gives

$$y_{*o} = 1 - \frac{0.111}{N_{r_*}} \tag{10}$$

It may now be concluded that for water flow under an ice cover, if the loading of frazil ice is high, the velocity distribution in the upper part of the flow is approximately given by equation (6). If the loading of frazil ice is low, the velocity distribution is given by equation (8). When loading of frazil ice is moderate, the distribution of velocity will be between the above two extremes. For the lower part of the flow, the velocity distribution is given by equation (8).

FIELD DATA COLLECTION

Field data were collected from Nottawasaga River in Southern Ontario in the Winter of 1969-70. This river has a meandering course and the experimental section, which is a straight section of about 600 feet, is the longest straight section in eight miles. The width of the river in the test section is about 80 feet. The bed material of the test section is silt and sand and is sensitive to erosion. A weir is built across the river about two miles upstream from the test site. An open section free from ice-cover always exists immediately downstream from the weir and provides a source of frazil ice under cold weather conditions.

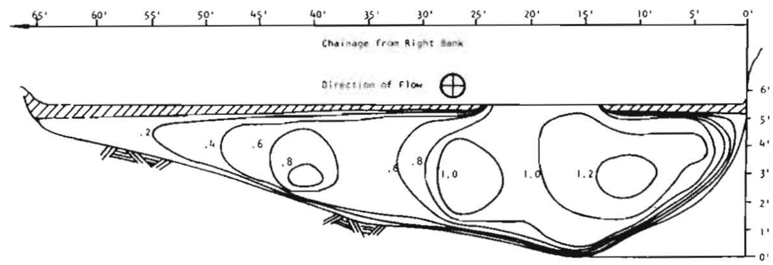
A metering crossing, where stream gauging was made, was selected about 150 feet downstream from the entrance bend to the test section. For obtaining velocity measurements and depths during the time of surface freezing and when the covering ice was thin, a cable bridge was built across the river at the metering crossing. Velocity was measured along verticals at 5 to 10 feet spacing across the river. At each vertical, velocity was measured from bottom to top at intervals from 2 to 6 inches. An Ott current meter with a 5 inch diameter propeller was used for measuring the flow velocity. Openings were drilled in the ice-cover for the Ott meter to pass through.

The cross-sectional contour and the depth of the river at the metering crossing and at two other crossings, one about 100 feet upstream and the other about 350 feet downstream from the metering crossing, were surveyed. The thickness and the elevation of the top of the ice cover were also obtained. The density of the flowing mixture could not be obtained because a workable instrument had not been developed.

EXPERIMENTAL FINDINGS.

From the velocity measurements, contour lines showing the velocity distribution under (1) a partial ice cover, (2) a complete ice cover and with low frazil ice loading, and (3) a complete ice cover and with high frazil ice loading, were obtained as shown in Figures 2a, 2b and 2c respectively. The rate of discharge of the river, which can be obtained from numerical integration of the velocity over the flow area, was found to be about the same in the three days in which the three sets of data were obtained.

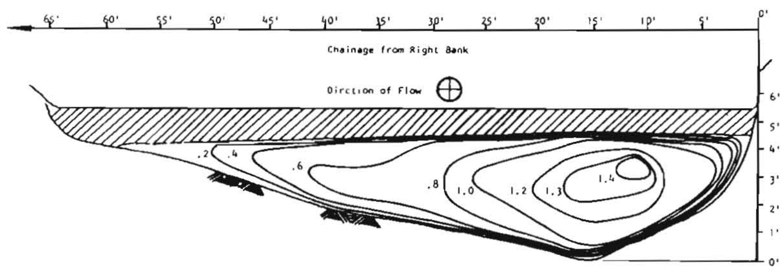
It is seen from Figure 2a that the gradual forming of an ice cover pushes the main stream towards the open section. It is noted that the point of maximum velocity was not in the open section but rather slightly in the ice-covered part at the right bank. This was likely caused by the remaining centrifugal effect of the flow in the upstream bend. There was a comparatively low velocity region



Date - Dec. 24, 1969. Location - Nottawasaga River near Alliston, Ontario.

Discharge - 170 ft^3/sec. Velocity Unit - ft./sec.

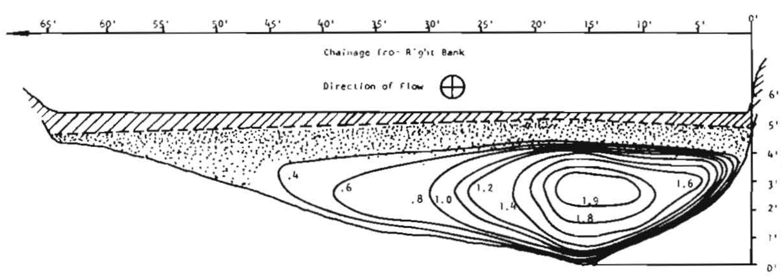
(a) - Velocity Distribution in the Metering Cross-Section under a Partial Ice Cover.



Date - Jan. 2, 1970. Location - Nottawasaga River near Alliston, Ontario.

Discharge - 161 ft^3/sec. Velocity Unit - ft./sec.

(b) - Velocity Distribution in the Metering Cross-Section under an Ice Cover and with a Low Loading of Frazil Ice.



Date - Dec. 27, 1969. Location - Nottawasaga River near Alliston, Ontario.

Discharge - 167 ft^3/sec. Velocity Unit - ft./sec.

(c)- Velocity Distribution in the Metering Cross-Section under an Ice Cover and with a High Loading of Frazil Ice.

Figure 2.

between two high velocity regions under the left ice cover. There is no satisfactory explanation for the existance of such a region except that it might be caused by instrumental malfunctions. It is unfortunate that the velocity distribution under the completely open condition was not obtained for comparing with Figure 2a.

From Figure 2b it is seen that as the surface freezing was completed and the ice-cover thickened, the flow area was reduced. The reduction in flow area caused the maximum velocity and the average velocity to increase. Coupled with the increase in maximum velocity is a general increase in velocity gradient.

It is seen from Figure 2c that when the loading of frazil ice was substantially increased, the maximum velocity and the velocity gradient of the flow increased significantly. although on December 27, 1969, the ice cover was thinner than on January 2, 1970, and thus the flow area was larger. The maximum velocity was still more than 30 percent higher. The comparison between Figures 2a and 2c shows that if an ice cover is present and the loading of frazil ice is high, the maximum velocity and the velocity gradient can easily be doubled. It was observed on December 27, 1969, that with a high concentration of frazil ice, the boundary between the covering ice and the water-frazil ice mixture was blurred. For this reason, the lower boundary of the ice cover in Figure 2c is shown by dashed line. Field observations showed there was no apparent overhanging ice-dam under the ice-cover to affect the velocity distribution.

The field data for obtaining the hydraulic gradient, and consequently the boundary shear stress, were found unreliable because of the inferior quality of the surveying equipment. This, plus the lack of density data, prevents a quantitative examination of the velocity distribution equations (6) and (8). However, a qualitative study can still be made by looking at the change of velocity distribution along a vertical under various conditions. The velocity distribution along the verticals 15 and 25 feet from the right bank for the three days is shown in Figure 3. The verticals were sufficiently far away from the two banks that flow there was approximately two-dimensional. It is seen from Figure 3 that when the loading of frazil ice was low, the presence of an ice cover introduced a thin boundary layer at the top. The velocity distribution between the top and bottom boundary layers was fairly uniform as would be expected for a turbulent flow. The velocity distributions along the 15-feet vertical showed that the presence of an ice cover increased the maximum and average velocity. When the loading of frazil ice was high, it is seen from Figure 3 that the velocity distribution in the upper part of the flow approached the parabolic form as predicted by equation (6). The parabolic trend of velocity distribution was more obvious along the 25-feet vertical. This might be due to the fact that the depth of the river at chainage 25 feet was less than that at chainage 15 feet. If the distribution of frazil ice varied little laterally,

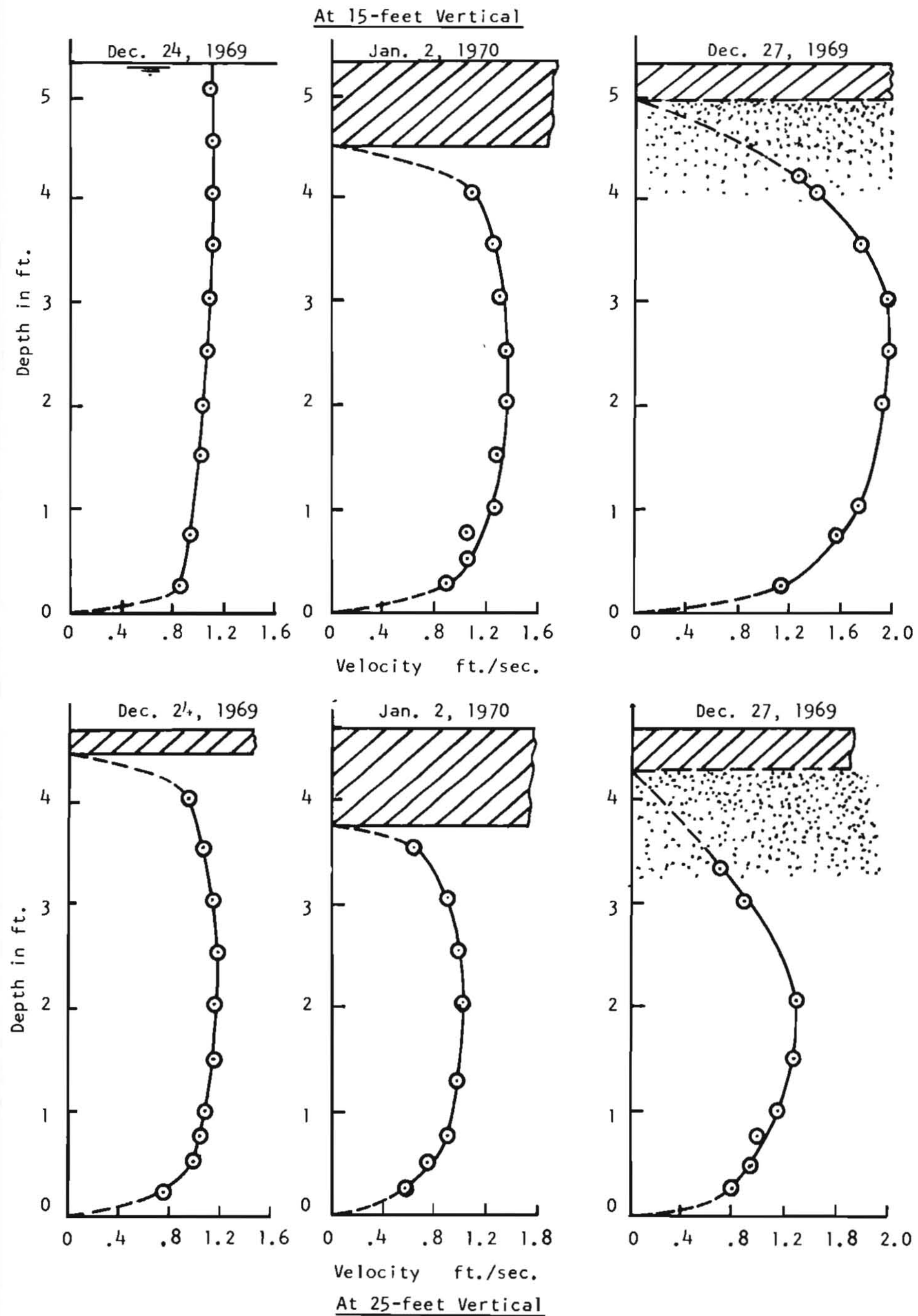


Fig. 3 - Velocity Distribution under Different Ice-Covered and Frazil Ice Loading Conditions.

3.2

the above means that proportionally there was a higher loading of frazil ice along the 25-feet vertical. This pushed the velocity distribution more to the parabolic extreme. It is noted that the lower part of the velocity distribution curve also tended to be parabolic when the concentration of frazil ice was high. This might be due to an increase in sediment concentration following the increase in bottom shear, seeing that the bed material was rather sensitive to shear. It may be presumably safe to conjecture that if the river bed was composed of coarser material, the parabolic trend of the lower part of the velocity distribution would be less marked.

CONCLUSIONS:

As a result of the present study, it may be concluded that

1. As the surface of a river is being frozen, the boundary condition of the flow changes. This causes a continuous change in flow pattern in the stream. More experimental data have to be obtained before a better understanding of the problem is reached and a theoretical analysis can be made.

2. The loading of frazil ice greatly affects the magnitude and distribution of velocity of flow under an ice cover. Instrumentation needs to be developed to measure the concentration of frazil ice in the flow.

3. Relationships governing the distribution of velocity for water flow under an ice cover with a high or a low frazil ice loading have been developed and were found at least qualitatively correct. Comprehensive experimental data, which include boundary shear, hydraulic gradient and velocity profile at regions close to the boundary, are required for a quantitative examination of the relationships and further theoretical exploration.

Acknowledgement.

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DISCUSSIONS.

Professor T. Carstens. The extrapolation by the author of the velocity distribution profile to the solid ice cover is hazardous when there is frazil underneath, such as on December 27. The consequence of the velocity profile assumed by the author is a reduced hydraulic gradient compared with that prevailing without frazil. But such a reduction, caused by frazil, has never been observed. On the contrary, frazil has invariably been observed to increase the energy losses. A more probable velocity profile, in my opinion, would show very low velocity through the frazil, followed by a strong gradient just above the depth range measured by the author.

Dr. S. Hanagud. This is a comment on Dr. Carstens' discussion. Dr. Carstens pointed out that the velocity profile obtained by the author for the frazil laden part of the flow does not agree with observed results. The discrepancy can be explained by considering the flow as flow through a porous material, instead of a viscous flow. If analysis is made, considering water and frazil are two different materials, improved velocity profile may be obtained.

Dr. E. Palosuo. The direction of current, not only the speed, is an important factor. In Finland I made some measurements of currents under the ice. The observations were made far from the coast in water which was about 30 meters deep. In the 10 to 20 meters layer, there was very often a constant current showing little variation in speed and direction. So I assumed that this current was a free one. In the layer between 9 meters and the undersurface of the ice, the speed of the current diminished slightly and the direction shifted to the left. I therefore suppose that here we can find an Ekman's spiral, inverted.

Author's reply: (1) To Dr. Carstens' comment. The velocity profile showed in the paper does not necessarily lead to a reduced hydraulic gradient. Although the velocity gradient, and consequently shear, is small in a parabolic velocity distribution, the displacement boundary layer is thick. This will cause the maximum velocity and the average velocity in the lower part of the flow to increase. Seeing that the head loss in the lower part of the flow increases with the square of the average velocity, the overall effect of the parabolic distribution of the velocity in the upper part of the flow could be an increase in hydraulic gradient for the total flow. I agree that the velocity profile in the part where field data are lacking is only hypothetical. Field data have to be obtained to show what actually happens. The non-parabolic behaviour of the velocity profile, if observed, may be accommodated by the non-uniformity of the molecular viscosity of the water-frazil mixture.

(2) To Dr. Hanagud's comment. There are no published field data showing the velocity in the region close to the ice-cover in a frazil-laden flow which is contradictory to the proposed equation. The frazil ice can not be considered as a porous material since it moves with the stream and the distance between any two frazil ice particles is not a constant.

(3) To Dr. Palosuo's comments. For open ocean currents, the Coriolis force is important which affects the direction of the currents. However, for streamflows, the boundary effect far outweighs the Coriolis effect so the direction of flow can be considered constant throughout the depth in a straight channel.

The ocean current observed by Dr. Palosuo seems to be a drift current. But it is puzzling to me that the shifting of direction should be in the opposite direction to the Ekman's spiral.

The diffusivity of a current of shifting direction in different layers is much higher than that of a current of uniform direction. Therefore, it is likely that the frazil-laden layer in an ocean current is much thicker than that in a stream flow of the same speed.

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REYKJAVIK

A STUDY ON THE FORMATION OF THE RIVER ICE IN A SNOW AREA

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Synopsis

River ice in heavy snow cover areas differs considerably from that in low temperature areas with a small precipitation. River ice in heavy snow cover areas is characterized by a layer of slush (like mixture of snow & water) sandwiched between layers of solid ice. It is pointed out in such case that the ice growth into the slush is a phenomenon independent of the thawing seen at the under surface of the river ice. In the present paper, the first mathematical approximation concerning the ice growth under a snow cover was led forth. It was shown that the second approximation was negligible compared to the first approximation under ordinary conditions. In addition, formation and thawing of river ice in a heavy snow cover area was discussed based on observed data obtained at the Ishikari river in Hokkaido.

Résume

Les glaces de rivière dans le pays neigeux est assez différentes de celles dans le pays peu neigeux. Les premières sont caractérisées par une couche de neige à demi fondue(une sorte de mélange de l'eau et de la neige) serrée entre deux couches de glace solide. Dans ce cas, nous pouvons montrer que la croissance de la glace dans la couche de neige à demi fondue est un phénomène indépendant du dégel vu sous la surface des glaces de rivière. Dans cet article nous avons montré la première approximation mathématique concernant la croissance de la glace sous une couverture de neige. Ensuite nous avons indiqué que sous une condition ordinaire, la deuxième approximation est négligeable en comparaison de la première. Enfin, nous avons fait une considération sur la formation et le dégel des glaces de rivière dans le pays neigeux en nous fondant sur les donnees obtenues à Ishikari : rivière du Hokkaido.

1. Introduction

The Ishikari river is 365 km in length, 200 m in width at the observation site. The observation post is located 25 km upstream from the river mouth. The discharge at the site is 535 m^3/sec on an average for 1 year, the discharge is 90 m^3/sec at its lowest in winter. This river generally completely freezes over around the middle of January and breaks up early in March.

Fig. 1 shows shematic cross sections of the river ice at the observation site in dates. Some terms used in the following disscusions are defined in fig. 1. The river ice in heavy snow cover areas consists of some strata of solid ice and slush(like mixture of snow & water). The water level in a hole drilled througth the river ice plays an important role in the formation of river ice. In other words, when the water level is high and is seen in the snow cover, the new layer of solid ice grows downward into the slush from the capillary potential level of water in snow.

At the same time a new twin layer of solid ice and slush becomes part of the river ice, and the river ice grows upward increasing the apparent thickness of it. However when the water level is lower than the upper surface of the river ice the apparent thickness does not show increase and the accumulated temperature of air is consumed by the growth of the uppermost solid layer of the river ice into the slush. Water appearing on the ice surface is due to floodings from exposed portions of the ice cover which were detectable both upstream and downstream from the observation site, and is due to somewhat thawing of the accumulated snow on the ice surface. Mechanism of the flooding from the exposed portions should be disscused by taking into consideration relations between displacement of the river ice in a vertical direction and the water level in the drilled hole through the river ice[1]. It is pointed out emprically and theoretically that the flooding arises under condition of a heavy precipitation. Fig. 2 shows the growth of solid ice into the slush under a snow cover as seen in fig. 1(The data presented in fig. 2 are independent of the data in fig. 1). On the other hand, the river ice thaws at the surface in contacting with the river flow.

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1) S.Kamada; Studies on the Ice Formation and Streamflow under Ice Condition in the River(in Japanese), No.38(Nov.1965), Civil Engineering Research Institute, HOKKAIDO Development Bureau.

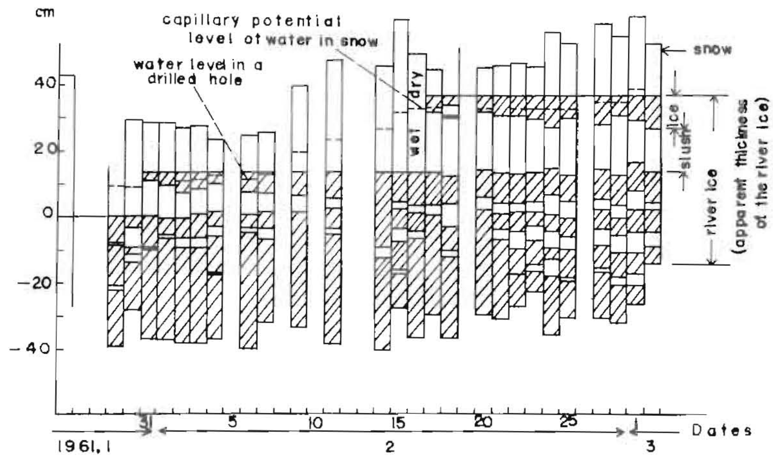
cm
capillary potential
level of water in snow
water level in a
drilled hole
dry
wet
snow
ice
slush
river ice
(apparent thickness of the river ice)
40
20
0
−20
−40
31
5
10
15
20
25
1
Dates
1961, 1
2
3

Fig. 1

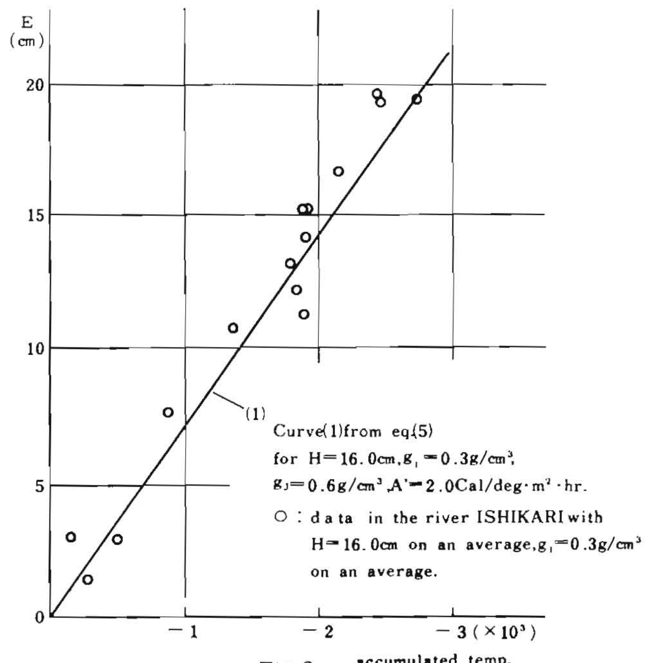
E
(cm)
20
15
10
5
0
−1
−2
−3 (×10³)
(1)
Curve(1)from eq.(5)
for H=16.0cm, g₁=0.3g/cm³,
g₂=0.6g/cm³, A'=2.0Cal/deg·m²·hr.
○ : data in the river ISHIKARI with
H=16.0cm on an average, g₁=0.3g/cm³
on an average.
accumulated temp.
(degree hour)

Fig.2

2. Ice growth under a cover

The authors led forth the following formula of ice growth under a snow cover in analogy with Ogura's formula[2] which is considered to be valid in the ice growth without a snow cover:

$$E \doteqdot E_1 + E_2. \tag{1}$$

$$E_1 = -sf + \left\{(sf)^2 - \frac{2k_2}{L' g_2}\int_0^t U\,dt\right\}^{\frac{1}{2}}. \tag{2}$$

$$\begin{aligned} E_2 = \frac{c_2 g_2}{k_2}\frac{1}{sf+E_1}\int_0^{E_1}\Big[\frac{Up}{r(sf+E_1)^2} + \frac{sf+E_1}{3r}\cdot U\cdot\Big(1-\frac{(sf-vH)^3}{(sf+E_1)^3}\Big) \\ + \frac{U'}{U}\Big\{\Big(\frac{k_2}{vk_1}-1\Big)\Big(\frac{vk_1}{A}+vH\Big)(sf+E_1)vH + p + \frac{1}{6}(vH+E_1)^2\cdot \\ (2vH-3sf-E_1)\Big\}\Big]dE_1. \end{aligned} \tag{3}$$

$$p = \frac{k_2}{A}vH\Big(\frac{vk_1}{A}+vH\Big) - \frac{k_2}{k_1}H\Big(\frac{vk_1}{A}+vH\Big)(sf-vH) - \frac{1}{3}(vH)^3 \Big(1-\frac{k_2}{vk_1}\Big)$$

$$f = \frac{k_2}{k_1}$$

$$v = \Big(\frac{k_2 c_1 g_1}{k_1 c_2 g_2}\Big)^{\frac{1}{2}}$$

$$r = \frac{L' g_2}{k_2}$$

$$s = \frac{k_1}{A} + H$$

$$L' = \Big(1 - \frac{g_T}{g_2}\Big)\cdot L$$

where E: thickness of ice
- k: coefficient of heat conduction
- H: thickness of the snow cover
- A: heat transfer coefficient=$k(du/dx)/(u-U)$ at the snow surface
- u: temperature in a layer of snow or ice
- x: coordinates in a vertical direction
- g: density
- g_T: density of snow in slush
- U: air temperature at a point sufficiently far from the snow surface
- L: latent heat of ice
- suffix 1: a suffix for snow
- suffix 2: a suffix for ice

A mathematical approximation with $(c_1 g_1 k_2/c_2 g_2 k_1)^{2}$ instead of k_2/k_1 as f in (2) was previously set forth by S.Kamada, one of the pre-

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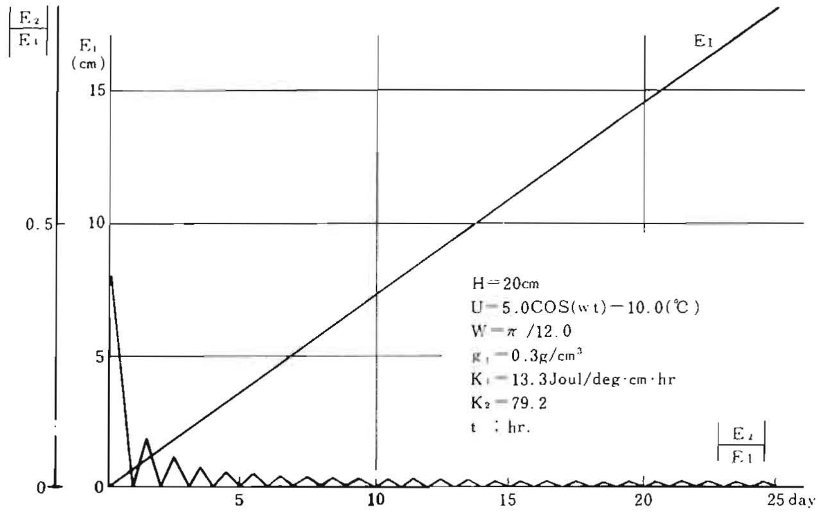
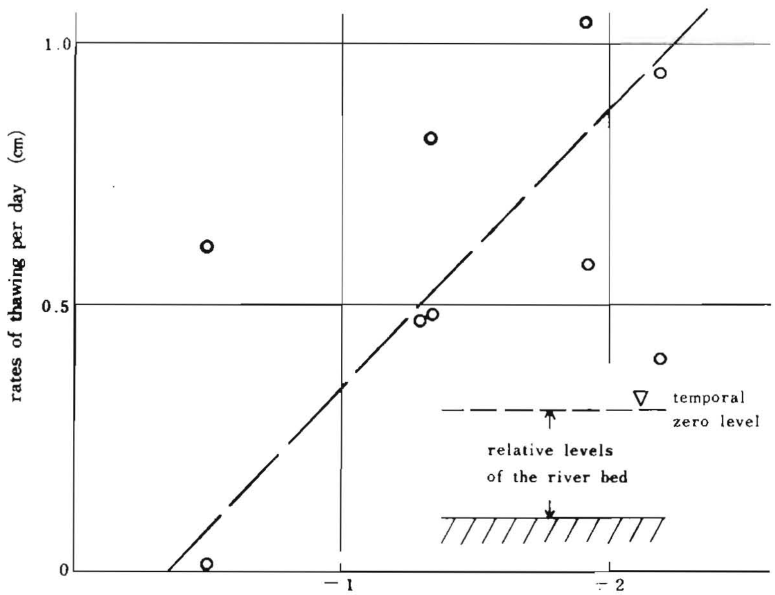
2) Y.Ogura; A Supplementary Note on the Problem of Ice Formation, p231, vol.xxx(1952), J.Met.Soc.Japan.



E2/E1
E1 (cm)
15
10
5
0
0.5
0
E1
H=20cm
U=5.0COS(wt)−10.0(℃)
W=π/12.0
g1=0.3g/cm³
K1=13.3Joul/deg·cm·hr
K2=79.2
t : hr.
E2/E1
5
10
15
20
25 day

Fig. 3



rates of thawing per day (cm)
1.0
0.5
0
temporal zero level
relative levels of the river bed
−1
−2
relative levels of the river bed (m)

Fig. 4

sent authors, in 1965. However, this formula shows an accuracy only for $c_1 g_1 / c_2 g_2 \doteqdot k_2/k_1$. The temperature at the upper surface of the ice growing under a snow cover was obtained during the process of leading forth (2),(3) as follows:

$$u_1 = U\left(1 - \frac{K_1/A + H}{K_1/A + H + K_1 E/K_2}\right) \qquad (4)$$

where u_1 is temperature at the upper surface of the ice. Examples of the calculation are indicated in table 1.

E \ H	5	10	20	30
5	0.14	0.08	0.04	0.03
10	0.24	0.14	0.08	0.05
15	0.32	0.20	0.11	0.08
20	0.39	0.25	0.14	0.10
25	0.44	0.29	0.17	0.12
30	0.49	0.33	0.20	0.14

A=40 Joul/deg.cm^2.hr

E \ H	5	10	20	30
5	0.14	0.08	0.04	0.03
10	0.25	0.14	0.08	0.05
15	0.34	0.20	0.11	0.08
20	0.40	0.25	0.14	0.10
25	0.46	0.30	0.17	0.12
30	0.50	0.34	0.20	0.14

$k_1/A \ll H$

H,E in cm, g_1=0.3 gr/cm^3 , k_1=13.3 Joul/deg.cm.hr, k_2=79.2 Joul/deg.cm.hr

Table 1 u_1/U

u_1 is a variable depending on the thickness of the ice even when H =const., U=const.. Thus it may be said that the formulas of Stefan, Neumann concerning ice formation may be restricted to a considerable extent under snow cover conditions. While E_1 presented above is not affected by U'(=dU/dt), E_2 gives rise to a discrepancy coming from U' and the assumptions of linear distribution of temperature in the ice and the snow. Fig. 3 indicates that E_2 is negligible as compared to E_1 in ordinary conditions of U such as sine curves. By taking $k_1/(k_1/A + H)$ as the apparent heat transfer coefficient A', (2) is reduced to

$$E \doteqdot -\frac{K_2}{A'} + \left\{\left(\frac{K_2}{A'}\right)^2 - \frac{2K_2}{L'\delta_2}\int_0^t U\,dt\right\}^{\frac{1}{2}} \qquad (4')$$

which coincides with (2) for H=0(sf=k_2/A) as a matter of form. The wind velocity on the Ishikari river in winter is 5~10 m/sec on an average and the assumption $k_1/A \ll H$ may be allowed in this case. Thus, the apparent heat transfer coefficient A' is given by k_1/H and E takes the form of

$$E \doteqdot -\frac{K_2}{K_1}H + \left\{\left(\frac{K_2}{K_1}H\right)^2 - \frac{2K_2}{L'\delta_2}\int_0^t U\,dt\right\}^{\frac{1}{2}} \qquad (5)$$

Consequently, the thickness of the ice layer under a snow cover can be estimated by (5). If (5) is applied to the uppermost solid ice at the Ishikari river in fig. 2 and it is assumed that the snow cover is uniform in a vertical direction and H=const.(16.0 cm), A' and g_J take the following values by taking the mean of observed data g_l =0.3 gr/cm^3 into consideration:

$$A' = 2.0 (Cal/m^2.deg.hr)$$
$$g_J = 0.6 (gr/cm^3).$$

3. Thawing at the under surface of the river ice

Fig. 4 shows rates of thawing at the under surface of the river ice per day caused by the river flow. The rate of thawing seems to be protortional to the distance between the river bed and the lowermost level of the river ice, although relative levels of the river bed are used in fig. 4. The factor contributing largly thawing at the under surface of the river ice is the turbulent heat transfer from the river bed and warming of water by the absorption of solar radiation in the exposed portion of the ice cover. In any event, it may be surmized that the rate of thawing per day is proportional to the distance between the river bed and the lowermost level of the river ice. If the turbulent heat transfer is taken as the main factor of thawing, the temperature of the river bed may be estimated as $10^{-2} \sim 10^{-3}$ (°C) from the rate of actual thawing in the Ishikari river[3]. The thawing at the under surface progresses to some extent and the river ice breaks up and a channel opens in the center of the river.

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ICE SYMPOSIUM 1970
REYKJAVIK

MEASUREMENTS OF ICE ROUGHNESS AND THE EFFECT OF ICE COVER ON WATER LEVELS IN THREE NORWEGIAN RIVERS

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SYNOPSIS

Some results from a field study of ice damming and ice roughness in three Norwegian rivers are referred and discussed. Average ice roughness values of n=0.013-0.020 (Manning's n) have been found, which is much less than the bed roughness, but still significant for the flow. The difficulties involved in the use of average slope as an aid to estimate winter discharges are discussed.

RÉSUMÉ

Quelques resultats d'une etude de rugosité de glace et d'influence de glace aux hauteurs d'eau dans trois rivières norvegiennes sont exposés et discutés. Valeurs moyennes de la rugosité de glace environt n=0.013-0.020 (n de Manning) sont trouvés. Ses valeurs sont beaucoup plus moins que le rugosité de lit, mais encore significants pour l'écoulement. Les difficultés rencontrée utilisant la pente moyenne pour estimer le débit d'hiver sont traitées.

1. INTRODUCTION

The influence of ice on the water stage in rivers is a complex practical problem for hydrologists and hydraulic engineers. The most important causes of ice damming are

a. actual damming, from ice dams
b. reduced cross section, by bottom ice, bank ice and ice cover
c. increased bed roughness, due to bottom ice etc.
d. added roughness, due to ice cover formation
e. modification of turbulent viscosity due to ice particles

a), b) and c) are local problems that have to be studied separately for each river, while d) and e) to some extent can be treated theoretically. The hydraulic aspects were studied by Devik (1,2) and also applied with success to data from Göta River (3) assuming equal roughness of ice and bed. More recent findings have shown that the situation varies much from river to river, even situations with much greater ice roughness than bed roughness have been reported (4). A bibliography by Michel and Triquet contains several references to the subject (5).

One way to treat the problem of ice damming is to include all the irregularities into what is commonly referred to as the winter coefficient, i.e. the ratio between actual discharge and the equivalent ice free discharge for the actual stage. This method was probably first proposed by Kolupaila (6).

A program to collect more field information about the ice roughness and the influence of ice on the stage/discharge ratings has been run last winter by the Hydrology Division of the Norwegian Water Resources and Electricity Board.

2. THE FIELD CONDITIONS

Discharge measurements with Ott propellers have been made in three rivers where ice damming occurs regularly. Emphasis has been put on correct registration of the velocities close to the ice and bed boundaries of flow. The measurements have been related to existing water stage gauges with known rating curves for summer conditions. Additional gauges have been installed to be able to control the average slope of the river.

The locations and surroundings of two measuring stations are shown on fig. 1, and the longitudinal sections of part of the rivers on fig. 2. The third station lies in a nearly flat reach of Numedalslaagen.

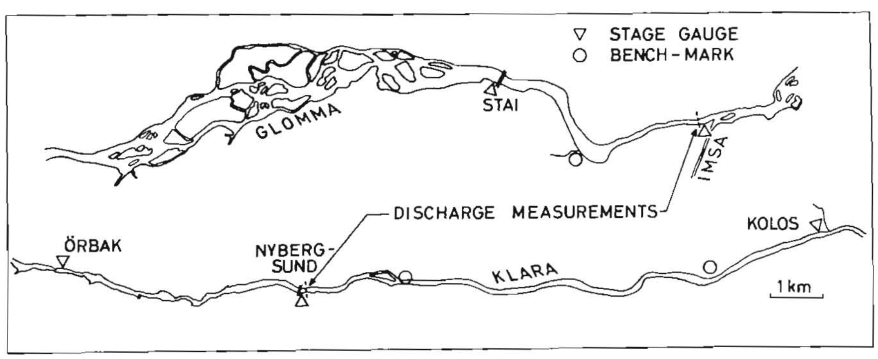

Fig. 1. Location of field stations in Glomma and Klara rivers

Klara has moderate slopes for more than 10 km both upstream and downstream of the station. A stable ice cover usually forms early in the winter. Pack ice or frazil ice accumulations normally do not occur as far down as the station, hence ice damming at the gauges is due to the plain ice cover only.

The permanent gauge at Stai in Glomma is usually disturbed by large accumulations of pack ice from ice flows or frazil ice formed in the rapids upstream. The ice cover is usually undisturbed at the measuring station near Imsa outlet.

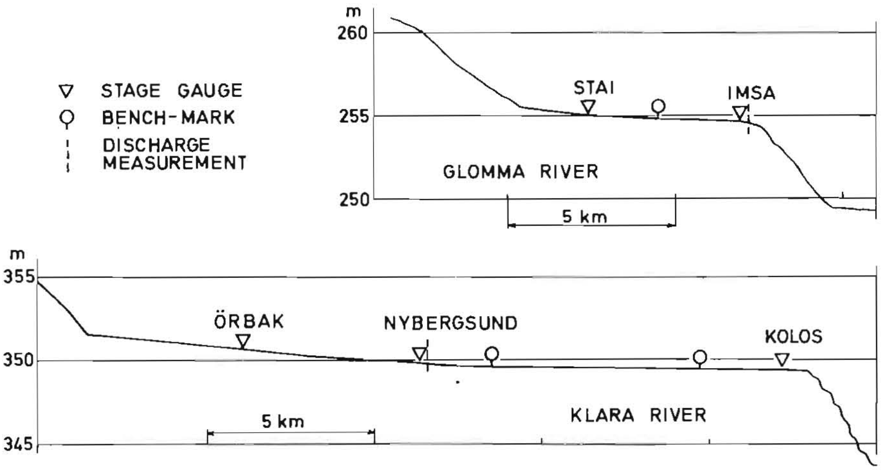

Fig. 2. Longitudinal sections of parts of Glomma and Klara rivers

3. THE ROUGHNESS OF THE ICE COVER

The velocity measurements have been used to calculate roughness parameters for the river bed and the ice cover. By substituting measured values from two points near the boundary in question into the Kårmån-Prandtl formula

$$v = 5.75 \quad v_* \ \lg \frac{33\ y}{k} \tag{1}$$

a value of k is obtained that can be used in "the rough channel law"

$$\frac{1}{\sqrt{f}} = 2 \ \lg \frac{14.8\ R}{k} \tag{2}$$

or the equivalent formula with Manning's constant

$$\frac{1}{n} = 2 \sqrt{8\ g} \ \lg \left(\frac{14.8\ R}{k}\right) R^{1/6} \tag{3}$$

Here v=velocity, v_* =friction velocity, y=distance from boundary in question, R=hydraulic radius, here assumed $R=y_m$, where y_m=distance to velocity maximum, g=acceleration of gravity, n=constant in Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{4}$$

k=roughness parameter, f=constant in Darcy-Weissbach's formula,

$$V = \left(\frac{8}{f} gRS\right)^{1/2} \tag{5}$$

V=average velocity and S=slope. (3) and (4) are in metric units.

A different approach is to calculate v_* from (1) and then use the average velocity V in each vertical to calculate f from the expression

$$f = 8(v_*/V)^2 \tag{6}$$

It is not likely that (2) and (6) will give identical f-values, due to the approximation $R=y_m$, and the use of local V-values.

Values of n according to (3) and of v_*/V have been calculated by electronic computer. The single values have been valuated to ommit obviously erroreous results. In particular values of $n < 0{,}01$ have been adjusted to $n = 0{,}01$. Simple means have then been calculated by hand both for each complete measurement (six verticals) and for the whole season (12-16 measurements). Also median and quartile values have been calculated. The average v_*/V-values have been used to calculate f from (6).

The use of simple means overemphasizes the importance of large values of Manning's n-coeffesient, but gives an idea of the importance of ice roughness against bed roughness. The results for the three locations are listed in table 1.

Table 1. Roughness data from field studies.

River Location	Klara Nybergsund	Glomma Imsa	Numedalslaagen Kongsberg
Period of measurements	18.11-29.4	28.11-23.4	18.12-11.4
Number of days with measurements	12	16	14
Ice: Number of n-data	61	74	58
Mean value of n	.020	.013	.014
1./2./3. quartile of n	.011/.016/.024	.010/.010/.013	.010/.010/.015
Mean value of v_*/V	.073	.035	.039
1./2./3. quartile of v_*/V	.04/.05/.15	.01/.02/.04	.02/.03/.06
f from mean v_*/V	.043	.010	.012
Bed: Number of n-data	66	91	65
Mean value of n	.040	.044	.037
1./2./3. quartile of n	.027/.038/.053	.033/.040/.055	.021/.029/.040
Mean value of v_*/V	.138	.124	.087
1./2./3. quartile of v_*/V	.08/.12/.18	.03/.12/.17	.04/.07/.11
f from mean v_*/V	.151	.124	.061
Ratios of ice and bed means: (n)	.50	.29	.38
(v_*/V)	.53	.28	.45
(f)	.28	.08	.20

The results show that assumptions of equal ice and bed roughness or negligible ice roughness are both insufficient for practical purposes.

The mean values for bed roughness vary little in the three cases, while there is a distinct difference between the ice roughness at Nybergsund and the two other locations. A possible explanation may be that the mean velocities were higher at Nybergsund (0.20-0.34 m/s) than at Imsa (0.15-0.22 m/s) or Kongsberg (0.11-0.21 m/s). The material does not allow further study of this aspect, however.

Though the n- and f-values have been obtained by different procedures, there are good agreement between the findings, as shown by the three lower lines of table 1. According to (2) and (3) as well as (6) f is proportional to n^2 and $(v_*/V)^2$. The ice/bed ratios for n and v_*/V agree well, at least for two of the locations.

4. THE ICE DAMMING

The results of stage and discharge measurements in Klara and Glomma are shown on fig. 3 and 4. The ice damming at the stage gauges has been calculated as the difference between the measured stages and the stages compatible with the measured discharges according to the rating curves. The diagrams also give the ratio k=Q/Q' between actual discharges and discharges according to the rating curves. Fig. 3 gives the average ice depth at Nybergsund, where the discharge measurements were made only about 100 m from the stage gauge.

Except for the first part of the ice season, both the ice damming and the values of k seem to vary smoothly. This adds support to the use of Kolupaila's method for estimation of winter discharges (6) based on the stage ratings and a few determinations of actual discharge during the winter season.

The ice damming at Nybergsund was roughly half of the ice depth during most of the season, which means that the average velocity under the ice has been greater than it would have been for the same discharge without ice. The added shear force from the ice cover must therefore be balanced by an increase of slope. This agrees with the fact that the ice damming has been higher at Örbak than at Nybergsund.

Ice depths were not measured regularly at Örbak, but check measurements indicated little difference from Nybergsund, so apparently the difference between ice depth and damming decreases upstream.

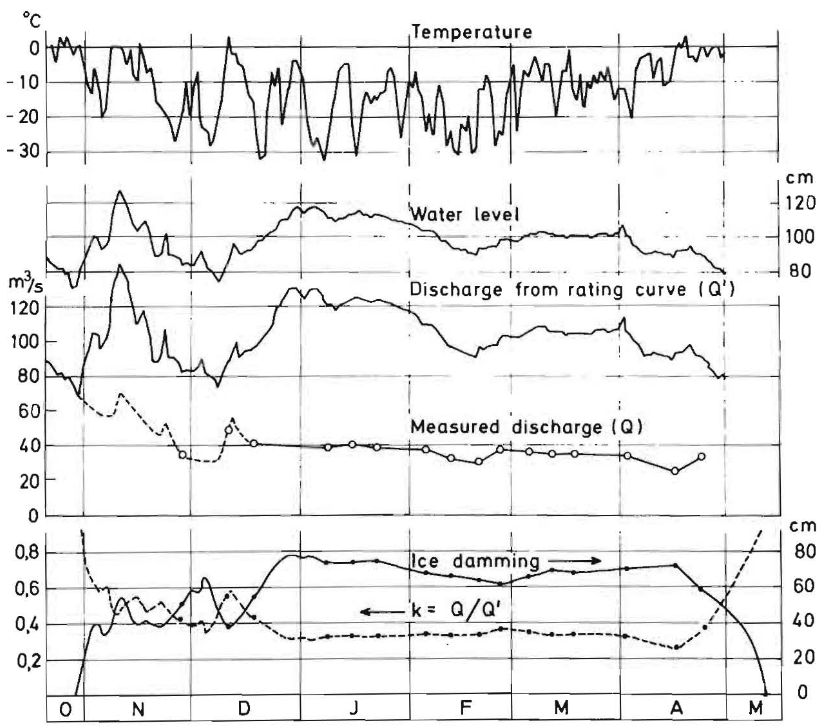

Fig. 3. Data from Glomma at Stai gauge winter 1969/70

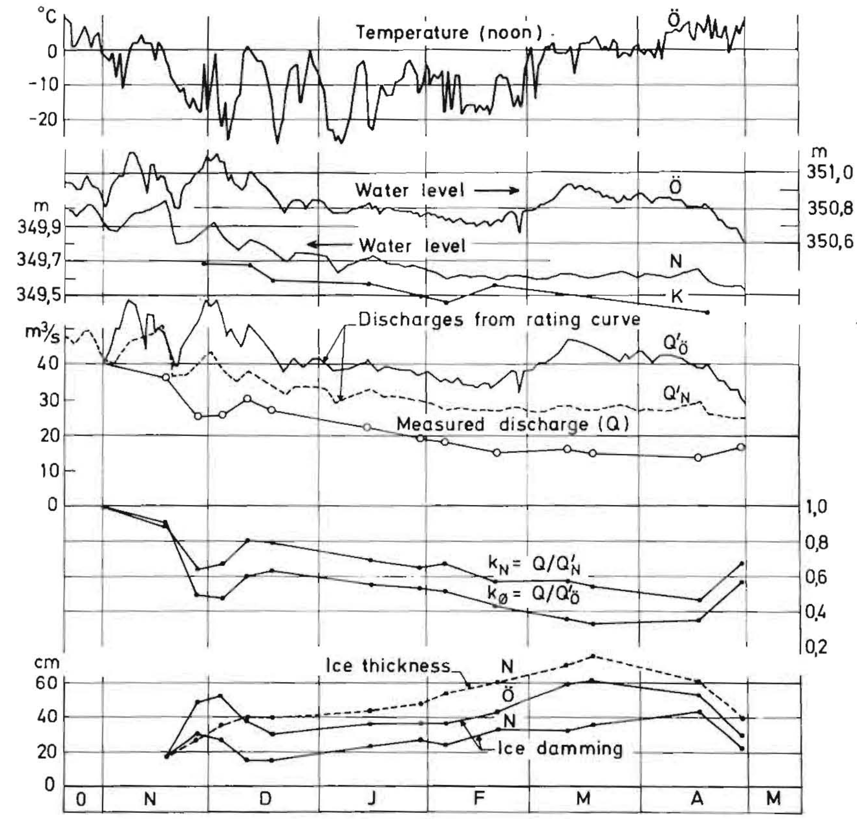

Fig. 4. Data from Klara winter 1969/70
Ö=Örbak, N=Nybergsund, K=Kolos

The ice damming in Klara increased during most of the winter, probably as a direct consequence of the increasing ice thickness. In Glomma at Stai the ice damming reached its highest value around 1. January, caused by ice flows and jamming, then decreased slowly until about 1. March, probably as a result of wearing or melting away of the most distinct irregularities and obstructions under the pack ice. After 1. March the ice damming at Stai varied in a similar manner as in Klara.

The ice damming (and discharge) varied significantly with temperature only in the first part of the winter.

5. DISCHARGE, SLOPE AND ROUGHNESS

Table 2 shows the results of slope measurements in the Klara and Glomma rivers.

Values of Manning's n have been calculated based on the obviously wrong assumption that the average slopes on reaches just upstream or dowmstream are representative for the measured section.

Some data for ice free conditions have been added at the bottom of table 2.

Table 2. Slopes and average roughness.

Klara river							Glomma river				
			Örbak-Nybergsd.		Nybergsd.-Kolos		Stai - Imsa				
Date	Q m^3/s	Vm m/s	S m/km	n	S m/km	n	Date	Q m^3/s	Vm m/s	S m/km	n
1970							1970				
18.11	36.7	.34	.206	.051	.0274	.019	28.11	34.6	.15	.058	.063
27.11	25.7	.26	.256	.072	.0142	.013	11.12	48.3	.22	.072	.048
04.12	26.0	.27	.271	.071	.0160	.015	18.12	41.4	.20	.104	.063
11.12	30.8	.33	.262	.056	.0160	.017	08.01	39.0	.19	.138	.074
18.12	27.4	.30	.247	.059	.0160	.019	15.01	40.6	.20	.142	.072
14.01	22.8	.26	.247	.068	.0140	.018	22.01	39.6	.20	.142	.072
29.01	19.5	.23	.260	.076	.0038	.011	05.02	37.6	.19	.124	.070
05.02	18.6	.23	.249	.073			12.02	32.3	.17	.122	.074
19.02	15.5	.20	.249	.085			19.02	30.5	.17	.120	.073
11.03	16.7	.22	.289	.080			26.02	37.3	.20	.106	.061
18.03	15.2	.21	.287	.081			05.03	36.4	.19	.104	.063
17.04	14.2	.18	.254	.093			12.03	34.9	.19	.112	.057
20.04					.0189		18.03	35.0	.20	.122	.063
29.04	17.5	.21	.240	.078			02.04	34.0	.19	.122	.065
14.03	67.0	.48	.217	.051	.0236	.017	09.04			.110	
							16.04	24.1	.14	.124	.089
1910							23.04	33.8	.18	.118	.072
28.07	51.8	.43			.0151	.016	11.08	93.0	.35	.046	.038
04.08	63.7	.48	.229	.056							

The only reliable measurement of the actual slope at a measuring station was made at Nybergsund 13.1. 1970 over a 200 m long reach and gave 0.125 m/km. Combined with the discharge etc. measured next day, this gives n=0.049.

Attempts to estimate n-values for calculation of discharge from measured slope and stage data will be very difficult, due to the large seasonal variation according to the table.

6. FINAL COMMENTS AND CONCLUSIONS

The study has provided some valuable information about ice roughness and winter discharges to be added to the still insufficient knowledge available for the estimation of winter discharges from stage readings. A more thorough report is being prepared where also earlier, more occasional measurements on the same locations will be included.

So far, the following conclusions can be drawn:

The roughness of the ice cover on calm reaches varies from place to place. The study has given roughness values varying from that of smooth river bed to glossy surfaces.

No simple relationships seem to exist between bed and ice roughness or between ice damming and ice depth.

The ice damming at a stage gauge and the related ratio k=Q/Q' (the winter coefficient) vary smoothly during periods with stable ice cover. The trends of variation differ with the local conditions.

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DISCUSSION

B. MICHEL

There has been interesting discussion on the fact that ice roughness some times increase as winter goes by and othertimes decrease with time. May I submit that this may be only an apparent contradiction which depends on the type of ice cover you are considering. With a solid ice sheet, there is thermal erosion under the ice approaching breakup, dunes are formed and roughness increases. In most ice covers in rivers, floes and slush get incorporated into the cover at formation time. With time they settle, a better channel is formed in the unconsolidated part of the cover, and the roughness decreases as winter goes by.

JOHN F. KENNEDY

Mr. Tesaker has presented some very interesting data on the roughness of the underside of ice covers. Data of the type he has reported are making it increasingly clear that the roughness of the ice-water interface is by no means constant, but rather evolves - sometimes decreasing (when the cover is composed of fragmented ice), sometimes increasing (when there is a solid ice cover) during the ice season. It took some years to realize that the roughness of sand beds of alluvial rivers is extremely variable as the ripple and dune configuration give way to a flat bed. In some instances, however, the roughnesses of river beds and ice covers are opposing. River beds of sand obtain their minimum roughness during periods of high discharge and large velocity, thereby reducing the stage rise accompanying floods. Continuous ice covers, on the other hand, appear to become rougher during the periods of melting from below, which is generally during the spring when the annual river flood may be beginning. Hence once again it seems that natural processes are such that they may favor man's works, but just as often they oppose them.

C. R. NEILL

Were any of the ice-cover roughness measurements made late in the season, when the roughness of the under-side of the cover might have been expected to increase due to erosion?

The data have been studied a little more in detail since the preparation of the report.

The variation of the ice roughness with time is shown on fig. 5 (Manning's n). If the period of initial ice formation is excluded, a gradual rise in roughness from n=0.012 to n=0.025 has been found for Nybergsund, falling again at the end of the season. No particular trend has been found for the two other locations, may be due to less average velocity and consequently less turbulence than at Nybergsund.

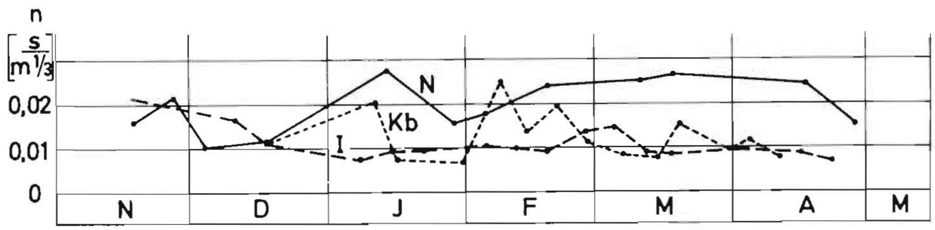

Fig. 5. Variation of Manning's n of ice cover during winter 1969/70
I=Imsa, Kb=Kongsberg, N=Nybergsund

To the bed roughness results can be added that recent summer measurements at Nybergsund and Imsa both gave bed roughness n=0.042, in good agreement with the winter values. Similar check measurements have not been made at Kongsberg.

By comparing the curves for ice thickness and ice damming on fig. 4 in the paper, itis found that the velocity under the ice actually has increased compared with similar summer discharges, despite the added ice friction. The reason is a comparatively greater increase in slope, documented by the Ö and N curves for ice damming on fig. 4, and also shown in table 2. A similar increase in velocity has also been found at Imsa.

ICE SYMPOSIUM 1970
REYKJAVIK

ACCUMULATION ET COURBE DE REMOUS SOUS LE CHAMP DE GLACE SUR LA RIVIÈRE VLTAVA

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Tchécoslovaquie

During the construction of the Vltava Cascade in 1935 till 1962, dangerous ice barriers occurred mainly at the end of the reservoir backwater. In this paper the passage of the ice jam through Vrané Reservoir in the course of the whole winter season in relation to the free surface area in the St. John Rapids is described. By measurements it was found that the free discharge area in the cross-section did not change to any great extent and that hence neither the velocity coefficient in the Chézy equation increased as stated by some authors, perhaps under other conditions.

Pendant la construction de la cascade des grands barrages sur la rivière Vltava durant la période 1935 - 1962 il y avait des barrages de glace dangereux, spéciallement au bout du remous des réservoirs. Dans le rapport on décrit l´avancement du sorbet de glace dans le bief de Vrané pendant tout l´hiver en relation avec la surface libre de l´eau dans"les courants de Saint-Jean". Par les mesurages on a constaté que la section de passage dans le profil transversal ne changeait pas d´une façon importante et que même le coefficient de vitesse de Chézy ne s´agrandissait pas au cours de l´hiver comme on l´indique parfois dans la littérature, quand il s´agit sans doute d´autres conditions.

1. Description de la voie fluviale et des ouvrages d'art.

Les barrages des glaces se produisent au bout du remous du lac de barrage, en amont duquel se trouve le courant libre avec des secteurs qui en conséquence des grandes vitesses d'écoulement ne prennent que très tard ou même pas du tout durant tout l'niver.

Ce déroulement des phénomènes de glace s'est manifesté d'une manière typique au cours de l'édification de la cascade sur la rivière Vltava. La situation la plus dangereuse c'était celle après l'édification du premier barrage à Vrané, km 173,4 /15 km environ en amont de Prague/ et elle s'est traduite pendant les hivers extraordinairement rigoureux, quand le barrage de Štěchovice au km 160,4 était en construction /Fig. 1/.

La glace flottante se formait dans le secteur qui se trouve en amont du chantier Štěchovice, secteur caractérisé par une plus grande pente du lit de la rivière /i > 0,2 ‰/ et où se trouvent les blocs de pierre dépassant le niveau de la nappe d'eau et elle se tassait d'une part en amont du chantier mais principalement au bout du remous causé par le barrage Vrané entre le km 162 et le km 167. On peut caractériser la rivière Vltava, en amont du km 160,4, par une grande vitesse d'écoulement /au-dessus de 0,5 m/sec./ pendant une gelée forte et d'une longue durée; c'est alors que prend sa naissance un régime difficile d'hiver, le sorbet de glace se produit pendant longtemps et d'une manière intense, ils se forment des barrages de glace, la glace flottante pénètre dans les installations de prise et elle se tasse dans la partie terminale des lacs de barrage.

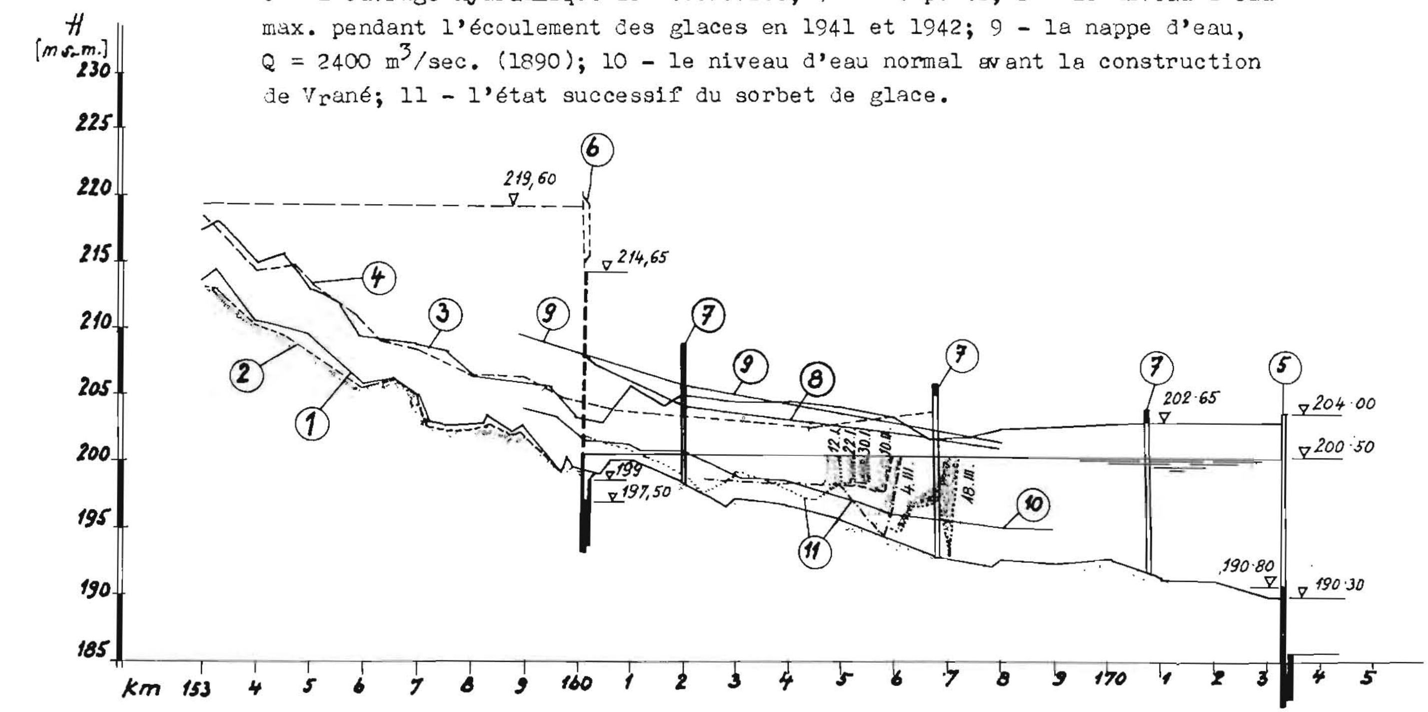
2. Description des phénomènes de glace.

a/ La période d'hiver 1928-29 a été la plus rude dans ce siècle. Le secteur poursuivi de la rivière était à cette époque sans construction haussant un remous. L'épaisseur de la glace a atteint à Štěchovice 48 cm.

b/ La période d'hiver 1940-41 a été, elle aussi, extraordinairement froide. Sur la figure 2 se trouvent les températures de jour max., min. et moyennes de l'air du poste Tábor, les niveaux d'eau H de Štěchovice et les débits Q constatés dans la rivière. Les niveaux d'eau à Štěchovice étaient influencés par le remous du barrage Vrané et par le bouchon de glace et de sorbet dans le trajet final du remous. C'est pourquoi au mois de janvier les niveaux d'eau ont été relativement hauts c.à.d. + 190 cm jusqu'à + 285 cm, les débits ayant été moyens /Q = 80 m^3/sec./ et au passage de glace

Fig. 1. Le profil longitudinal de la Vltava entre les km 153 - 174.

1 - le fond dans l'axe de la rivière; 2 - les points les plus bas du fond; 3 - la rive gauche; 4 - la rive droite; 5 - l'ouvrage hydraulique de Vrané; 6 - l'ouvrage hydraulique de Štěchovice; 7 - les ponts; 8 - le niveau d'eau max. pendant l'écoulement des glaces en 1941 et 1942; 9 - la nappe d'eau, Q = 2400 m^3/sec. (1890); 10 - le niveau d'eau normal avant la construction de Vrané; 11 - l'état successif du sorbet de glace.



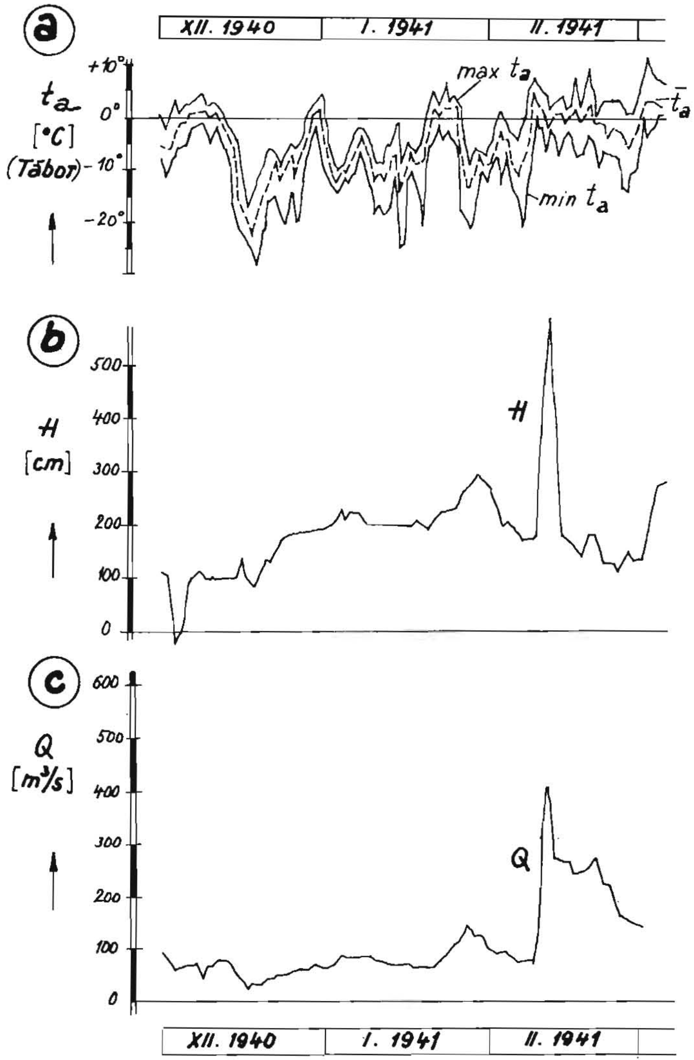


Fig. 2. Le cours de l'hiver 1940-1941 à Štěchovice:

a - les températures de l'air t_a /min.; max.; moyenne de jour $\overline{t_a}$/, °C;

b - les niveaux d'eau H, m;

c - les débits Q, m³/sec.

le niveau d'eau a atteint le 12 février une valeur catastrophique de + 590 cm.

Malgré que l'hiver en 1940-41 était plus modéré que celui en 1928-29, l'écoulement des glaces a été beaucoup moins favorable et cela précisément en conséquence du remous de l'eau causé par le barrage Vrané.

c/ Dans la période d'hiver 1941-1942 j'ai fait des observations des phénomènes de glace dans le secteur km 151-170, c.à.d. sur le trajet où la glace flottante se formait et se déposait. Par le travail du bateau brise-glace, par l'amorcement de la glace et par la manipulation avec la nappe d'eau du barrage Vrané on est arrivé à un écoulement des glaces non nuisible, malgré que cet hiver-ci a été plus froid que l'hiver précedent [table 1].

Table 1.

Caractéristiques des hivers les plus rudes

Caractéristique	1928-29	1940-41	1941-42
Le total des températures de jour moyennes négatives de l'air, $\Sigma \overline{t_a}$ [°C]	- 720	- 446	- 612
La température $\overline{t_a}$ min. [°C]	- 33,8	- 23	- 25,5
Le niveau d'eau max. au passage du glace (± 0 de l'échelle d'eau = 199,55 m)	+ 510	+ 590	+ 335

En conséquence du barrage des glaces flottantes les niveaux d'eau à Štěchovice étaient, durant janvier et février 1942, dans les limites de + 250 cm à + 325 cm, les débits étant au-dessous de la moyenne et ils n'ont atteint que la valeur maximum de + 335 cm /Fig. 3/. Il est évident que les mesures prises ont donné la satisfaction.

L'origine des plus grandes difficultés consistait, comme d'habitude, dans la formation des glaces flottantes qui se produisent durant tout l'hiver dans les raies de la nappe d'eau libre en amont du chantier du barrage de Štěchovice. La surface de ces raies change constamment suivant la variabilité de la température de l'air. Dans la table 2 figurent les dimensions d'une nappe libre,

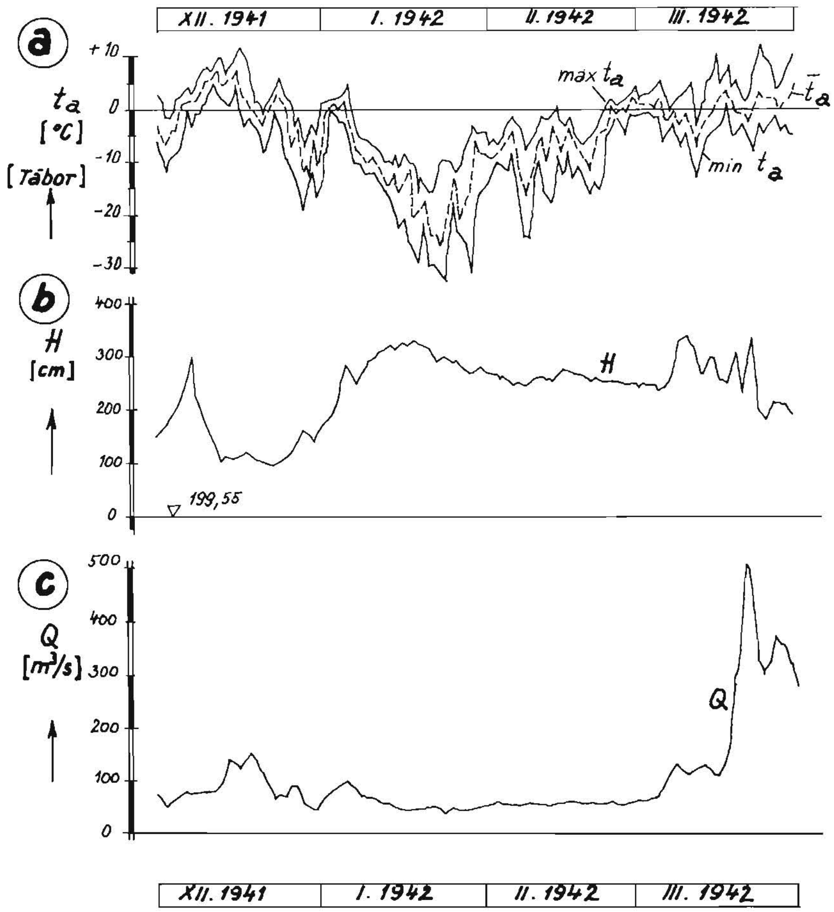


Fig. 3. Le cours de l'hiver 1941-1942 à Štěchovice:
a, b, c - comme sur la figure 2.

après les gels d'une longue durée, du 12 février 1942, et pour les températures élevées de l'air du 5 mars 1942. Le réservoir de Vrané a été sous une serre de glace du 27 décembre 1941 jusqu'au 19 mars 1942.

Table 2.

Les dimensions des raies de la nappe non prise

km	Dimensions de la nappe libre /m/				Remarques
	12/2/1942		5/3/1942		
	Longueur	Largeur	Longueur	Largeur	
151,850	-	-	10	5	
152,450	-	-	40	5	
152,700	-	-	20	10	
153,000	-	-	80	10	
153,5 - 154,2	-	-	5	5	5 orifices
154,300	100	20	150	30	5 x 5 m
155,700	200	20	300	25	
156,000	40	10	50	10	
156,100	60	3	-	-	
156,200	60	5	80	10	
156,2 - 157,3	400	10	1100	20	
	250	10			
159,100	40	2			
Surface totale, m^2	13 380		35 300		

Une part de la glace flottante est absorbée le long des bords du trajet d'amont, mais le propre trajet de déposition commence dans le km 159,5. C'était déjà le 12 janvier 1942 que la glace flottante a pénétré jusqu'au km 165,0 et le 18 mars jusqu'au km 167,2 /Fig. 1/, c.à.d. 6 km environ dans le réservoir. Le front des glaces flottantes avançait alors durant tout l'hiver.

Par des mesurages multiples effectués dans le km 162,041 de la section transversale des débris flottants de glace et du courant d'eau durant tout l'hiver on a constaté, que la surface relative des débris ne change pas d'une manière substantielle au cours de l'hiver. Plusieurs observateurs sont au contraire d'avis qu'au cours de l'hiver la quantité des débris de glace diminue et alors ils soutiennent l'affirmation que le coefficient de vitesse dans l'équation Chézy augmente d'après la durée de l'hiver. Le parcours suivant le temps de la relation mutuelle de la surface totale du débit S, de la partie bouchée par les débris flottants de glace et du lit libre est marqué dans la figure 4. De la comparaison avec les températures de l'air d'après la figure 3 on voit que même sous l'influence de l'augmentation considérable de la température de l'air au-dessus du zéro la partie du profil bouchée par les débris de glace n'a pas diminué; le débit a augmenté et par suite le niveau d'eau est monté et la partie du profil avec l'eau courante s'est agrandite, mais la surface du profil appartenant aux débris de glace est restée à peu près constante. Cet état a duré jusqu'à l'écoulement des glaces et on pouvait observer seulement une diminution de la densité des débris.

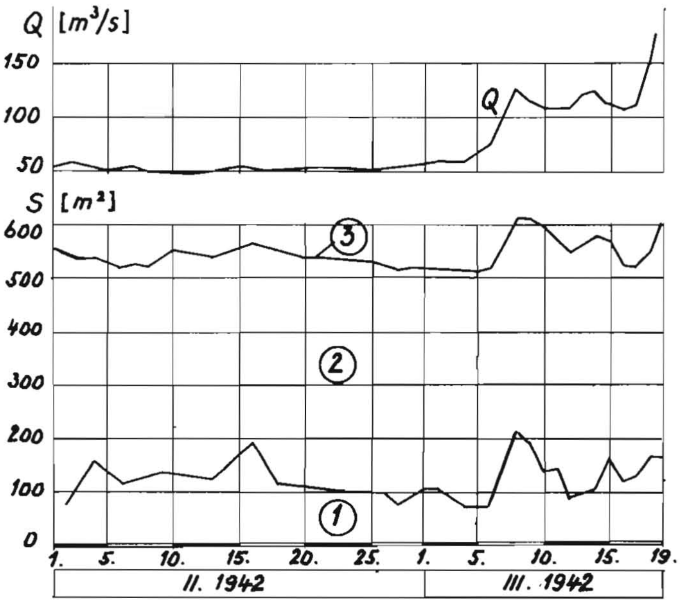


Fig. 4. Le cours du bouchage du profil km 162,041 par sorbet

1 - l'eau coulente; 2 - sorbet;

3 - surface totale

ICE SYMPOSIUM 1970
REYKJAVIK

ON FORECASTING MAXIMUM WATER STAGES DUE TO ICE JAMS FOR THE CASE OF THE DNIESTER RIVER

SUR LA PREVISION DES NIVEAUX MAXIMA DUS AUX EMBACLES POUR LE CAS DE LA RIVIERE DNIESTRE

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Synopsis

Considered is the possibility of forecasting maximum stages due to ice jam formation at locations in the backwater of repeatedly occurring ice jams. The relation between the maximum water stage due to ice jams at the location considered and the maximum daily rate of water stage rise upstream is shown to be very strict and to lead to a timely (several days in advance) prediction of the maximum water stage on rivers for which sufficiently comprehensive long-term data are available (e.g. the Dniester).

Résumé

Les auteurs analysent le problème de la prévision des niveaux maxima d'embâcle dans les zones de remous du aux embâcles qui s'y forment constamment. On a démontré que sur les rivières ou quelles on a fait les observations détaillées et à long terme, (Dniestre) il existe une relation très stricte entre le niveau maximal d'embâcle dans le site considéré et l'intensité maximale journalière de la levée du niveau dans les sites amont, cette relation permettant de prévoir à temps le niveau maximal.

Ice jams are inherent to the annual regime cycle of many rivers in the Northern Hemisphere. Such phenomena are detrimental to the national economy bringing about floods and possible failures of hydraulic structures.

One of the methods of controlling ice jams is their timely forecasting, i.e. predicting their location and the maximum possible rise of the water stage due to the ice jam – ΔH_j .

The problem of forecasting the maximum water stages due to ice jams cannot be solved without predicting the probable locations of ice jams. The latter proves infeasible or quite impossible in many cases. Therefore any more or less justified forecasting of the magnitude of an ice jam can be carried out only for locations where ice jams will occur repeatedly, viz. morphological anomalies, reaches characterized by gentler slopes, etc. The rise of the water stage due to an ice jam (ΔH_j) depending on diverse factors and/or their combinations in different years, chance plays a major part in evaluating it. The same considerations preclude solution of theoretical or experimental problems on ice jam prediction for specific cases. Hence relationships to be used in ice jam forecasting can be obtained only by way of using field observation data considering the main factors causing ice jam formation.

The latter include:

1. Amount and rate of ice movement towards the ice jam;

2. Flood rate, in particular, the maximum (based on mean daily values) rate of water stage rise during the ice run,(J_S);

3. Obstacles to ice drift (abrupt bends and contractions of the channel, islands, etc.) .

When a sufficiently comprehensive series of long-term data on the elements of river hydrometeorology is available, it becomes possible to establish a correlation between ΔH_j, and the main factors causing ice jam formation. The Dniester was chosen for our studies as a river where ice jams are liable to occur most frequently. At the same time, data for the period from 1945 to 1969 being available, it is one of the most thoroughly investigated rivers.

Based on long-term data series for nine gauges (located in the back-water of the ice jam) relationships between J_S and diverse factors clearly indicate that the rate of water stage rise, ΔH_j, is the key factor here. It should be noted that in certain years other factors may become predominant (due to ice conditions during the debacle, meteorological conditions during jam formation etc.) and change the relationship $\Delta H_j = f(J_S)$. Suitable corrections are to be introduced in such cases using correlations between ΔH_j and other factors.

Being given such correlations of $\Delta H_j = f(J_S)$ for individual line gauges located in the backwater of ice jams at stationary sites, and evaluating J_S beforehand from the expected snow melt run-off and weather forecast, one can predict the stage rise due to the ice jam with a certain degree of accuracy.

However such a procedure is schematic and does not consider the actual specific conditions. The authors suggest that the correlation between ΔH_j, at the line gauge in question, and the rate of rise of stage, J_s' , at the upstream gauges should be made use of.

Such relationships were established for three locations on the Dniester river (Mogilev-Podolski, Soroki, Kamenka) in the backwater of permanently recurring jams, with gauges located 100 to 6oo km upstream.

Quantitative evaluation of the effect of the factors conducive to jam formation was carried out by the least squares method, the degree of correlation being determined by correlation analysis. The correlation coefficients in the equation $\Delta H_j = f(J_s')$ are near unity, which is indicative of a close agreement of the values.

Figure 1 shows the correlation between ΔH_j (the town of Soroki) and J_s' (the village of Zhvanets) , the distance between them being 280 km. The curve can be described by the equation

$$\Delta H_j = 114 + 1{,}15 J_s' \qquad (1)$$

The values of ΔH_j obtained from this equation fit the actual ones fairly well. During some years occur deviations from the relation (Fig.1) the effect of additional factors (viz. greater thickness of ice cover in 1956, 1963, 1969; simultaneous breaking of ice cover on the tributaries in 1964 etc.) .

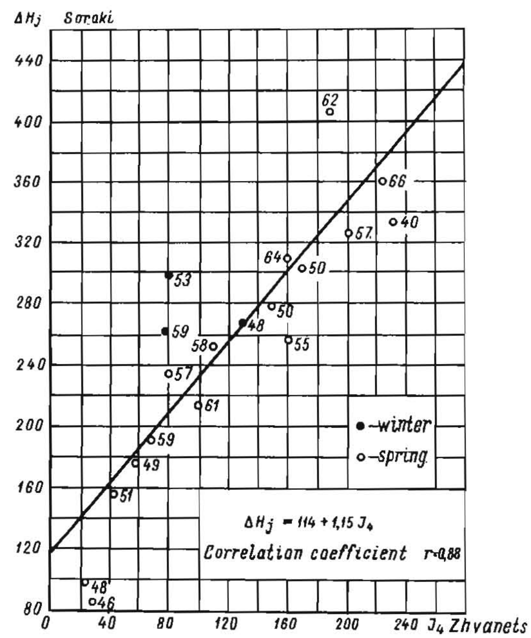

Fig.1.

They can be taken into account by supplementary dependencies $\Delta H_j = f(\Sigma\theta_-)$ and $\Delta H_j = f(J)$. Similar relationships between ΔH_j and secondary factors were derived for locations mentioned above, the correlation coefficient equalling 0.5—0.6.

Considering the above factors the relationship for ΔH_j may be expressed by

$$\Delta H_j = C_1 + C_2 J_S' + C_3 \Sigma\theta_- + C_4 J \qquad (2)$$

Where $\Sigma\theta_-$ = the accumulated negative air temperatures, which indirectly characterize ice thickness; J = rate of temperature rise equal to the accumulated positive air temperatures from the moment of the steady transition to above-zero temperatures up to the start of the ice run, related to the duration of the period; C_1, C_2, C_3 and C_4 = coefficients established from hydrologic data for a number of years in the past.

The ratio adopted in the Weather Service Manual i.e. that between the root-mean-square error ($\bar{S}$) and the standard deviation of the variable ($\bar{\sigma}$) to be predicted is used as a basic criterion to check the effectiveness of the procedure for forecasting ΔH_j. The values obtained for the ratio $\frac{\bar{S}}{\bar{\sigma}} = 0.50$ and the correlation coefficient $r = 0.88$ for the procedure developed to forecast ΔH_j on the Dniester river at the town of Soroki fall into a good quality category.

The timely prediction of ΔH_j, by the method evolved depends on the time necessary for the ice edge movement during the ice break from the first upstream gauge to the line gauge under consideration.

For the case of the Dniester river at the town of Soroki according to the relationship represented in Fig.1 ΔH_j was forecast on the average two days in advance, the time varying between 24 hrs and 4 days depending on the specific conditions of the debacle.

ICE SYMPOSIUM 1970
REYKJAVIK

ETUDE DU PLONGEMENT DES GLAÇONS SOUS UN OBSTACLE

A STUDY ON DIVING OF ICE FLOES UNDER AN OBSTACLE

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Résumé

On étudie le problème du plongement des glaçons sous un obstacle dont l'épaisseur est égale à celle d'un glaçon. Sur la base de la nature physique de ce phénomène on a établi les frontières de reproduction du phénomène lors de son étude en laboratoire, les frontières étant confirmées expérimentalement. Les expériences effectuées dans un canal hydraulique avec des glaçons en paraffine ont montré que la vitesse critique à laquelle commence le plongement ne dépend que de la longueur du glaçon.

Synopsis

The problem on diving of ice floes under an obstacle whose thickness is equal to that of an ice floe is considered. Based on the physical nature of this phenomenon, the boundaries of simulating this phenomenon when studied under the laboratory conditions are established, these boundaries being corroborated experimentally. The experiments performed in a hydraulic flume using paraffine ice floes showed that the critical velocity at which diving starts is a function of the ice floe length only.

Le mouvement des glaçons dans un écoulement, et en particulier les phénomènes surgissant lors de l'impact des glaçons sur un obstacle, se rapportent au nombre de problèmes de l'hydraulique et de la glace dont les lois sont mal connues. Cependant, ces problèmes ont une grande importance pratique dans toute une série de cas (formation des embâcles en rivières, progression du bord d'un champ de glace vers l'amont en période de congélation, arrêt de la glace par les barrages pour les corps flottants et par les estacades).

Il est à noter que l'étude de l'entraînement des glaçons était effectuée principalement pour les cas d'un écoulement par-dessous la vanne sur la crête d'un déversoir et les pertuis approfondis afin de déterminer les ouvertures admissibles /1 et 2/.

Dans le cas considéré l'épaisseur de l'obstacle est beaucoup plus faible que dans le cas d'un écoulement par-dessous la vanne et le plongement des glaçons dépend essentiellement de la vitesse superficielle. Les recherches analogues étaient effectuées au Canada, mais les résultats obtenus donnent à les traiter critiquement et, en particulier, à étudier ce phénomène par de nouveaux procédés.

Le plongement des glaçons sous un obstacle dépend d'un grand nombre de facteurs parmi lesquels on peut citer la vitesse V et la profondeur H de l'écoulement, l'approfondissement h de l'obstacle, la longueur ℓ et l'épaisseur δ du glaçon. Il est plus commode d'adopter constants les facteurs complémentaires tels que: angles de biseau du glaçon et de l'obstacle, angle d'approche du glaçon, résistance de la glace, etc.

En considérant un glaçon de forme carrée de côté ℓ, ayant, comme obstacle, un angle de biseau droit et une résistance empêchant la destruction lors d'un impact, ainsi qu'en considérant l'impact du glaçon sur l'obstacle sous un angle droit, on peut écrire une expression pour la vitesse superficielle de l'écoulement à laquelle commence le plongement:

$$V_{cr} = f(H, h, \delta, \ell)$$

La profondeur de l'ecoulement influencera le plongement dans le cas ou la dimension du glaçon ℓ et la profondeur H sont commensurables. Pour rendre plus facile l'analyse, considérons le cas où la profondeur n'influence pas le plongement ($H \geqslant 2\ell$). Cette hypothèse permet de confronter les résultats des expériences réalisées dans les conditions des profondeurs égales.

En considérant le cas du plongement des glaçons sous le couvert de glace de la même épaisseur, on peut écrire:

$$V_{cr} = f(\delta, \ell)$$

Dans les essais de laboratoire on a utilisé en tant que glaçons des minces plaques en paraffine carrées (poids spécifique de la paraffine est de 0,92) de côté ℓ et d'épaisseur δ . 28 "glaçons" étaient subdivisés en cinq groupes - δ - 0,6; 1,0; 1,5; 1,8; 2,8 cm, dans chaque groupe la valeur de ℓ variant de 2,5 a 40 cm.

Considérons une partie de l'écoulement limitée par des plans tracés par la surface libre et parallèle à celle-ci à une profondeur $H_1 = \ell$. De telles zones peuvent être choisies dans un courant quelconque et indépendamment de la profondeur H dans chaque zone auront lieu les conditions plus ou moins analogues, en ce qui concerne la répartition des vitesses. Un tel procédé permet de confronter et de généraliser les résultats des essais expérimentaux effectués pour différentes profondeurs H , mais si $H \geqslant 2\ell$. De plus, en admettant différentes valeurs de δ , on peut parler d'une série d'échelles ce qui donne lieu à la transposition des résultats obtenus en nature.

Le cas considéré du mouvement du corps dans l'écoulement peut être représenté si Fr = idem et Re = idem. Comme il est impossible de remplir simultanément ces deux conditions, la reproduction du phénomène doit s'effectuer dans la zone d'auto-simulation.

Si l'on construit, sur la base des essais effectues, la relation $V = f(Re)$ ou $Fr_{cr} = \frac{V_{cr}^2}{gH} = f\left(\frac{VH}{\nu}\right)$, on obtient Fr = const = 0,013. Cela donne $\frac{V}{\sqrt{gH}}$ = 0,112. Cette valeur se trouve entre les valeurs données dans les travaux des savants canadiens /3 et 4/. Cela confirme encore une fois que, selon les conditions de l'expérience, on peut obtenir une relation particulière Fr = const qui ne peut être transférée en nature à cause du choix des valeurs arbitraires des profondeurs de l'écoulement sur le modèle.

Si l'on examine, selon le schéma choisi, un écoulement de profondeur $H_1 = \ell$, la relation.

$$Fr_\ell = \frac{V_{cr}^2}{g\ell} = f\left(Re = \frac{V_{cr}\ell}{\nu}\right)$$

aura la forme montrée sur la figure 1. Elle réunit les résultats des essais effectués pour différentes δ (c'est-à-dire elle réunit la série d'échelles) et pour les rapports δ/ℓ = 0,05 - 1,0.

La forme de la relation montre que l'auto-reproduction commence quand $Re_\ell > 40 \cdot 10^3$. Cela signifie que les données obtenues pour la zone avec $Re_\ell > 40 \cdot 10^3$ et caracterisées par Fr_ℓ = 0,035 peuvent être transférées en nature dans le cas du plongement des glaçons $\ell \leqslant 0{,}5H$.

Si Fr = 0.035, on obtient $V_{cr} = \sqrt{0{,}035 g\ell}$. Ainsi, on détermine expérimentalement que la vitesse critique assurant le plongement des glaçons sous le bord du couvert de glace de la même épaisseur ne dépend que de la longueur du glaçon.

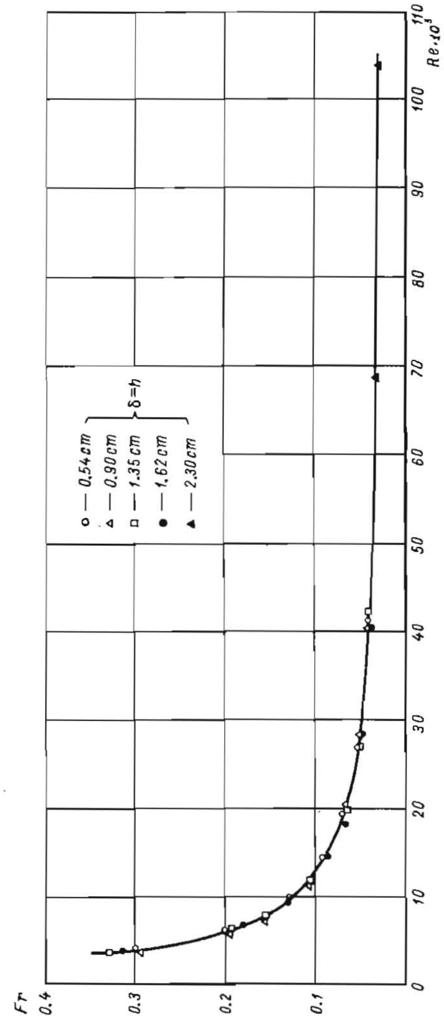
Fr
0.4
0.3
0.2
0.1
0
10
20
30
40
50
60
70
80
90
100
110
Re·10³
o — 0,54 cm
Δ — 0,90 cm
□ — 1,35 cm
● — 1,62 cm
▲ — 2,30 cm
δ=h

Fig.1.

Références

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ICE SYMPOSIUM 1970
REYKJAVIK

PILING UP OF ICE ON SEASHORES AND ON COASTAL STRUCTURES

ACCUMULATIONS DE GLACE SUR LES RIVAGES MARITIMES ET SUR LES STRUCTURES CÔTIÈRES.

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SYNOPSIS

This paper describes some results of field observations of ice pilings in the Baltic and in the Great Lakes in the United States and Canada. Although the forces exerted on the ice by winds, currents and waves may vary considerably and the structural characteristics of the ice as well as bottom conditions and geometry without doubt play an important role for modes and relative magnitudes of ice pilings, certain common features seem to exist particular with respect to the geometrical behaviour of pilings. Certain simple experiments in a laboratory tank seem to have confirmed the experience from the field qualitatively regardless of shortcomings in model scaling due to lack of knowledge on forces and structural abilities of the field ice in various degrees of crushed condition.

Cette publication décrit quelques résultats obtenus en observant, directement sur le terrain, des accumulations de glace sur la Baltique et sur les Grands Lacs aux U.S.A. et au Canada. Bien que les forces exercées sur la glace par les vents, les courants et les vagues puissent varier considérablement, et que les caractéristiques structurales de la glace ainsi que les conditions et la disposition des fonds jouent certainement un rôle important pour la formation et les dimensions relatives des accumulations de glace, il semble exister un certain nombre de traits communs, particulierèment en ce qui concerne le comportement géométrique des accumulations. Certaines expériences simples, dans un réservoir de laboratoire, semblent avoir confirmé celles faites sur le terrain, réserve faite qualitativement des insuffisances imputables, pour échelonner les maquettes, à une connaissance incomplète des forces et des possibilités structurales de la glace réelle, soumise à des conditions variées d'écrasement.

INTRODUCTION

Only little information seems to be available on the behaviour of sea and lake ice versus shores and shore structures. Many different kinds of sea and lake ice exist (ref.5). The part of the ice/shore - ice/shore structure interaction which has particular interest in this paper is the piling up of ice on or in front of a shore or shore structure. The forces which cause piling up of ice are mainly due to winds and currents. Wave forces are in action when the belt of ice in front of the shore structure is so narrow that waves penetrate through the belt without losing all its energy by damping effects.

Floating ice is exposed to shear and drag forces by winds, currents and waves. The shear force by wind may be written $\tau_w = 1/3600\ u_w^2$ (kp/m^2), where u_w = wind velocity in m/sec, and the friction coefficient for wind tensile stress is given the value 0.0023 (ref.4). The shear force by water current can be written $\tau_c = 1/4\ u_c^2$ (kp/m^2), where u_c = current velocity in m/sec (ref.6).

Wind, waves and currents transfer huge forces to floating ice. Any shore or coastal structure that obstructs movement of the ice may be exposed to ice pilings, the extent and geometry depending upon depth, frontal slope and roughness, ice properties and the actual forces by the ice (ref.1).

FIELD EXPERIENCE ON PILING UP OF ICE ON SHORES AND COASTAL STRUCTURES

The Department of Port and Ocean Engineering asked different institutions in Scandinavia, the United States and Canada for information on ice piling on shores and coastal structures. A short abstract of some results is given below.

Scandinavia

Nordre Røse Lighthouse is located in the Sound at Copenhagen. The construction is shown in Fig. 1. The dashed lines indicate the design before the reconstruction in 1893. This lighthouse was exposed to considerable ice piling in 1892 and 1956. In 1892 the ice reached an elev. of ab. 10 m above sea level (Fig.2) and in 1956 an elev. of ab. 8 m. It seems to be no significant difference in the piling before and after the reconstruction. From Fig. 3 it may be noted how ice first filled up the platform just above sea level (see also Fig.11). This resulted in a nearly straight slope from the water line up to the top of the deck upon which the ice piled up easily. The resulting slope of underlying ice was somewhat steeper than the slope was before the reconstruction. This together with "the delay" due to the platform, may cause a somewhat less piling on the lighthouse.

In the Danish seas, particularly at the Øre-sound, (between the Danish island Seeland and the southernmost Swedish province Skaane), ice piling of ab. 10 m and more takes place as a result of wind and current action, particularly at sloping shores, while ice piling hardly ever occurs at vertical walls if the water is not very shallow in front of them. Typical examples of ice piling on sloping shores is shown in Fig.4, which are from the shores near Hamlets castle KRONBORG, at the entrance of the sound.

Many ports in Denmark and Sweden occasionally experience ice piling. Examples in Denmark are the Port of Aarhus and the Port of Skagen, both in Jutland, where ice occasionally climbs the rubble mound breakwaters up to elevations exceeding 4 m, and damage the armour layer. However, the ice does not climb the old vertical caisson jetty at the Port of Aarhus. Fig.5 and 6 show ice climbing in March 1963 on the 1:2 slope rubble mound breakwater at Knudshoved ferry-port on the Great Belt shown in Fig.7. Maximum elevation of the ice was ab. 8 m. The ice caused considerable damage to the armour layer. Knudshoved was exposed to ice piling in February 1941 too, when the ice also climbed the breakwater up to elevations of ab. 8-9 m. The ice floes had a thickness of ab. 0.4 - 0.5 m and up to 20 m floes were found on the lee-side of the breakwater. Many ports which occasionally are subjected to strong ice drift, have experienced that ice does not climb vertical faced structures if the depth in front of them exceed ab. 5 m. An example of ice piling in front of a vertical structure is shown in Fig. 8, the Port of Hälsingborg on the Øre-sound between Sweden and Denmark, Design Fig.9. The depth in front of this breakwater varies from 6 to 11 m.

United States and Canada

At Duluth, Lake Superior, northeasterly winds cause westward ice drift. When the ice runs aground on the shore at Duluth or is stopped in front of structures, it builds up pressure ridges above the surrounding field usually at a certain distance from the point of contact. Storms may form more ridges in front of the first ridge. Before Lake Superior froze over during the 1966/67 winter season, a total of three ice-ridges were formed west of the Superior Entry. Surveys revealed that ice climbed to 9 m elev. on a rock reef with slope 1:5 to 1:10. The actual thickness of the ice from bottom to crest of ice pile may have been of the order 9 to 15 m. Accumulation of broken ice under wave action as well as ice pilings up to 9 m on rock reefs and slopes were observed. Vertical walls however, did not cause ice pilings.

Based on information by the Northumberland Causeway's interests in Canada, the ice at Cape Tormentine, Lake Eric, may pile up very fast on rubble mounds to elev. exceeding 7 m. At Summerside, Prince George Sound, ice piled up on a steep beach to elev. 9 - 11 m above lake level. According to experience by Dr. Peyton (ref.2) ice-stacks up to 10 m high have been observed along gently sloping beaches in the Arctic Ocean.

DISCUSSION AND CONCLUSION

Based upon observations in the field the following general conclusions may be drawn:

1) Sloping shores and structures favor ice piling. As a result of wind and current forces, ice may pile up to elev. of 10 - 15 m above still water level.
2) Vertical walls do not favor ice piling. If the depth in front of the structure is sufficient the ice does not climb but is rather forced down. When depth in front of the structure including any filling e.g. in the form of a rock mound was above 5 m, no piling up took place in most cases, but with 4 m depth (including a rock mound) piling occurred at Hals Barre Lighthouse in Denmark. This does not allow the conclusion that 4 to 5 metres is a "critical depth". Such depth must depend upon the actual exposure. Cases mentioned in this paper are examples on medium to heavy situations. In Lake Superior ice ridges formed at depths up to approx. 4.5 to 5.5 m which is in agreement with the experience from Denmark.
3) Fig.10 illustrates scematically ice piling on a vertical wall, Fig. 11 piling on a sloping wall, Fig.12 piling on a berm wall and Fig. 13 the formation of ice ridges in shallow water.
4) Laboratory tests were run in an attempt to simulate the behaviour of ice when it is forced against coastal structures. The tests were carried out by pushing the constructions up against a "sea" covered with ice floes of wax. Performing the tests like this,side wall effects were largely eliminated. These tests confirmed qualitatively the field experience that ice does not climb vertical walls located at deeper waters (e.g. exceeding approx. 5 meters) and that tendency to climbing increases with decreasing slope of structure.
5) The experience mentioned above seems to reveal two main principles:
   a) Prevention of ice piling requires vertical walls of substantial design to withstand heavy horizontal ice forces.
   b) Sloping walls cause ice pilings whereby ice forces and energy are absorbed by gravity, friction and compression forces. The elevation of the ice pilings as well as the forces involved depend upon slope angel, roughness of armour, and ice properties.

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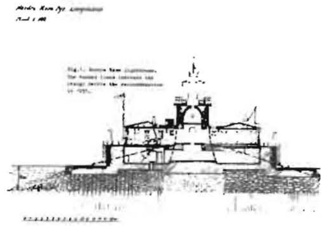


Fig. 1 Nordre RØse Lighthouse

Fig. 2 Ice piling on Nordre RØse Lighthouse

Fig. 3 Ice piling on Nordre RØse Lighthouse

Fig. 4 Ice piling at Kronborg, Denmark

Fig. 5 and 6 Ice piling at Knudshoved ferry port, Denmark

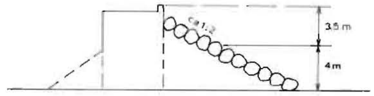
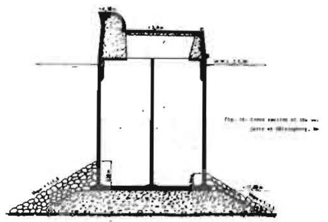


Fig. 7 Cross section of breakwater at Knudshoved ferry port, Denmark

Fig. 8 Ice piling on the vertical breakwater at Hälsingborg, Sweden



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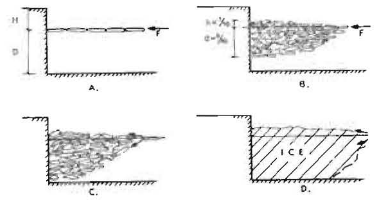

Fig. 10 Piling on vertical wall (schematic)

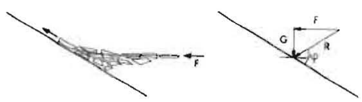

Fig. 11 Piling on sloping wall (schematic)

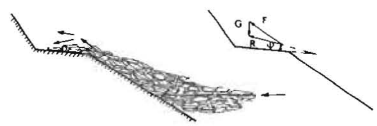

Fig. 12 Piling on berm wall (schematic)

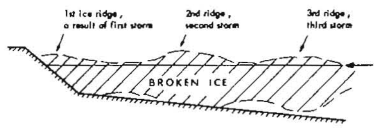

Fig. 13 Formation of ice ridges (schematic)

ICE SYMPOSIUM 1970
REYKJAVIK

THE MORPHOLOGY AND PHYSICAL PROPERTIES OF PRESSURE RIDGES: BARROW, ALASKA, APRIL 1969

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ABSTRACT

Cross-sectional profiles, internal structures, temperatures, salinities and brine volumes have been obtained from eight pressure ridges off Barrow, Alaska. Two of these ridges are described in detail and underwater photographs of the ridge keels are shown. Ridges form by either marginal "crushing" or overthrusting. Lack of isostatic adjustment appears to be common. The degree of bonding between ice blocks in a ridge is related to the cold reserve of the blocks when they are incorporated into the ridge. Representative ridge salinities are 4‰ and the keels are commonly at near melting temperatures (brine volumes between 40 and 120‰). The repose angles of the upper and lower parts of ridges appear similar.

RESUME

On a obtenu les profils de coupes transversales, les structures internes, les températures, la salinité et les volumes de saumure de 8 crêtes de poussée au large de Barrow, Alaska. Deux d'entre elles sont décrites en détail et on y montre des photographies sous-marines des bases des crêtes. Les crêtes se forment soit par "écrasement", soit par chevauchement. Un manque d'adaptation isostatique semble être courant. Le degré de cohésion des blocs de glace se trouvant dans une crête dépend de la réserve de froid des blocs quand ceux-ci sont incorporés à la crête. Les salinités typiques des crêtes sont de 4‰, et les bases sont généralement proches de la température de fusion (volumes de saumure entre 40 et 120‰). Les pentes naturelles des parties supérieures et inférieures des crêtes paraissent être semblables.

INTRODUCTION

The fact that pressure ridges and hummocked ice are an integral part of the deformation that accompanies the drift of sea ice has been known since the voyages of men such as Frobisher and Davis in the 1500's. Detailed observations of the formation and properties of pressure ridges have, however, been rare, even though ridges have long been considered the principal impediment to the movement of both surface and submarine shipping in the polar seas. To gain information on the external morphology, internal structure, and properties of pressure ridges, a number of ridges located in both the fast ice and in the pack north of Barrow, Alaska, were examined. The techniques used were crude but effective: the profiles of the upper surfaces of the ridges were determined by leveling, spot measurements of ice thickness were made by drilling, and the centers of the ridges were sampled by coring. Ice temperatures, salinities, and densities were obtained. Brine volume was computed from temperature and salinity. A total of eight ridges were investigated, two of which will be described in the present paper.

TWO RIDGES NEAR BARROW

Figures 1, 2 and 3 respectively show a general view of Ridge A3, a cross-section of the ridge, and its salinity-temperature-brine volume profiles. The profile locations are referenced to the distance axis in Figure 2. The ridge consisted in large part of ice blocks 15 to 20 cm thick, indicating that it formed when the surrounding ice was thin. Salinities in the ridge are consistently lower than in the surrounding ice and show less variation. The temperature profiles are all roughly linear, with surface temperatures varying between -6 and -9C. The ice in the ridge, therefore, has the same average temperature as the surrounding plate ice. As a result of these temperature and salinity profiles, the brine volume in the ridge was consistently lower than in the surrounding ice.

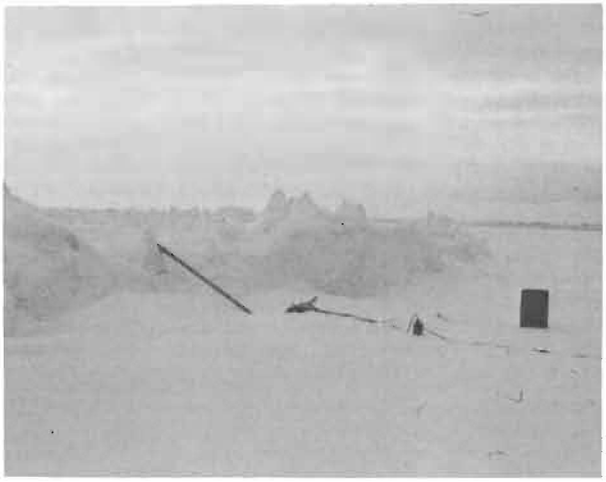

Figure 1. North edge of Ridge A3 on the profile line.

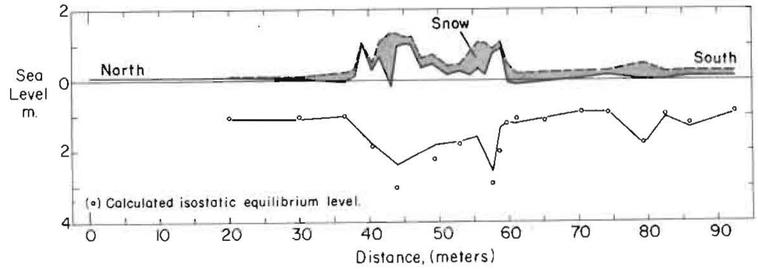

Figure 2. Transverse cross-section of Ridge A3.
The shaded area indicates snow.

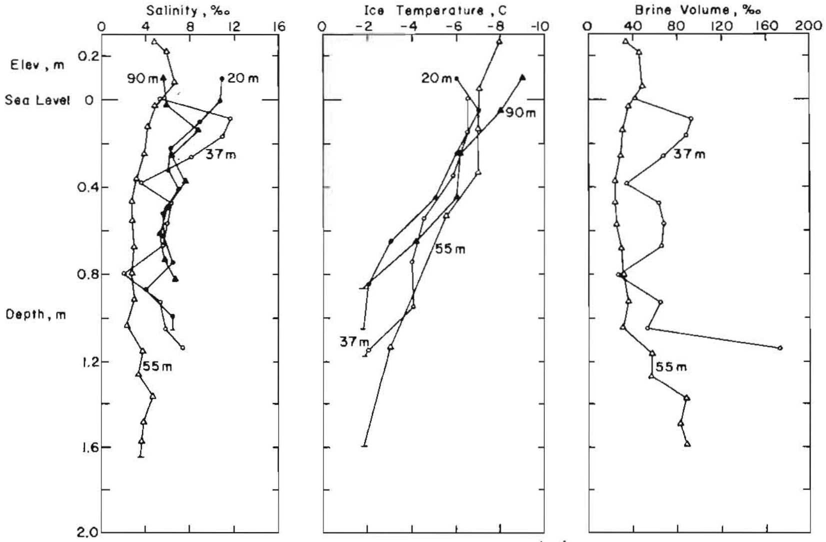

Figure 3. Salinity, temperature and brine volume profiles of Ridge A3.

No large voids were observed below the water line. The ice was surprisingly "solid" when cored and it was apparent from examining the cores that the ice in the ridge was composed of innumerable small ice blocks that had been frozen together. This type of structural identification was simplified by the fact that the plate ice incorporated within the ridge had a pronounced horizontal layering of plankton. Many times these brownish layers would be found tilted at high angles in cores from the ridges. Figure 4 is an underwater view of a portion of the keel of a smaller but similar ridge that had just formed. Note the numerous ice fragments that have been mashed together. The open circles in Figure 2 indicate the calculated isostatic levels of the lower ice surface based on the average determined densities of the snow and ice in the ridge. The sheet surrounding the ridge is in isostatic equilibrium. However, the ice in the ridge itself does not appear to be isostatically compensated. This is also indicated by the deflections in the plate ice at the edge of the ridge where flooding was observed after coring.

Figure 4. Underwater photograph of the ice in Ridge A1. The light area in the upper left-hand corner is the lower surface of the undeformed plate ice.

Significantly larger snow accumulations on and around the ridge indicate that the ridge acted as a snow fence. The fact that ridges are natural barriers to drifting snow is immediately apparent to anyone who has flown over the arctic pack in winter.

Figure 5 shows the spatial relations of three profiles from a larger ridge (A7). A general view of this ridge is presented in Figure 6. Examination of the ice blocks in the ridge showed that there were two principal thicknesses: 15 to 20 cm and 50 to 60 cm, indicating that the ridge had formed by the interaction of ice sheets of two different thicknesses. The coring logs, temperature and salinity profiles and calculated brine volume profiles are shown in Figure 7. As indicated in the core logs, this ridge was heterogeneous in structure with "layers" of sea ice alternating with snow and granular slush ice. The slush ice was distinctive with an equiaxed structure and a grain size of 1 to 3 mm. Because the slush was very poorly bonded core recovery was poor. Pronounced deterioration cavities up to 2 cm in diameter were observed in the lower portion of the ridge.

Figure 8 is an underwater photograph of a portion of the ridge keel. Note the rounding of the corners of the larger blocks, indicating that appreciable melting has occurred, and the general deteriorated appearance of the ice. In this ridge the calculated isostatic levels are of doubtful value because of difficulties in obtaining adequate densities for the snow and slush. However, the deflections of the plate ice near the ridge suggest that the weight of the ridge is partially supported by the surrounding ice sheet. The ice temperatures are as expected: the surface of the ridge is colder than the surface of the surrounding plate ice and the keel of the ridge is very near the freezing temperature of the sea water. With the exception of two "irregularities," the salinities of the ridge ice averaged 4‰. There does not appear to be any consistent difference in the brine volume profiles when compared relative to sea level.

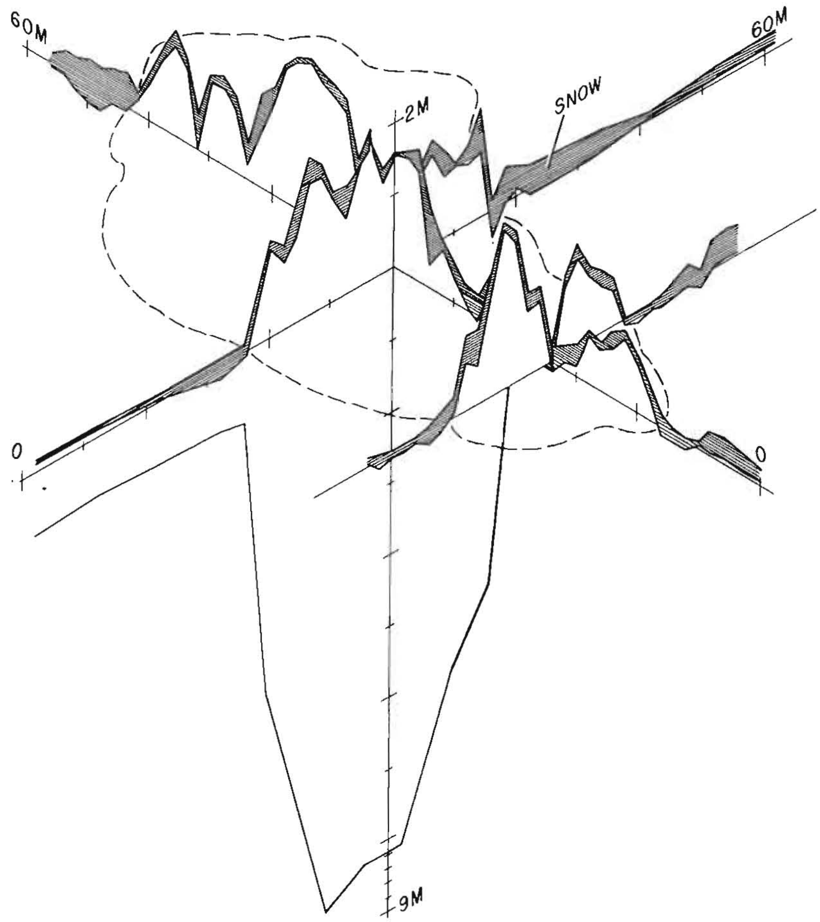

Figure 5. Fence diagram of a portion of Ridge A7.
The shading indicates snow.

GENERAL RIDGE CHARACTERISTICS

The observations at Barrow in combination with ridge profiles published by Japanese (1) and Russian (2) investigators suggest that the following statements can be made about ridges:

1. New ridges can be classified into two main types: a) those produced by marginal breaking and grinding of the interacting ice floes, and b) those formed by the thrusting of flaps of one sheet over, or under, the next sheet. When, during the formation of ridges of type (a), the relative motion is primarily compressional, the general trace of the ridge may be irregular. When the motion is primarily shear, the resulting ridge is straight. A fragment of such a shear ridge is shown in Figure 9. Large overthrusts grossly similar to finger rafting occur even in very thick ice (>2m).

Figure 6. Drilling on Ridge A7.

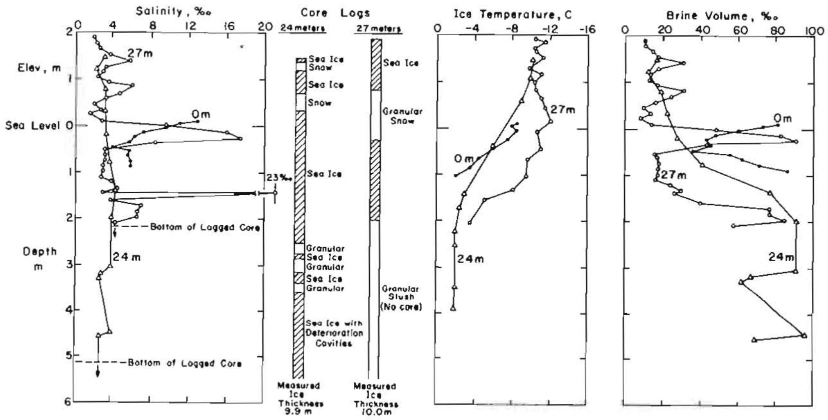

Figure 7. Salinity, temperature and brine volume profiles from Ridge A7.

2. Lack of local isostatic adjustment is common in ridges. In new ridges a significant portion of the load of the ridge is supported by deflections in the surrounding plate ice. When ridges form by thrusting, their upper and lower portions may be laterally separated by tens of meters. This obviously results in a nonisostatic condition which is compensated by deflections of the local plate ice.

3. The degree of bonding between ice blocks and, therefore, the overall structural integrity of the ridge keel would appear to be variable, presumably changing with the temperature of the ice being incorporated into the ridge. It can be shown that during the winter the cold reserve of ice blocks being incorporated into a ridge can be sufficient to cause significant inter-block ice growth.

Figure 8. Underwater photograph of ice in Ridge A7.

Figure 9. Shear ridge fragment off Barrow, Alaska.
Note the steep straight side.

4. A representative salinity for the ice in the ridges we examined was 4‰. The temperature profiles were reasonably linear except in the lower parts of ridges with pronounced keels where temperatures were roughly constant at near freezing values. The brine volume of the ice blocks in the keels varied between 40 and 120‰ corresponding to a flexural strength of 2 to 4 kg/cm^2 (5).

5. Present results do not support the contention that the average angle of repose in the above-water portion of a ridge (25°) is larger than that in the underwater portion (32°).

6. Ridges act as effective snow fences, causing large amounts of snow to accumulate both in and around their upper parts.

ACKNOWLEDGMENTS

The field work reported in this paper was performed by G. Frankenstein, N. Smith and the authors, assisted by P. Upton and R. Eisinger of AINA and by Dr. L. Breslau and LT M. Trammel of the U.S. Coast Guard. The support of the Arctic Research Laboratory at Barrow is appreciated. The overall pressure ridge research project was funded by the U.S. Coast Guard.

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DISCUSSION

R.O. Ramseier

Based on your travels in the Arctic and particularly on your detailed work on pressure ridges, could you give me some idea of the distribution of pressure ridges, especially the keels? What kind of network can be expected?

Authors' reply

Submarine sonar data indicate that the frequency distribution of keel depths is exponential. If this is so the depth-distance relationship must also be exponential. A recent study of ridge orientations in the area east of Barrow indicates that when ridges of all ages are considered together, the orientation distribution is close to random.

ICE SYMPOSIUM 1970
REYKJAVIK

THE STRUCTURE OF AN ICE RIDGE IN THE BALTIC

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Dr. Phil

Rather big ridges have been studied. The hight above sea level has been 1-2 meters and the submarine part 6-10 meters, but in some cases as much as 20 meters. The dimensions seem to depend on the thickness of the level ice from the ridge has been formed.

Characteristic for an ice ridge is that pieces on the surface will freeze together, but at a depth of 1-1,5 meter the pieces are loose.

Ice ridges are an obstacle to modern winter navigation and the icebreakers have to cut their way through them. Investigation of the structure of the ice ridges is thus even more important than before. Another problem is estimating the pressure causing the ridges.

The surface formation has been studied from aerial pictures. An ordinary camera with a focal distance of 15 cm was used. Before taking the pictures "references", i. e. black cardboard sheets 50 x 50 cm, were placed on level ice at the side of the ridge (Fig. 1). I also tried using soot, but the soot spots were not sharp enough for stereoscopic measurement.

Photographs were taken at heights of 1000, 500 and 200 meters. The best altitude proved to be 200 meters, at which a profile through an ice ridge could easily be drawn. In this case profiles were drawn at intervals of 4 meters (Fig. 2). From a height of 500 meters only a topographic map with contour lines could be drawn (Fig. 3 and 4). Experiments were made with using a stereoscopic camera on the bridge of an icebreaker, and the results looked promising.

The submarine part of the ice ridge was investigated by diving and photography. To obtain the original form of the profile an opening was made in the level ice close to the ridge using a chainsaw. If the opening was made by an icebreaker the bow propellers moved the loose pieces, which rose to the surface. This happened even if natural cracks were used, and loose pieces also rose to the surface through cracking.

When the opening was ready the skindiver went down and stretched a marked rope under the ridge. Recording the depth of every knot, he was able to make a profile of the under surface of the ice. If the water was clear he had a good view from below (Fig. 5). If possible he also measured the thickness of ice pieces or floes.

Photos were taken with an underwater camera. In order to get a continuous view of the submarine part of an ice ridge a water-tight television camera was used, which the skindiver brought along the ice wall. He was not, however, able to focus, so the pictures in our experiment were not sharp enough (Fig. 6 and 7). To avoid the inconvenience of diving in cold weather a motor-driven frame was used to carry the camera under the ice. In the frame were a television camera for following the view and an ordinary camera for taking the pictures. Unfortunately the range of the television camera was limited and the objects could not be chosen very well.

Fairly big ridges were investigated (Table 1). The highest peak rose one meter above the surface. In a few cases the altitude of the visible part of the ridge was two meters. The average depth of submarine ridges varied from 6 to 10 meters. In most cases they were formed by fairly thin ice, 15-20 cm thick. The biggest ridges were formed by considerably thicker ice, 40-50 cm thick. They also lay much deeper, the maximum depth observed being 20 meters. Ridges formed by thick ice lay at great intervals, whereas the smaller ridges were often close to one another.

One characteristic of the ice ridges in the Baltic was that the pieces were loose at the bottom. On the surface the pieces were frozen together and a coherent ice sheet was formed during the winter. The thickness of this ice sheet could be one meter or a little more. In addition, some vertical floes were frozen to the sheet at one end. This made it possible for the icebreakers to remove the loose pieces with their bowpropellers and then break the coherent sheet.

The situation of the sludge and small pieces was a subject of special interest. In the case of deep ridges only big floes were found at the bottom of the ridge. But this happened late in spring, when small pieces had probably already melted. In some cases the photos showed that pieces got smaller and smaller deeper down (Fig. 8). This aspect of the problem therefore remained unclear.

The inclination of submarine ice ridges was measured. When the ridges were formed of rather thin ice the angle was 30 degrees at most. But in the case of a deep ridge formed by big floes, the inclination was high, 50-60 degrees.

Calculations were made to determine the ice pressure necessary for formation of these ridges. Using the theory of Caquot and Michel and estimating the porosity of ice accumulation as 0.2 I obtained the following values:

d = 4 m	p = 6 tons per meter
8 m	23 " " "
12 m	52 " " "

where d is the total thickness of an ice ridge and p the horizontal pressure.

This last value may seem high. But in winter 1967, when the ice in the Gulf of Finland pushed caissonlight Tainio from its position, it was evident that such high pressure force values occur in the Baltic. This accident happened as follows.

In the fall of 1966 the cone-shaped substructure of caissonlight Tainio had been placed on its groundwork and was standing on a nine meter deep levelled rock on which crushed stones or granules were spread. As winter came early, time did not permit fixing the building to the bottom.

On February 7, 1967 it was observed that the caissonlight had been pushed some fourteen meters to the east by the ice. In the surrounding ice field the thickness of the level places varied from thirty to fifty centimeters and the brine content of ice from 0.9 to 1.2 per thousand. It was noted that floes of this thickness had formed a heap as much as 3.5 meters below the water surface on the windward side of the caissonlight, and as much as one meter on the lee side. On the night between 6th and 7th February there was a 13 m/sec wind blowing from west south west and the minimum air temperature was -5 C^{o}. It was assumed that this wind caused the dislocation of the caissonlight. According to preliminary calculations using the value 0.6 as the friction coefficient between the caissonlight and the granules under it, and considering that the estimated weight of the caissonlight was 779 tons, the force required would have to be at least 467 tons in order to move the caissonlight in a horizontal direction. As the diameter of the cylinder-shaped upper part of the caissonlight was 3.5 meters, this would give a value of 134 tons per meter.

As this value seemed considerable the technical bureau at the Board of Navigation performed tests with a miniature model, according to which the friction coefficient could in this case have been as small as 0.3, and the pressure force would thus have been 67 tons per meter across the side. However, it is worth nothing that the caissonlight had come to a stop when the corner edge met with a raised level of rock, whereas a greater pressure force could have been possible.

Efforts were made to determine the stress caused by wind and current. The calculation gave an approximate stress of 3.0 tons per meter when wind was 13 m/sec. If the velocity of the current under the ice is estimated as being as much as one knot or about 50 cm per sec. it would, according to Shuleikin, give a stress of 4.5 tons per meter. It thus seems likely that the static pressure will not give sufficiently high values.

Table 1.

The ridges investigated

Nr	Date of observation	Place	Height above sea level (m)	Depth under sea level (m)	Thick-ness of ice pieces	Angle of ice wall
1	1965 IV 1	64°39'N 24°13'E	0.8	8	30	20-25°
2	1966 II 24	64°39'N 24°13'E	0.3-0.4	0.8-1	40	(10°)
3	1966 III 3	64°39'N 24°13'E	0.3-0.4	0.8-1	40	(10°)
4	1969 II 21	63°57'N 22°25'E	0.7	5-7	55	30°
5	1969 III 3	60°08'N 26°15'E	1.2	8	(15-50)	x
6	1970 IV 8	63°41'N 21°57'E	2.6	20	40-80	50-60°
7	1970 IV 8	63°45'N 21°42'E	0.8	6-7	20-30	(30°)

Explanations:

1. The ridge lay at the edge of fast-ice west of Raahe. On the first meter below sea level was a coherent ice sheet. Then to a depth of 8 meters there were loose ice pieces and sludge. The skindiver cut the ice straight above the ridge, removed the loose pieces came up and went down.

   The brine content in the ice was not measured, but at a coastal station nearby it varied from 0.0 to 0.5 ‰.

   On April 24 the icebreaker Tarmo made a channel through the ridge, but it took some two - three hours to penetrate it.

2 and 3. Spot measurements of the ice thickness were made along a 9 nautical mile line from the fast-ice edge to the sea. On January 24 there was fairly even ice 40 cm thick. On February 24 the ice was already ridged and the thickness of the ice at level point varied from 40 to 60 cm. The dimensions of three ridges were nearly the same as those given in the table. On March 8 the ice field was broken and had moved a little towards the coast. The old ridges were almost unchanged in shape (given in the table). Some new ridges had formed.

4. The ice ridge investigated lay 10 nautical miles off the fast-ice edge, west of Mässkär in the Bay of Bothnia.

5. The ice ridge lay beside the Tiiskeri shoal in the Gulf of Finland. It is possible that the ice had been pressed against the coast several times.

6. Bay of Bothnia, 17 nautical miles southwest of Mässkär, the ice ridge investigated can be seen in Fig. 1 and 5.

7. Bay of Bothnia, 23 nautical miles west of Mässkär, the ice ridge investigated can be seen in Fig. 3 and 4.

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Fig. 1. Aerial picture of an ice ridge on Bay of Bothnia April 8, 1970.

The photograph has been taken at the height of 200 meters. The "references" for stereoscopic measurement, i. e. black cardboard sheets placed on the ice have been marked with figures. The black spots are soot.

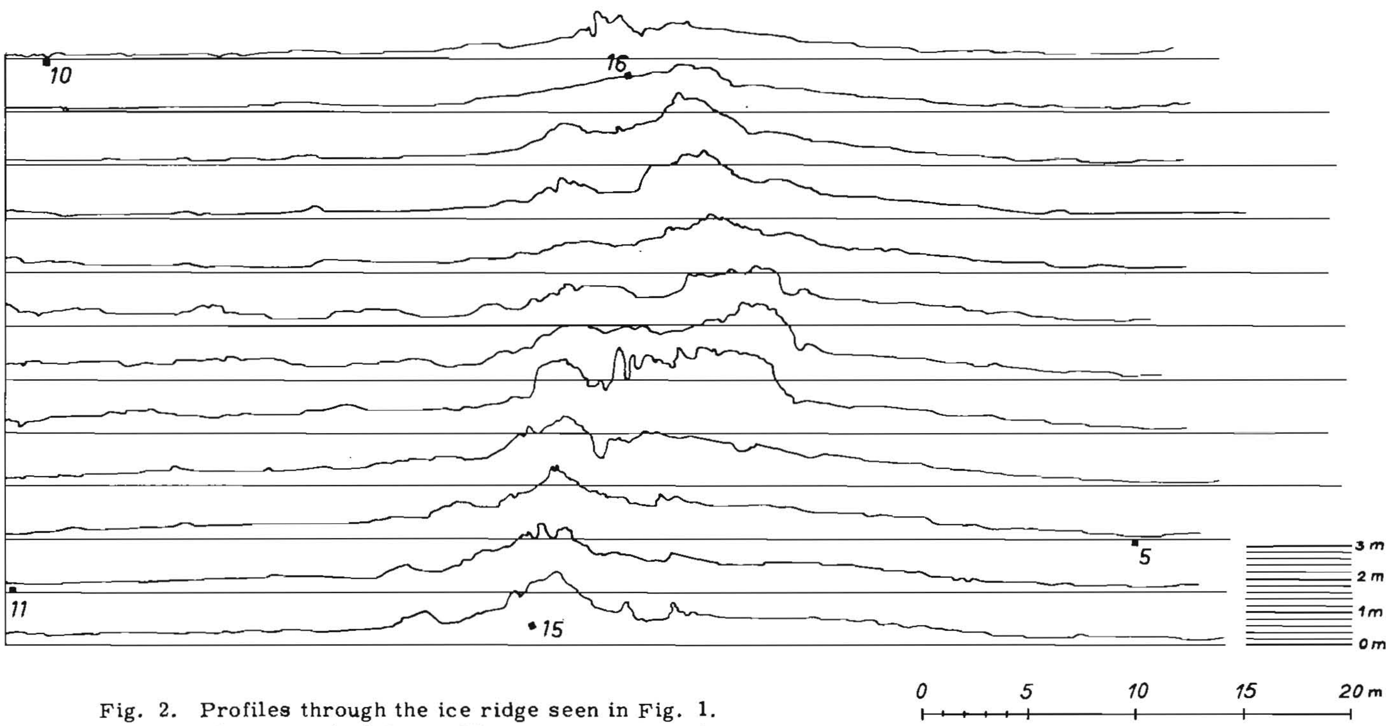

Fig. 2. Profiles through the ice ridge seen in Fig. 1. The interval of the profiles is 4 meters. The references are marked with same figures as in Fig. 1.

Fig. 3. Aerial picture of a hummocked ice field on Bay of Bothnia April 8, 1970. The photograph has been taken at the height of 500 meters. References for stereoscopic measuring are marked with figures.

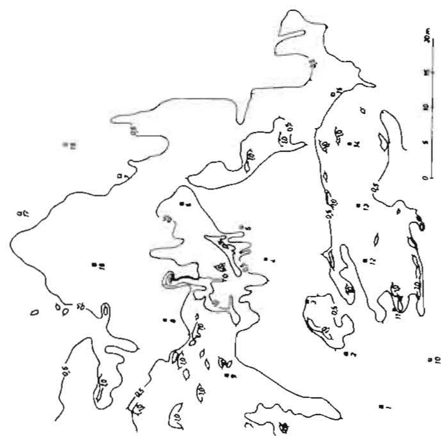

Fig. 4. Topographic map with contour lines of the hummocked ice field seen in Fig. 3.

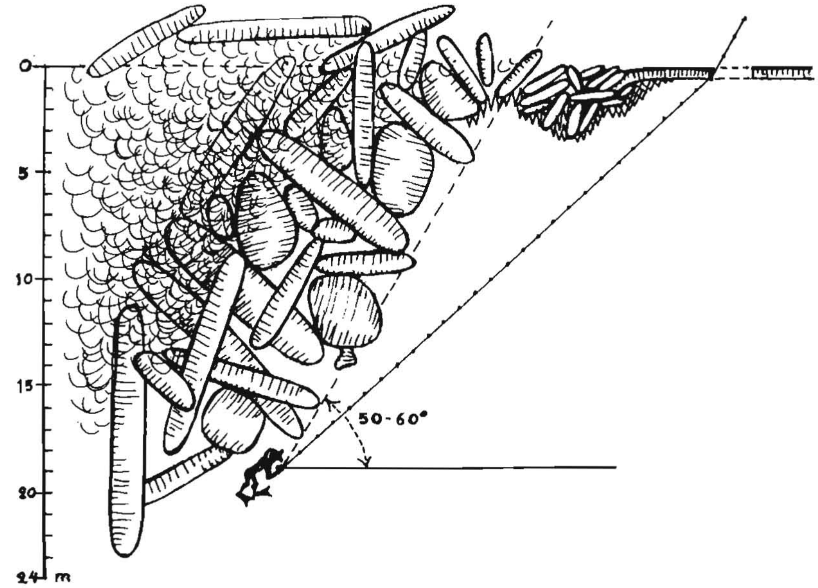

Fig. 5. The submarine part of an ice ridge sketched by a diver. The thickness of ice pieces have not been corrected.
Explanations: Close by the ice ridge sludge and pieces of thin ice, thickness 5-10 cm. Deeper down the floes get thicker.
The ends of the floes are rounded and erroted.
Sponge like pieces were seen.

BAY of BOTHNIA 8. 4. 1970

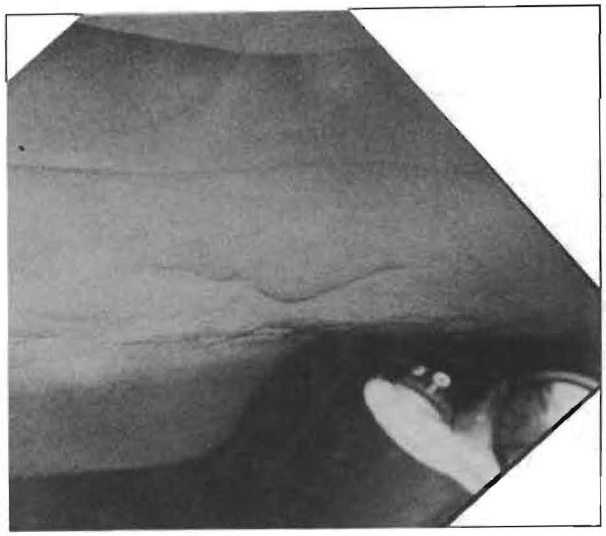

Fig. 6. The upper part of the ice ridge seen in Fig. 5. The ice sheet from sea level to a depth of 120 cm (in the left corner) consists of four floes frozen together. The`depth gauge on the front of the underwater camera is seeing in the right corner.

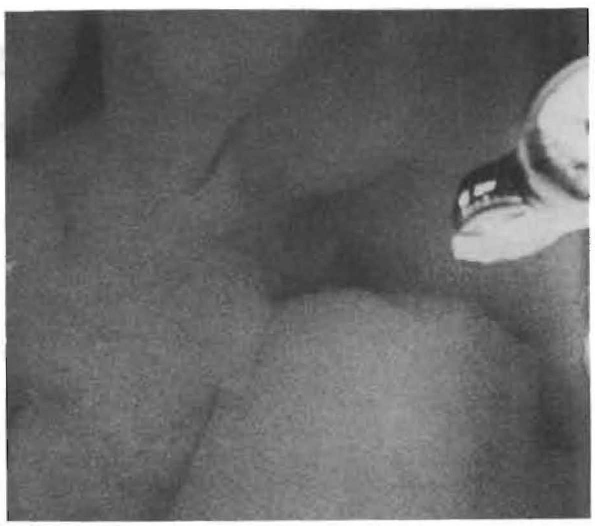

Fig. 7. The ice at the depth of 13 meters (in the same ridge as in Fig. 5.) The pieces are loose and the porosity of the ice mass is estimated to be 0. 3.

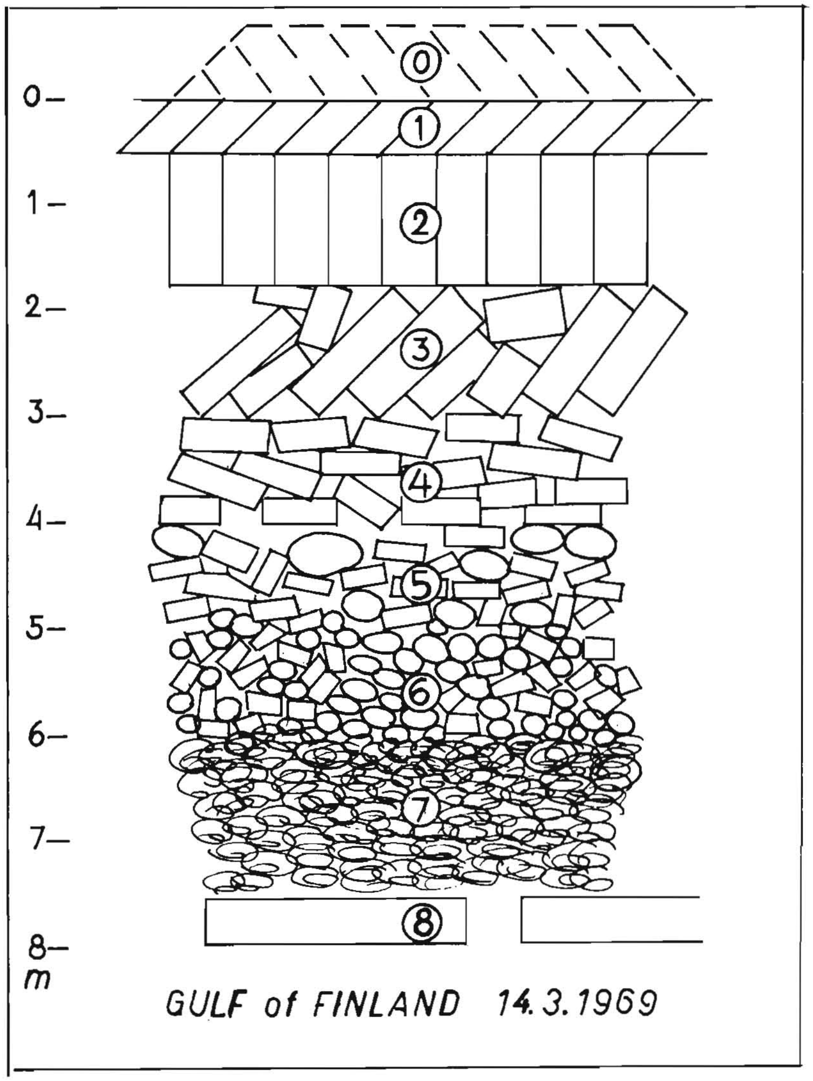

Fig. 8. Schematic drawing of an ice ridge drawn according to the pictures seen in a television camera. Explanations: 0 = the surface formation, 1 = a coherent ice sheet, 2 = floes frozen together, intermingled, 3 = big floes askew, 4 = smaller floes loose, 5 = rounded off ice pieces, 6 = small ice pieces, 7 = sludge, 8 = a level ice floe.

3.10

Discussion:

Assur: We assume that the height and other dimensions of pressure ridges are proportional to the thickness of surrounding ice sheet. In the Arctic Ocean, for example, pressure ridges are more higher than in the Baltic Sea.

What was the thickness of the adjacent ice sheet in the cases studied by Dr. Palosuo? Such date would be valuable in studying the forces which lead to the formation of ridges.

Palosuo: The thickness of ice pieces in Table 1 is nearly the same as the thickness of the adjacent ice sheet.

ICE SYMPOSIUM 1970
REYKJAVIK

ICE CONDITIONS IN THE THJORSA RIVER SYSTEM

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Synopsis.

The purpose of this paper is to give a general description of ice conditions in the Thjorsa river system, Southern Iceland. The Thjorsa river with its tributaries has the greatest power potential of all rivers in Iceland.

By the mountain Burfell, 75 km from the sea, a run-off-river power plant is situated that went into operation in 1969 (1).

Reaches of the river which remain open throughout the winter act as a great frazil ice producers (2), causing ice jams as high as 15-18 m with a volume of 10-40 Gl (3). The maximum rise in the water level occurs immediately below the section with the greatest slope.

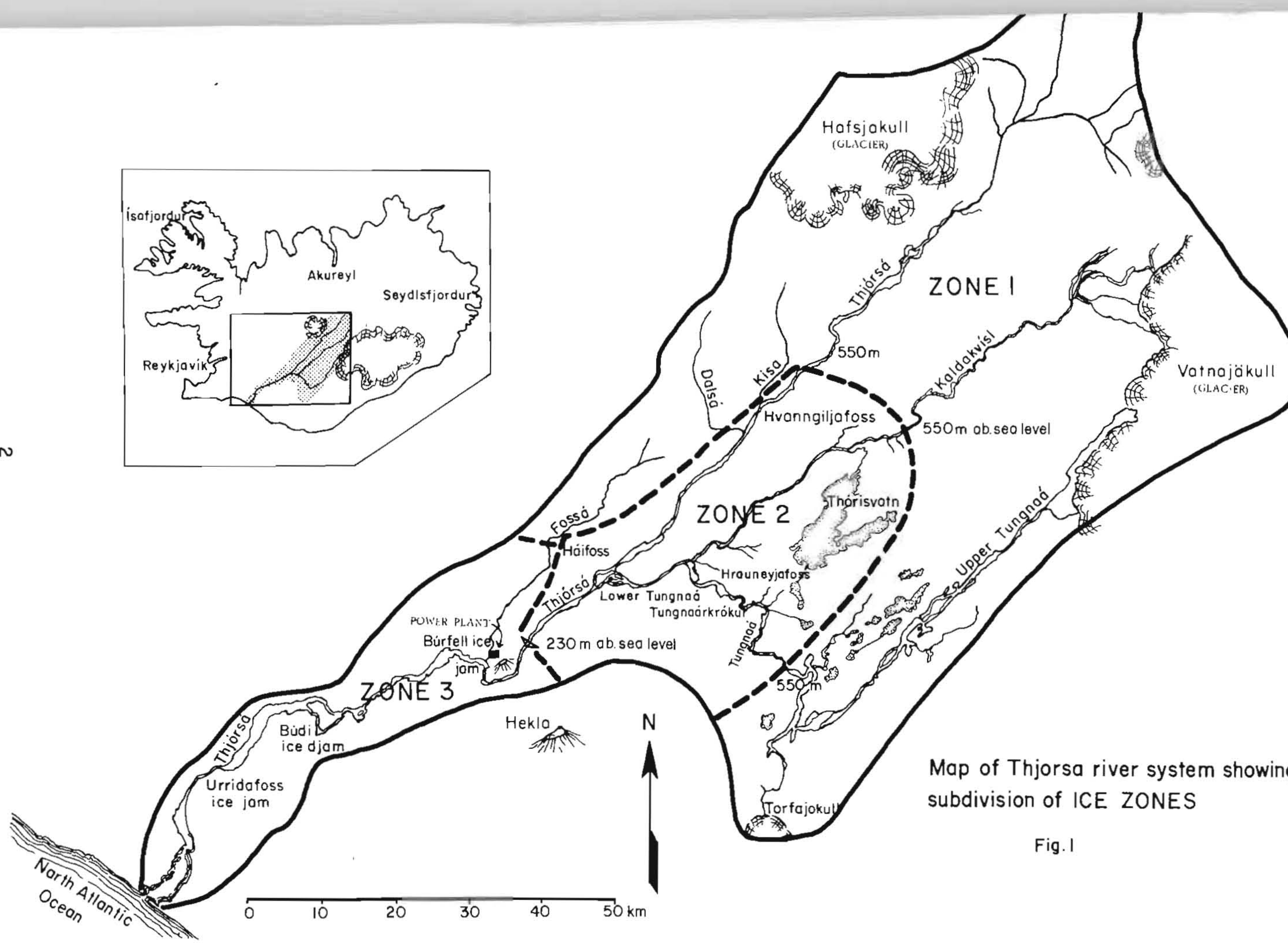
FLOW.

The Thjorsa drainage system covers 7530 km, 16% of it or 1200 km^2 is covered with glaciers. The length of the river is 230 km. Hydrological records are available from 1947. Average flow is 380 m^3 sec^{-1}. Maximum floods observed are about 3500 m^3 sec^{-1}. They occurred during melting of snow accompanied by rain in a winter storm in March,1948, and during melting of snow in June, 1949.

At the Burfell run-off-river power plant the average flow is 340 m^3 sec^{-1} but during prevailing periods of frost the flow is usually within the range of 110-170 m^3 sec^{-1}.

STREAM TYPES.

According to the classification (4) of rivers in Iceland, the Thjorsa river must be considered as a mixture of the three stream types recognised, glacial streams "J", direct run-off streams "D" and spring-fed streams "L". Interaction between these hydrologically different streams have a profound influence on the ice phenomena.



Map of Thjorsa river system showing subdivision of ICE ZONES

Fig. I

Glacial melt water "J" supplies the Thjorsa river with fine mud that remains suspended in the water and has a great effect on the shape of the river bed. It makes a wide channel on flat areas, filling all depressions between rock controls with alluvial sand and mud. On its course the river changes repeatedly from a braided river into a swift current in a gorge.

Direct run-off stream "D", also erode wide channels almost with the aid of ice-floes. The amount of glacial water and surface run-off supplied to the main river is only very minor in the wide channel during frost. When a spell of frost sets in anchor ice and ice dams easily cause a great backwater effect, decreasing the flow farther down stream.

Spring-fed streams "L" supply most of the water in the Thjorsa river during the winter. As the name implies the spring-fed streams derive their flow from ground water. The temperature of most springs in the Thjorsa river drainage area varies from 3^{o} to $6^{o}C$. The discharge is very constant all the year round. These streams originate dominently in areas of recent volcanic eruptions where the bedrock is extremely porous. All the year round these streams are carrying drift sand as bottom load. Their channels are rather narrow and deep. One of the main characteristics of spring-fed rivers is that they do not freeze near the head spring even in the most severe frosts. In this respect they are like outflow from reservoirs.

CLIMATE.

The Thjorsa river drainage area is situated between 64^{o} and 65^{o} N so solar radiation is very insignificant in midwinter but this does not tell the whole story. In Iceland there is a maritime climate. Continental climate is hardly anywhere to speak of although the difference between the weather in the outermost coastal regions and in the central mountain regions is quite pronounced. The Thjorsa river drains primarily the central highlands. For this reason and also because of variable geology of the drainage area it seems likely that the ice regime will be most clearly propound if the drainage area is divided into zones.

THREE ICE ZONES.

Zone 1 includes the uppermost headwater parts of the Thjorsa river itself and its tributaries Tungnaa and Kaldakvisl above elevation of about 550 meters. The main rivers in this zone are mostly braided rivers, so the temperature of the air affects the water easily. The flow in this zone is mostly derived from glacier meltwater and surface run-off. Due to lack of melting of glaciers in the winter and the fact that the precipitation in the headwater areas is entirely in the form of snow, the discharge in this zone decreases rapidly as the winter sets in and usually remains low throughout the whole winter season. Although the weather in Iceland is very variable from one year to another and from day to day, it can be expected that, as a rule, by the end of September or early October, that sludge ice is flowing down the main rivers out of this zone.

Although the freezing-up process is interupted perhaps many times the rivers become frozen over in this zone not later than about the middle of November and remain under stable ice and snow cover until the end of April or the beginning of May the next year. Mild weather during one or two days when storm of ordinary magnitude is blowing Atlantic warm airmasses north across Iceland is not sufficient to break up the ice. The rivers in Zone 1 can become free of ice in exceptionally mild and long periods in winter time as far up as the margin of the glacier. During such mild periods the whole river system is exposed to enormous quantities of frazil ice production with ice jams as the common sequence. This is what happened on the 10th April, 1963, when in the course of 24 hours the discharge at the gauging station, Urridafoss, 23 km from the sea dropped from 340 m^3 sec^{-1} down to not more than 20 m^3 sec^{-1}

In the freezing-up periods step-burts (Icelandic: þrepahlaup) occur occasionally and they can always be expected during those periods. These step burts are rather small, and not as catastrophic as this phenomenon may be, although from gorge stretches they can carry large blocks of almost blue ice of rectangular shape, so huge that no side is under 2 m.

Zone 2 is downstream from Zone 1 and reaches down to the lowland, that is near the Burfell damsite. A characteristic feature of this zone are large areas of open water, especially in the tributaries Tungnaa and Kaldakvisl and in Thjorsa itself downstream from the Thjorsa/Tungnaa confluence. This is primarily due to supply of heat to the river water from ground-water flowing from post-glacial lava fields such as the spring-fed streams mentioned above. Secondly the rivers tend to remain icefree because of frictional heat and the swift current, the fall in this zone is 300 m. During frost periods, especially when accompanied by strong wind the turbulent flow in a wide rectangular river bed cutting through post-glacial lavas gives rise to a great heat loss by convection, and large amounts of ice are produced.

When there is a high pressure area moving southwards over Greenland and a low pressure area west or south of Iceland is moving eastwards, strong and cold northerly or northeasterly winds can be expected in Iceland, frequently accompanied by blizzards in the highlands. Records show that this weather is the most effective in producing ice. Therefore a close watch of position and movement of the major pressure system in the vicinity of Iceland, would be profitable to give a warning to the power plant before the ice production starts.

Due to the supply of heat to the river water from ground water mentioned previously, a moderate rise in air temperature may be sufficient to render the heat balance of the river in this zone positive, and this may occur although the air temperature is still a few degrees below zero. Anchor ice becomes loose from the bottom and blocks of shore-ice are also transported downstream. The sludge is now flowing downward in a passive condition, instead of in active condition before.

The open water areas increase by such rise in temperature but also the heat losses. So it may or may not shift over again to a negative balance, depending upon the weather.

In the extremely changable weather conditions in Iceland the ice regime consists of alternating periods of freezing and thawing, a number of incomplete or poorly developed ice-cycles. The open water area in this zone that is near Burfell damsite is about 10 km^2 and is reduced rather soon by about 50% and in severe frost periods it can come down to almost 1 km^2.

Zone 3. The zone from Burfell to the river mouth. The most salient feature of the ice in this zone is the accumulation of sludge ice and the formation of large ice jams predominantly in three places. The uppermost place is southwest of the Burfell mountain, if one considers the conditions before the construction on a dam above Burfell. The other two ice jams are farther south and submerge the waterfalls Budi and Urridafoss.

Ice jam data.

Name	Max. rise in water level (m)		Approx. Volume (Gl)	
	Normal	Extremes	Normal	Range
Burfell jam	10	15	20	15-40
Budi jam		12		0-30
Urridafoss jam	13	18	20	10-40

Although this 75 km zone of the Thjorsa river is farthest from the glaciers the river is still maintaining its braided character although its bed narrows into a gorge for a 1 km long stretch. Both in this zone and in the middle zone ice bridges form where the river bed is a gorge and therefore deep and narrow. But such ice bridges which are not supported by resting on the bottom do not stop the prevailing downward moving sludge. The sludge ice passes under the bridges and so it moves farther downstream. On the other hand the sludge is retarded in its downstream movement on the braided stretches because here the water is shallows and the ice scrapes the bottom. The braided sections, with weak current and the water divided into a number of small, irregular channels, between sand bars greatly impede the flow of ice with the result that the ice starts to accumulate in these sections. An ice cover is formed and it progresses rapidly upstream until it reaches higher gradient by the next upstream rapids. If the flow of ice from above is maintained it will be carried under the ice cover as long as the velocity is above the critical value 0.5-0.6 m sec^{-1} and accumulate beneath it. The buoyance of the water will lift the cover and cause further rise in water level behind the ice jam until the up-stream velocity is reduced below the critical value. When this stage is reached the sludge ice will no longer flow under the cover but will freeze at its upstream edge thereby extending the cover further upstream. In this manner the jam formation proceeds up the rapids. The maximum rise in water level occurs immediately below the section of greatest slope.

ACKNOWLEDGEMENTS.

The author wishes to express his gratitude to Jakob Gislason, general director of the National Energy Authority, for making possible the study of ice in the Thjorsa river system. Members of the staff of the National Energy Authority are acknowledged for their good cooperation. Last but not least sincere acknowledgements are expressed to observators of water level recorders and local observators of ice conditions in Thjorsa for their assistance.

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ICE SYMPOSIUM 1970
REYKJAVIK

BREAK-UP AND CONTROL OF RIVER ICE

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This review paper presents background information on the process of river break-up and control of ice during break-up. The main factors affecting break-up are briefly outlined. As the process of break-up depends on variable weather conditions and on local variations in channel morphology, break-up patterns vary from river to river and from year to year for a given section of river. The limitations and main problems associated with various control methods are summarized. Economic and local site conditions usually determine not only the method but also the extent to which river ice can be controlled. All methods of control have the basic requirement of being able to handle successfully the worst ice conditions likely to occur. Forecasting the different events associated with break-up is thus an important part of river ice control.

This review paper presents background information on the process of break-up and control of ice during break-up. Two major sources were used extensively in its preparation: a recent survey of literature on river ice jams by Bolsenga (1) and the U.S. Corps of Engineers bibliography on snow and ice (2) which contains abstracts of much of the voluminous literature available on these subjects. As most of the references on break-up are case histories, describing conditions at a particular site, only a few selected references have been cited.

Figure 1, showing the average dates of clearing of ice from rivers in the northern hemisphere (3), indicates the extent of the area in which river ice occurs. The zone where the mean monthly air temperature is 0°C during the

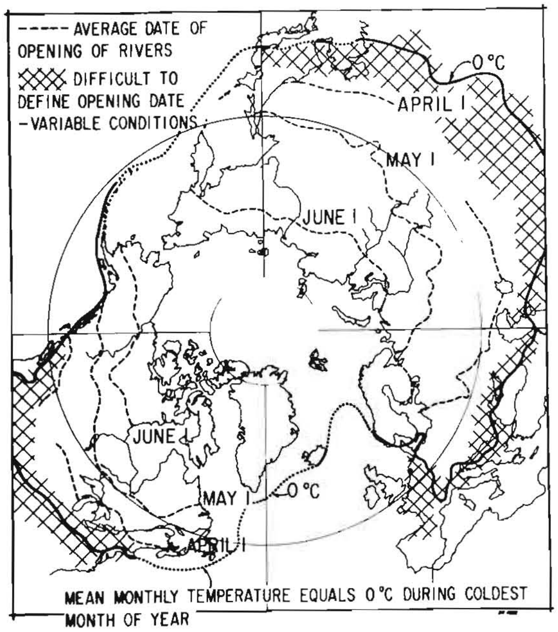

Fig. 1 - Occurrence of river ice in Northern Hemisphere.

coldest month of the year (4) marks the approximate southern limits of ice formation in rivers. The duration of ice on most rivers in this zone is about one month or less, depending on the severity of the winter. North of this zone of unstable ice, rivers are closed for several months of the year because of ice. Ice conditions on these rivers vary tremendously, ranging from massive ice jams extending over several kilometres in the large northward-flowing rivers such as the Mackenzie, (5), to the relatively minor ice formations in rivers such as the Po in Italy, where significant ice has occurred during only a few winters in this century (6). Patterns of break-up of rivers also vary from river to river, and from year to year for a given section of a river.

THE PROCESS OF RIVER BREAK-UP

The term "break-up" refers to a process that extends over a period of days or weeks, beginning when the ice in a river starts to move, break, or deteriorate, and ending when the water is completely free of all ice (7). The term, usually associated with the disappearance of ice in the spring, is used in this paper to include the movement and break-up of unstable river ice during the late fall or winter months.

The process of break-up is complex, depending on weather conditions that prevail during the ice season and on local variations in channel morphology. Figure 2 outlines the general sequence of events that end with the complete disappearance of the ice. The type of ice that forms, the pattern of movement, the

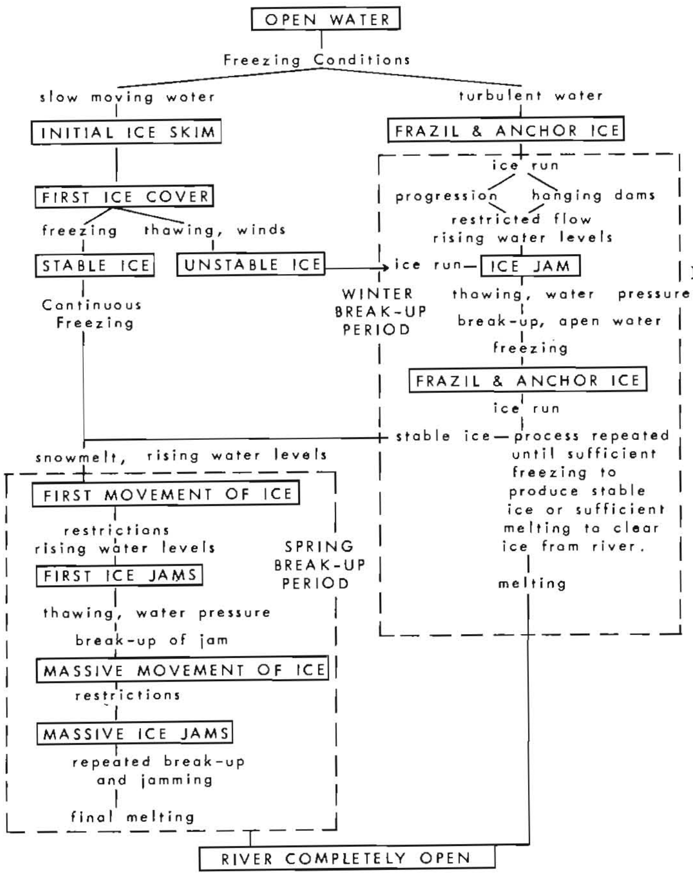

Fig. 2 - General sequence of events during winter and spring break-up periods.

process of jamming, and the type of jams that form, are quite different for winter and spring break-up periods. In both cases, however, serious ice problems occur when drifting ice in a river cannot be transported because of flow restrictions. The drifting ice accumulates and jams, resulting in high water levels, flooding, bank erosion, damage to structures and hazards to shipping and hydro-plant operation.

The first movement of ice can be considered as the beginning of the break-up period. For winter break-up, the movement of ice is more or less a continuous process as long as weather and flow conditions are favourable for frazil and anchor ice production. During spring break-up, the first movement and intensity of break-up of stable ice is determined primarily by snow-melt runoff.

The amount of ice available for movement past a site in a river usually determines the severity of break-up problems. The amount of ice produced during winter break-up depends primarily on the area of open water and on the rate of heat loss from the water surface in ice-producing rapids. Rivers with steep gradients and extensive rapids, exposed to frequent freeze-thaw periods, are subject to severe ice problems because of the vast quantities of frazil produced. Enormous quantities of frazil can also be produced in rivers before a stable cover forms if sudden intense cold weather with high winds occurs after a late fall when much of the river is open. During spring break-up the amount of ice available for movement depends on the thickness and areal extent of the ice covers which are fed to the site from upstream reaches of a river, including tributaries and lakes.

The channel morphology of a river determines whether it can transport drifting ice or whether jams will form. Jams usually start at restrictions to flow caused by bends in meandering rivers, islands, bridges or other structures, and by stable, immobile ice covers. They can also form at the mouths of rivers emptying into shallow, tidal estuaries or ice-covered lakes. In general, the jam is destroyed when the force from the head of water created by the jam is sufficient to overcome the forces resisting downstream movement of the accumulated ice. Once the jam is destroyed, the flood wave will move the masses of ice downstream until either the river is clear of ice or another jam is formed at another restriction.

The size of the jam, degree of deterioration of the ice in the jam, the extent of grounding of the ice, the pattern of drainage channels through the jam, and the configuration of the river channel are factors that determine how readily the jam will break up. The strength of the ice cover is especially important in northward-flowing rivers when southern tributaries choked with ice meet thick solid ice covers that have not yet started to melt. In contrast, the break-up of southward-flowing rivers can be quite gradual without appreciable jamming if ice covers in downstream southern reaches have had a chance to melt or deteriorate before spring runoff begins.

Jams formed from frazil and anchor ice are generally of lower strength than spring break-up jams. Winter jams can, however, consist of high strength ice if they are exposed to severe freezing weather.

Many local factors, too numerous to mention in this paper, are important in the break-up process. Wind and tidal effects have an appreciable influence on the break-up of ice at the mouth of large rivers flowing into the sea. Break-up is affected by watershed conditions, i.e. break-up of rivers in basins with extensive forest cover are considerably delayed because of delayed snow melt when compared with the break-up of rivers in basins with limited forest cover. Ice formation and break-up of rivers in mountains have special characteristics (8). Rivers that freeze to the bottom have unique break-up features. The break-up period on each section of every river is an individual event that can only be described adequately by considering all the factors that affect it during a particular year.

ICE CONTROL DURING BREAK-UP

River ice problems during break-up can be handled in three general ways:

(1) Ice Modification. - Ice covers or jams are melted or broken up by ice dusting, warm water discharge, explosives or ice breakers at locations in a river where break-up problems are known to occur.

(2) River Modification. - The river channel is modified and river control structures are built to control the formation and flow of ice and thus prevent ice problems.

(3) Design and Location. - Structures are so located and designed to withstand ice movements, jams, and flood conditions.

Economic conditions usually determine not only the method but also the extent to which river ice can be controlled. It is often impractical to attempt to control massive ice jams extending over several kilometres using any form of ice modification. In these cases structures must be designed or located to avoid serious ice damage. The complete control of serious ice problems by river modification usually requires extensive engineering investigations and major expenditures.

All methods of ice control have the basic requirement of being able to handle successfully the worst ice conditions likely to occur. Ice modification techniques are sometimes not completely reliable because they cannot be depended upon during years with adverse weather and ice conditions. The success of river modification techniques and the design and location of structures depends primarily on the ability to predict the most serious ice conditions that can occur.

ICE MODIFICATION TECHNIQUES

Control by Ice-Dusting

Early melting or deterioration of the ice cover may be induced at critical sections of a river by applying a thin layer of suitable dust to the ice surface during the early stages of break-up (9). The dust layer increases the melting rate by decreasing the reflectivity of the surface and hence increasing the amount of solar radiation absorbed. Almost any dark material applied in a thin layer on an ice surface will increase the rate of melt under suitable weather conditions.

Air temperature and the amount of sunshine during the early stages of break-up are critical factors in ice-dusting operations. Ice-dusting is not very effective if the daily minimum air temperature falls much below 0°C. If the air temperature is too low, heat losses from the surface by convection, evaporation and long-wave radiation will offset the increased solar energy absorbed by the darkened ice surface. Ice-dusting is most suitable for ice control of rivers in high latitudes as available solar energy can be high during early stages of break-up. Approximate calculations indicate that dusting can increase melting rates up to about 1.0 cm a day in latitudes of 45°N and up to 2 to 3 cm of ice a day in latitudes of 70 to 80°N (10). The thickness of stable ice will range from 50 to 70 cm in the south to 120 cm or greater in the far north. Several days of weather conditions conducive to increasing melt rates are needed to decrease significantly the length of the break-up period or weaken thick, stable ice covers.

As air temperature, snowfall and solar radiation at a site vary greatly from year to year, so will the success of dusting. In southern latitudes weather conditions are frequently completely unsuitable for dusting operations; even at high latitudes weather conditions can be quite unfavourable (10).

Successful dusting operations require detailed studies of past weather records, past ice conditions and dusting techniques. Information is needed on the frequency of new snowfall during potential dusting periods, the availability of suitable dusting material, and the cost of transporting and applying the dust. If potential benefits exceed anticipated expenditures, large-scale trials may have to be carried out before the most effective technique is developed for a site.

Chemicals such as calcium chloride have been combined with dusting to increase the rate of melting and deterioration of ice. As large quantities of salt are needed to melt ice, the success of chemicals may depend more on their ability to melt holes completely through the ice, creating zones of weakness that may expedite the natural process of deterioration. Research is needed on this aspect of ice control as there are few pertinent references in the literature.

Control by Thermal Discharge

The formation of ice in canals and rivers may be retarded or prevented by warm water discharge from thermal heating plants, atomic power plants and other sources. This method of ice control is attractive because extensive stretches of river can be kept completely free of ice. A major difficulty is locating thermal plants so that they can be useful for ice control and, at the same time, meet other more important requirements such as nearness to markets and adequate water supply during the summer months.

The main technical problem in ice control by thermal discharge is to estimate the length of river that can be kept open by a given discharge of waste water at a known temperature. The problem requires a reasonable estimate of heat losses from the anticipated area of open water using weather observations available for the stretch of river under consideration. Dingman, Weeks and Yen (11) have recently outlined a procedure for such estimates. The calculations are made for a steady-state ice-free reach when weather and flow conditions are relatively constant.

Other sources of warm water can be used to prevent or retard the formation of river ice. The success of these methods depends on the amount of heat available for ice prevention. Air bubbling systems, which bring warm subsurface water to the surface by various techniques (12), are especially successful in deep lakes or reservoirs where considerable heat is stored in the deep water under the ice cover. The method has serious limitations in rivers because the temperature of well mixed water under river ice seldom exceeds 0°C by more than a fraction of a degree. Warm water from deep wells (13) and waste water from sewage disposal plants have been used to help control river ice. Specially designed intakes that draw water at different temperatures from deep reservoirs (14) have some potential for ice control at particular sites.

Ice Control by Explosives

Explosives have been used extensively for removal of ice jams. The main requirement is to have sufficient knowledge and capability to place the charges in the right place at the right time. Unfortunately, the effectiveness of various surface explosives (dynamite, thermite, ammonium nitrate compounds, etc.) and the best procedures for placing them are still debatable (1). Explosives are best suited for breaking up relatively small jams where a small amount of explosive placed at key locations will clear a section of river of ice. An open channel and sufficient flow of water are needed to carry away the ice loosened by explosives.

One of the more comprehensive investigations on the use of explosives for river ice control was conducted in the Netherlands by Van Der Kley (15). His calculations, giving the size of charges for different ice thicknesses, indicate that explosives placed immediately beneath the ice cover are most effective. He concluded that explosives tended to make craters in the ice with very few cracks and, consequently, the ice was often not weakened enough to aid removal by ice-breakers. His conclusion that ice-breakers are always preferable to explosives for ice-control is of special interest; if explosives are not very effective in the shallow ice covers encountered in the Netherlands, they would be even less effective in the thick, solid ice covers of more northern areas. This conclusion may not be valid for all locations and problems; recent experiments with explosives in Alaska are somewhat more encouraging (16).

Aerial explosives (bombs, rockets, etc.), including conventional explosives dropped or placed from planes or helicopters, have usually been used on an emergency basis and have not been the subject of extensive investigations. Their effectiveness will depend on adequate preparation and organization based on knowledge of the river and the jams to be destroyed by such action.

Control by Ice Breakers

Ice breakers have been widely used to open up channels in an ice cover before natural break-up begins and to break up ice jams that have formed. Ice breakers are best suited for ice control on large rivers. Frequently, several ice breakers work together in a well-established plan of action. The ice breakers open up a channel by working upstream against ice jams and ice covers. Although ice breakers provide one of the most effective means of controlling river ice, economic considerations limit their use to rivers where serious damage, loss of hydro power or loss of shipping time will occur unless the river ice is controlled.

The many problems associated with the design and operation of ice breakers are beyond the scope of this paper. Waas (17) describes some of the design requirements for shallow draft ice breakers used on the Rhine River; other recent references are reviewed by Bolsenga (1).

Combined Ice Modification Programs

It is often advantageous to combine various techniques for breaking up an ice cover at key locations on a river. For example, explosives combined with ice cutting operations are used to control ice jamming on a section of the Rideau River draining through the City of Ottawa, Canada. Long narrow cuts are made in the ice by power-driven saws a few weeks before break-up starts. Sections of the ice cover between the cuts are blasted loose with explosives and then floated downstream. The operation starts at the mouth of the river and is gradually moved upstream until the section of the river, where jamming previously occurred, has enough open water to transport floating ice over the Rideau Falls into the Ottawa River. Figure 3 is an aerial view showing the start of operations near the mouth of the Rideau. This operation illustrates several basic requirements for ice control: potentially serious economic loss because of jamming, adequate preparation and organization before break-up, a relatively small section of river where the ice needs to be broken up, and a site where broken ice can be transported downstream.

ICE CONTROL BY BOOMS, STRUCTURES AND CHANNEL IMPROVEMENT

Booms

Ice booms are placed as barriers to ice flow in rivers to promote the formation of stable ice covers. If a stable ice cover is formed upstream from

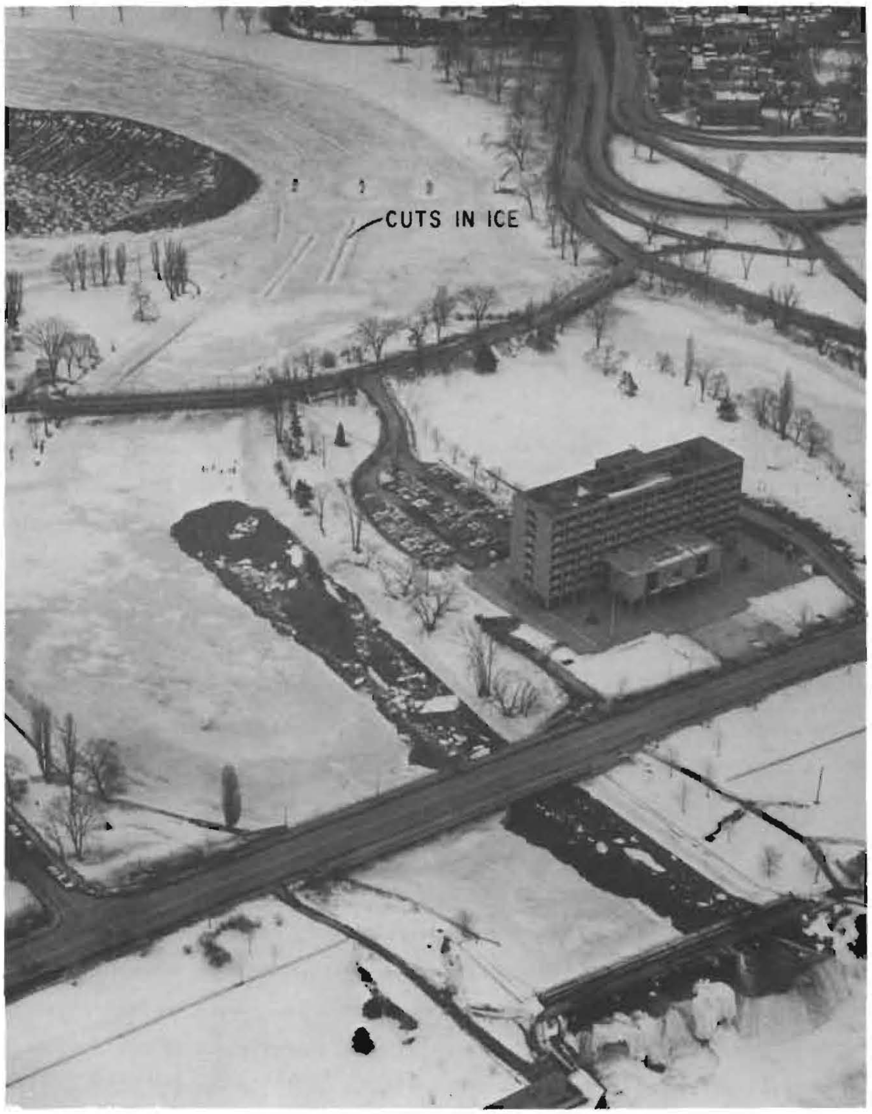

Fig. 3 - Ice control operations, Rideau River, Ottawa, Canada.

the boom, the area of open water is reduced thus diminishing the rate of frazil production. Booms of this type range from log booms and artificial ice bridges or barriers (18) to massive ice control structures such as the one on the St. Lawrence River near Montreal, Quebec (19). Detailed engineering investigations are needed to determine the forces that will act on the boom and to determine the effect of the boom, barrier, or structure on water levels and ice movement.

Booms are also used to stabilize border ice and prevent the movement of lake ice into rivers and diversion channels. One of the most successful booms of this type has been constructed on Lake Erie to prevent funnelling of ice covers into the Niagara River (20).

Ice Control Structures

Ice control structures such as ice diversion dams, submerged weirs and dykes are used to hold ice accumulations or direct them during break-up to locations where the ice can melt in situ (21). The structures present obstacles to the movement of ice at certain predetermined water levels and flows.

A power dam can be considered as an ice control structure although its main purpose is power production. The dam may drown out rapids which previously had produced large quantities of frazil. Water levels and flows can often be controlled by power plant operations to minimize river ice problems or break up ice covers downstream from the dam.

The construction of a power dam does not always solve frazil problems. In some run-of-the-river power plants it is difficult to form a stable ice cover upstream from the dam and frazil problems occur every year. Frazil can still be produced in large reservoirs where a stable cover normally forms, if strong winds, combined with low air temperatures, occur at the time when a stable ice cover is just starting to form. This hazard from frazil, not always recognized because of infrequent occurrence, has resulted in power plant shutdowns for short periods of time.

Channel Improvements

Channel modifications that improve the flow of ice, or cause stable ice to form at chosen locations, provide a reliable, effective means of river ice control (22). Modifications include removing restrictions, deepening the channel, widening the river, stabilizing banks, straightening out bends and relocating bridge piers and structures. Hydraulic models are useful for determining the effect of proposed improvements on the flow of ice. The cost of extensive channel improvements can often be prohibitive on large rivers with severe ice problems.

THE ROLE OF FORECASTING IN BREAK-UP CONTROL

Efficient planning and design of ice control projects for hydro-plant operations, construction, and navigation require forecasts of ice movement, height of water levels, occurrence of ice jams, and the magnitude of the forces associated with ice movement (23). The success of various ice control methods often depends on the ability to forecast accurately the different events associated with the break-up of river ice.

The seasonal forecast of maximum water levels during break-up is one example of a basic forecast problem. The forecast of water levels during spring break-up is part of the more general problem of forecasting snow melt runoff, which requires: snow survey data from the watershed, snow melt equations relating weather parameters to rates of snow melt, and flood routing techniques to translate the snow melt forecasts into flow forecasts. Forecasting water levels during winter break-up requires information on the rate of ice production and on the behaviour of ice downstream from ice-producing rapids, i.e. whether ice will progress upstream to form a stable cover or move further downstream to form hanging dams or jams at restrictions (24). For both winter and spring break-up periods the usefulness of water level forecasts depends on the accuracy of short-term weather forecasts.

Forecasts of the probable occurrence of ice in rivers, occurrence of ice jams and probable date of clearing of the ice are especially useful in regions of unstable ice where serious ice problems do not occur every year. The probabilities of an ice-free winter and probabilities of obstructions by ice are an essential part of ice control for shipping in the navigable rivers of the Netherlands and Germany (25). The events associated with spring break-up tend to be more predictable on rivers where a stable ice cover forms each year, but even on these rivers the frequency and dates of occurrence of serious jams are often required.

In designing ice control structures or structures exposed to ice action during break-up, an estimate is needed of the worst ice conditions likely to be encountered. Most of the failures of structures because of ice could have been prevented if designers had been able to anticipate the severe conditions that caused failure. A classical example of a failure due to ice was the collapse of a bridge across the Niagara River (26) The collapse was caused by massive jams that created enormous lateral pressure on the bridge piers. The problem of designing for the worst ice conditions is similar to, but much more difficult than, the problem of predicting the so-called "100-yr. flood" for spillway design. In spillway design, past records of peak flood flows are usually available for statistical analysis. In designing for severe ice conditions adequate past records are often not available and designers must rely on information from local observers and on studies of the effects of previous high water levels and ice movements on rivers, banks and vegetation.

CONCLUDING REMARKS

The process of break-up of river ice depends so much on local site conditions that it is difficult to apply control methods and forecast techniques developed for a particular site to other locations. Several years of field observations of ice behaviour for different weather and flow conditions are usually needed for each stretch of river to develop reliable control methods. In the past, solutions to river control problems have often been necessarily empirical without developing a solid scientific basis for the advancement of ice engineering practice. It is only within the last few years that a more scientific approach has been taken in the design of ice breakers, the use of explosives on ice, the theory of ice movement and ice jam formation, and the many other problems associated with the break-up and control of river ice.

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DISCUSSION by G. FRANKENSTEIN

on paper by G. P. WILLIAMS (3rd Session)

I disagree with Mr. Williams that the strength and other properties of the ice are important or have to be known before effective blasting can be accomplished. Our tests were conducted on very strong clear ice of Northern Alaska and the weak river ice, which is typical during the breakup period, of New England and Minnesota. The results were approximately the same and are discussed in our paper (co-author, N. Smith).

AUTHOR'S REPLY

It is surprising that the strength of ice is not of importance in determining the amount of explosive to be used. I do not think that this point has been made in the literature I reviewed on the subject. The results of Mr. Frankenstein and Mr. Smith investigations are most useful for engineers who have to use explosives for clearing ice from rivers.

ICE SYMPOSIUM 1970
REYKJAVIK

THE USE OF EXPLOSIVES IN REMOVING ICE JAMS

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Synopsis

A brief history of the use of explosives for ice jam removal is discussed. Ammonium nitrate mixed with fuel oil is considered the best explosive for ice jam control because of its cost and safety features. For maximum effect, the charge should be placed in the water below the ice. A curve is included which gives maximum crater hole diameter as a function of the cube root of the charge weight.

Introduction

Ice jams have been a problem to man for hundreds of years. One of our earliest forms of transportation was by water, which encouraged settlers to locate as close to river banks as possible. During the spring ice breakup periods, the discharge (or flow) of many rivers was restricted by ice clogging in the bends and shallows to form an ice jam. The net result was a flooded settlement. Today man settles along the river for esthetic reasons and is still subject to spring floods caused by ice jams.

There is no general or universal method for preventing ice jams. Each river, even in the same geographic area, has its own flow characteristics so a preventative measure which works on one stream may not be effective on another.

One of the many methods used (extensively in Alaska) to prevent ice jams is to dust the ice sheet with heat-absorbing materials such as coal dust to weaken the ice sheet upstream from a potential jam area. Ice diversion structures have been constructed in Canada and the United States to force the floating ice to accumulate in selected areas where overbank flow will cause little damage. Channel improvement, i.e. eliminating sharp bends, sand bars, or large boulders, is another preventative measure. When jamming does occur, explosives are used to eliminate the jam and release the flood waters. Damages from spring floods are minimized in many areas by moving residents and transportable belongings to higher ground.

Discussion

The first record of the use of explosives to remove an ice jam was in 1758 (1). This first experiment consisted of exploding bombs placed under the ice. Since then explosives have been used regularly for ice jam removal. The most common explosive used has always been 40% dynamite and the usual practice was to place two to twelve sticks within the ice jam. The results of this type of blasting were not very effective.

Thermit, a high-heat release powder, was used extensively by Barnes (2) in his original ice jam work prior to 1940. Thermit, not really considered an explosive, is composed of aluminum powder and iron oxide and is used in incendiary bombs.

Until a few years ago ammonium nitrate was known only as a fertilizer and an ingredient in blasting agents. During World War II it was mixed with trinitrotoluene (TNT) to form a much-used military explosive. About ten years ago it was discovered that if a finely divided carbon fuel was added to prilled ammonium nitrate, packed tightly and confined, the mixture could be detonated. Further testing found that 6% by weight of diesel oil mixed with bulk ammonium nitrate (or 1 U.S. gal of fuel per 100 lb of ammonium nitrate) was an ideal explosive when detonated with a powerful booster charge such as TNT or a stick of 40% dynamite (3). It is not only cheap but also very safe to handle and is now commonly referred to as ANFO (Ammonium Nitrate Fuel Oil). The first users of ANFO mixed the ammonium nitrate and fuel oil in the field. Ammonium nitrate, which is readily available in a granular, crystalline, or prilled form when mixed with Number 2 fuel oil should stand for a minimum of 24 hours to assure maximum effect. It is important to keep the mixture dry, and for a more uniform mix the prilled ammonium nitrate is better than the fertilizer grade. Today it is possible to purchase ANFO pre-mixed and packaged in plastic bags of various weights.

During the winter of 1965-66 the U.S. Army Corps of Engineers conducted a series of tests designed to determine the optimum placement depth of an explosive to yield the maximum crater hole diameter and maximum cracking in sheet ice. Three different explosives were used, namely Military C-4, ANFO, and TNT.

"Operation Peggy" (4) was conducted by the Alaska District, Corps of Engineers, on Peggy Lake near Anchorage, Alaska. The objective of the tests was to determine the optimum placement depth to give maximum cracking of the ice sheet. The explosive used was ANFO placed in plastic bags inside used oil drums.

The project was considered a success even though the crack data were not as good as anticipated.

"Operation Breakup" (5) was conducted by the Corps of Engineers with cooperation from USACRREL and the Alaska Fish and Game Division. The objectives of these tests were to determine the optimum placement depths to give maximum crater radius, the effect on the fish population when explosives are detonated under the ice, and the optimum horizontal distance that row charges should be placed to give a continuous failure hole. Figures 1 and 2 show detonations of single and row charges. The project was conducted on Blair Lake, south of Fairbanks, Alaska. The explosive used was Military C-4. The project was very successful and the results have been directly applied to the elimination of ice jams on many rivers in the United States.

The results from the project clearly indicate that the charge should be placed under the ice. For an ice thickness of 36 in. and a semi-infinite water depth a 130-pound charge should be placed 10.0 ft below the bottom of the sheet. This can be expressed by

$$h = 1.98\ W^{1/3} \qquad \text{Eq. 1}$$

where h = placement depth in feet below the ice sheet and W is the charge weight in pounds.

The first application of the test results of Project Peggy and Operation Breakup was an ice jam on the Upper Mississippi River in April of 1966. A typical river ice jam is shown in Figure 3. It was decided to try various explosives to determine their usefulness in eliminating the ice jam. It was found that ANFO, TNT, and dynamite produced approximately the same crater diameter for equal charge weight when placed at the same distance below the ice surface. It was also determined that Thermit was not really useful in ice jam elimination. Because of its low cost per pound and low detonation velocity (12,000 ft/sec), ANFO was recommended for ice jam elimination.

Whenever possible the placement depth, charge weight, and crater radius were recorded. Figure 4 (6) is a plot of the crater hole diameter in feet as a function of the cube root of the charge weight in pounds. The data are tabulated

Figure 1. A single charge.

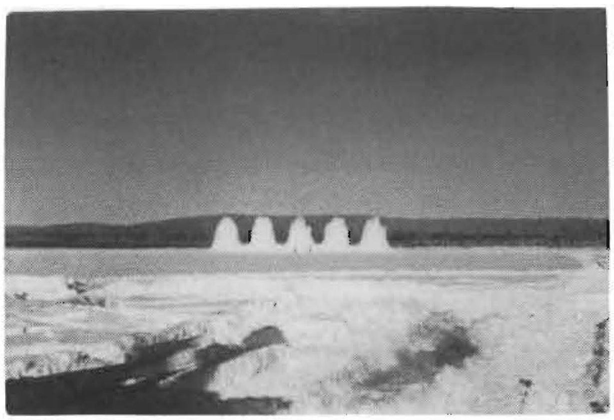

Figure 2. A row charge.

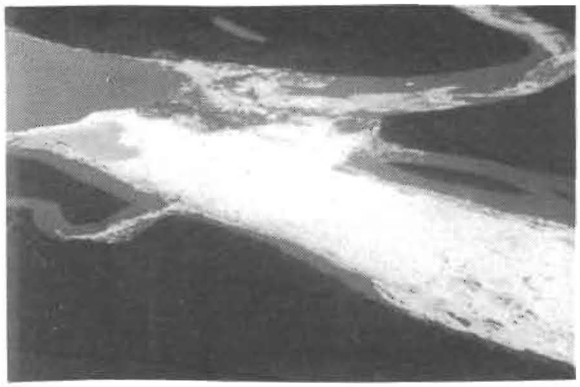

Figure 3. A typical ice jam.

TABLE I

Charge Weights and Crater Diameter Measurements

Charge Wt. (W) pounds	$W^{1/3}$	Crater Diameter (D) feet
940	9.79	167
133	5.10	70
48	3.64	42
32	3.18	32
7	1.91	9*

* Estimated from project film.

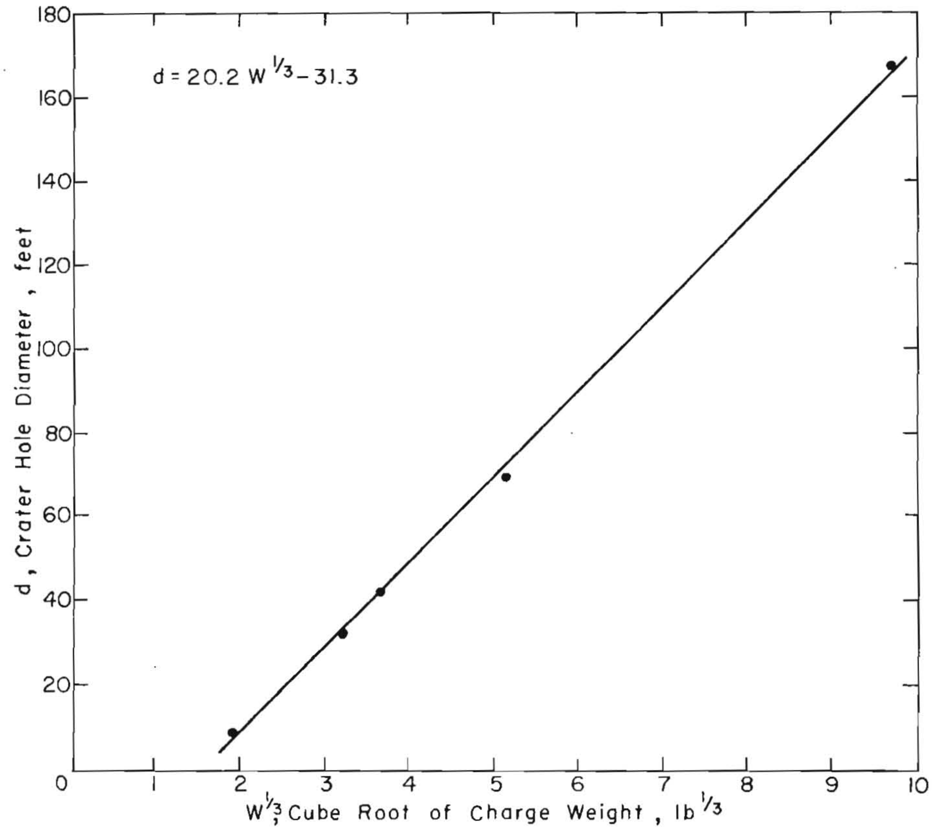

Figure 4. The crater hole diameter versus the cube root of the charge weight.

in Table I. The charges are placed at optimum depth below an undisturbed ice sheet. It is interesting to note the linearity and consistency of the results. The data cover a range of ice thicknesses from 6 to 40 in. Figure 4 is now used universally in the United States, including Alaska, for determining the charge weight for the desired crater hole size.

During the spring breakup period of 1970 the U. S. Army Corps of Engineers was asked to assist a number of communities in eliminating ice jams. In a number of areas the decision was made to use the explosive technique. It was quickly determined that the most efficient method was to place between 10 and 20 charges to detonate simultaneously. The desirable charge spacing is one hole diameter as determined from Figure 4. More than a mile of river ice can be broken up in an 8-hour period using this method.

For maximum effect the charges should be placed in the water below the ice and never on the surface. To keep the charges dry the recommended procedure is to transfer the ANFO from its original container into strong plastic bags (Fig.5). If more than one bag is to be used for each charge these should either be tied together or placed in another container such as a burlap bag (Fig. 6). The charges should be weighted so that the water current will not alter their placed position as determined from equation 1. In sheet ice the placement hole can be easily drilled (Fig. 7) or cut with a power chain saw. In broken up ice one can usually find an opening to the water by probing with an ice chisel or long bar. When one or more charges are to be detonated simultaneously they should be joined with primacord.

If desired one can use delay fuses when setting off more than one charge; however, the authors feel that this method is really not necessary. Either electric or fuse caps can be used depending on local restrictions.

Conclusion

Explosives can be effective in eliminating ice jams if used correctly. The ammonium nitrate and fuel oil mixture (ANFO) is preferred for this application. The two important parameters are the placement depth and the correct charge weight. Explosives can be safe to use if all of the proper safety precautions are taken.

Figure 5. Transfer of the ANFO from its original container to a strong plastic bag.

Figure 6. Placing multiple charges into a burlap bag.

Figure 7. Drilling a placement hole with a power auger.

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THE BURFELL PROJECT
A CASE STUDY OF SYSTEM DESIGN FOR ICE CONDITIONS

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SYNOPSIS

The design of the Burfell Hydroelectric Project on the Thjorsa River in Southern Iceland was greatly influenced by the great quantities of slush ice carried down the river each winter. The approach to the evaluation of the problem is described, as well as the methods used to solve it. One winter of operating experience confirms the soundness of the solution.

THE THJORSA RIVER

In the early fifties investigations were started by the State Electricity Authority toward harnessing the Thjorsa River for hydroelectric power. The Thjorsa River is the biggest river in Iceland and has by far the greatest hydroelectric power potential. All the appraisals of the potential sites pointed toward the Burfell Project as being the cheapest for initial development. At this site water from about 85% of the entire watershed could be developed and this plant could develop considerably higher head than any other development conceived in the initial appraisals, except one. It was, therefore, no wonder that all attention was focused on this site as the best and cheapest initial development in the Thjorsa River Basin. The consulting engineering firm, Harza Engineering Company International, was engaged to study the feasibility and later design this project.

The drainage area at Burfell is 6350 km^2 and the average flow 338 m^3/sec. Due to the stabilizing effects of the groundwater in the extensive lava fields in the basin,the discharge is rather constant, with a minimum flow on record of 70 m^3/sec and a maximum flow of 2000 m^3/sec. The river channel is relatively wide, shallow and steep, especially where the river flows on top of the lava fields.

The Thjorsa River has for centuries been notorious for the great amounts of ice carried down the river each winter. In several places huge quantities of ice pile up. For example, below the Urridafoss Waterfall, only 10 km from the ocean, the ice pile can reach a height of 18 meters,and both at Urridafoss Waterfall and below the Burfell Mountain the ice piles amount to tens of millions of cubic meters each year.

Mr. Sigurjón Rist, state hydrologist, will in a separate paper describe the ice formations in the Icelandic rivers(1), so here only a brief description will be given. The most troublesome ice formation is the slush ice, but other types of ice conditions are also present, such as drifting sheet ice, anchor ice, and ice bridges and jams. When ice jams break, a flood wave goes down the river, carrying with it large amounts of ice. This is called step-burst. Although these ice conditions are important, the slush ice problem completely overshadows them because of the tremendous volume of material involved. It has been estimated that up to 15 tons per second of ice may be carried down the river. This would amount to between 40 and 50 m^3/sec of the loosely packed slush ice. During step-bursts these quantities can be even greater, and step-bursts also carry large amounts of broken sheet ice and bank ice. The worst ice producing periods are associated with the lowest discharges.

This suffices to show that any power development on the Thjorsa River had to take the ice problem into account in one way or another.

DESIGN CRITERIA

The ideal solution to an ice problem as the one described above is to build large reservoirs where all the ice can be stored during the winter. The topography of the project site does not, however, allow any such reservoir construction. The only reservoir that could have been constructed economically would have been too small to store all the ice. Such a reservoir would fill very quickly, and after that,water and ice would have to find its way in one place or another over the spillway and/or into the waterways of the power plant. A small reservoir is, therefore, only a temporary solution. For economical reasons the Burfell Project was designed as a run-of-river project with very little reservoir capacity. For those types of developments, there are only two possible solutions to the ice problem:

1. Separate the ice from the water, flush the ice on downstream,and divert the water to the power intake.

2. Divert both slush ice and water to the power station and allow the ice to go through the turbines along with the water.

The primary design criteria of the diversion features selected for the Burfell Project was to flush all the ice over the dam and down the river. By flushing the ice, considerable water will be wasted at times when water is most needed. From this standpoint it would be preferable to let the ice-water mixture go through the turbines and use the ice in that way for power production. This has been done successfully at several small power plants in Iceland. This operating procedure would, however, involve the risk of complete blockage of the waterways by ice, putting the power station out of operation for considerable periods of time. Therefore, it was not considered feasible to depend on this operating procedure, although the design of the waterways is such that it will be possible to employ it. The primary solution will be, as stated before, to flush the ice over the dam and divert ice-free water to the power station. In addition, several measures will be taken to reduce the ice production upstream from the dam.

THE BURFELL PROJECT

At this place it is appropriate to give a short description of the project. The layout of the Burfell Project is shown on figure 1. A low, 370 m long concrete diversion weir in the river channel diverts water through a canal with a gated control structure into Bjarnalon Pond, a small pond which will be utilized for daily ponding. From Bjarnalon the water flows through a canal and a 10 x 10

Figure 1. General layout of the 210 MW Burfell Hydroelectric Project.

meter horseshoe-shaped tunnel driven through basaltic rock. A concrete lined bifurcation connects the power tunnel with the two penstocks which deliver the water to six 51600 HP Francis turbines. From the turbines the water flows through a short tailrace canal into the Fossa River which joins the Thjorsa River about 2 km downstream. Two interconnected surge tanks straddle the upstream end of the penstocks and act as gate houses for the penstock emergency gates. The design calls for the installation of three 35 MW units initially, with provisions to add three additional units, bringing the total capacity to 210 MW. The project is designed for run-of-river base load operation with a net head of 115 m and station flow of 225 m^3/sec.

EVALUATION OF THE ICE PROBLEM

As described above, nature and economics formed a framework within which an engineering solution had to be found. Furthermore, the element of time made it necessary to produce estimates for design purposes long before any results from scientific studies could be expected.

The elements required for the engineering solution can be enumerated as follows:

1. Collection of qualitative and quantitative information about the ice formations in the river.

2. Evaluation of the magnitude of the ice problem from the information already available and a quantitative estimate of its influence on the operation.

3. Analytical evaluation of the various types of operational conditions that might be encountered.

4. Verification of these solutions by all means possible, such as hydraulic model tests, observations in nature, and analytical studies.

5. Development of instruments for the quantitative collection of data and to give continuous information about the condition of the river during operation of the diversion features.

6. Establishing design criteria for the structures of the diversion features.

7. Developing operational procedures based on the hydraulic model tests and analytical evaluation.

The first field study especially for the study of the ice conditions in the Thjorsa River at Burfell was started in February 1963. Two men were stationed at Burfell making daily observations of the meterological and hydrological conditions. Estimates were made of the quantity of ice discharge and observations of its character.[2] The hydraulic model studies were later based to a large extent on these very limited observations.

After this first experimental field study two well-known ice specialists were consulted and they conducted field surveys here during the next two winters.[3] These studies involved general observation of ice formations and collection of meterological data, as well as important calorimetric measurements for evaluating heat losses from the river. This evaluation of heat losses has since been expanded and is described in a separate paper by Freysteinsson.[4] However, no quantitative observations or estimates of the ice discharge were made.

From the beginning it was evident that instrumentation for the measurement of ice discharge was totally lacking. As early as 1962 I suggested investigating several ways to measure the ice concentration, especially the possibility of using resistance meters similar to the ones used in coastal engineering for measuring wave hights. At that time it was not considered practical to start this development work, but two years later the State Electricity Authority engaged the firm Rafagnataekni for the purpose of developing the ice discharge meter. After the National Power Company was established this work has been carried out in cooperation between these two agencies.

The instruments developed are described in a separate paper.[5] They consist of an ice discharge meter, an ice thickness meter, and a step-burst indicator. All these instruments are very valuable to aid in the operation of the plant in addition to being of scientific importance in collecting basic data. The developing of these instruments has taken a lot of time and effort, but I believe that, with exception of the step-burst meter, most of the troubles have now been ironed out and these instruments can be considered as fully developed.

No quantitative appraisal of the influence of the ice on the operational characteristics of the Power Plant had been given in the Project Planning Report. A dispute arose over the feasibility of the project due to the danger caused by the ice and the magnitude of the ice problem. At this time it was of utmost importance to analyze quantitatively the importance of the ice on the operation of the Power Plant. No precedence was known for this type of evaluation. However, it was assumed that it would be possible to flush the ice over the dam and that the amount of flushing water would be directly proportional to the amount of ice to be flushed. The amount of ice in the river was computed, based on the equations for heat loss developed by Devik.[6] No meterological information was at that time available from the upper part of the Thjorsa River Basin and simple assumptions were made to transfer and use meterological data from nearby stations. Very little was known about the size of the open water surface area producing ice, but it was assumed constant.

Although the evaluation of the influence of ice on the operation was based on a number of questionable assumptions it showed first of all that several days each year one could expect very severe ice conditions during which most of the available water would have to be used for flushing purposes. Secondly, it showed that, provided the ice could be separated from the water and flushed on downstream, the overall reduction of the energy production due to ice flushing was not too serious.

DESIGNING OF THE DIVERSION FEATURES

To solve the problem of arranging the diversion features in such a way that they could efficiently perform its function of separating ice from water and flush it downstream, the Hydraulic Laboratory of Trondheim, Norway, was engaged. The laboratory built three models, the biggest one about 650 m^2, to investigate the diversion features with respect to handling water, ice, and sediment. Sand was used as bed-load and plastic shavings as ice. No precedence was available to guide the modeling technique for the ice, but the behaviour of the plastic in the model was very encouraging.

It was soon evident that it was not easy to find an ideal design. One design after another was tried and gradually a design developed that could handle the ice fairly efficiently. From the model studies it appeared that the following factors were of primary importance for an effective design:

1. By maintaining reasonably high velocities and gradual changes in the cross section, mixing of ice and water is prevented at the same time as the ice is kept in steady motion with minimum deceleration towards the intake.

   To fulfill this requirement the water level at the intake had to be kept within rather narrow limits.

2. To minimize the water requirements for flushing ice, it is necessary to concentrate and guide the flow along the inlet wall.

3. Stagnation zones are dangerous as ice accumulation can easily start there and build up.

4. Shallow depths should be maintained upstream from the gated weir. With an ice carpet on the surface and a considerable depth in front

of the weir, there is a definite tendency to draw clear water from below the ice carpet over the weir.

A description of the model is given in a paper by Carstens.[7]

The adopted design of the diversion structures is shown schematically on figures 2 and 3. The flow is guided towards the inlet, and in a direction approximately parallel to it, by a rock jetty. In front of the inlet, a depression is excavated in the river bottom. This depression has gentle slopes to avoid any sudden changes in cross section and the resulting turbulence and mixing of ice and water. The depression slopes towards the inlet, which has the threefold function of passing ice, water, and bed-load. At the top level there is a long overfall into a trough or ice sluice which opens into Bjarnalaekur Canal. On the second level there are six, 5 x 10 m, openings for passing water into the diversion canal. The lowest level opens into three undersluices which serve the dual purpose of passing bed-load and providing slugging water for the Bjarnalaekur Canal, should ice begin to accumulate there.

Ice can also be passed over the gated weir. In this case the ice would go into the side runway channel. A gate connects the side runway channel to the Bjarnalaekur Canal. By closing this gate, the ice and flushing water will go down the river channel, but otherwise it will go into Bjarnalaekur Canal that discharges into the Bjarnalaekur Creek, a steep gradient creek, that joins the Thjorsa River about 8 km downstream from the dam.

Considerable water is required to pass the ice over the dam. From time to time this will result in shortage of water for power production. Therefore, it is of great importance to be able to utilize the available water as effectively as possible. To accomplish this, a gated control structure is constructed in the diversion canal. By manoeuvring the gates in this control structure, it is possible to control the diversion of water from the river, independently of the power production. In this way, the amount of water diverted, and the water level at the inlet, can be kept constant while the station is operated with fluctuating water level in the Bjarnalon Pond.

OPERATIONAL EXPERIENCE

The Burfell station has only been operated for one winter, and then at less than ½ the design capacity. Last winter was milder than average with no extremely severe ice producing weather. The river discharge was also somewhat below average. The experience from last winter is, therefore, rather limited and generally characterized by relatively small amounts of ice and abundant flushing water.

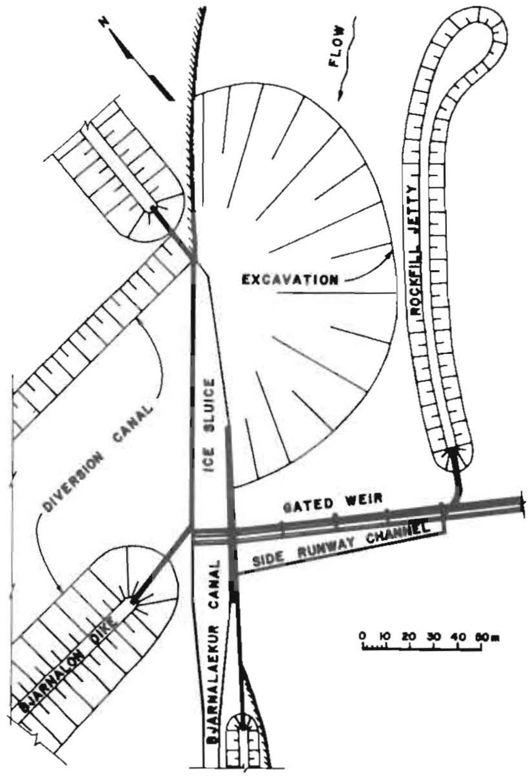

Figure 2. Schematic drawing of the diversion structures.

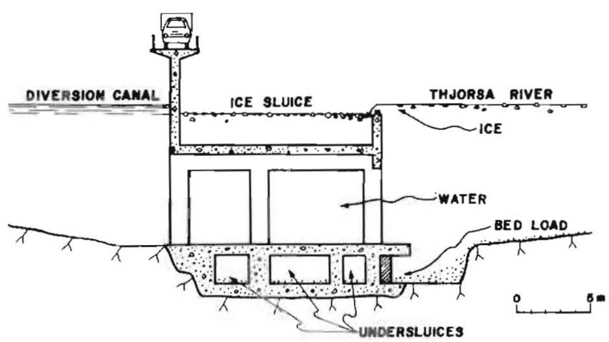

Figure 3. Section through the diversion inlet.

As the station picks up load the operation will have to change in the sense of being more consious of conserving water. This will be necessary even though release from upstream storage will become available to firm up the winter flows. Nevertheless, the experience from last winter has been very interesting and educational with respect to the future operation of the Project.

The design of the diversion features was the outgrowth of extensive model tests carried out in the Hydraulic Laboratory of Trondheim, Norway. These tests showed that for the operation the water level and thus the approach velocities to the inlet should be kept within a rather narrow range, depending primarily on the total flow in the river. If the water level was lowered beyond this limit the turbulence at the edge of the depression in front of the inlet would carry into suspension a portion of the ice and carry it into the diversion canal. If on the other hand the water level was too high an ice bridge would form upstream from the inlet.

For parts of this past winter the approach flow to the inlet was through a narrow gap in the rock jetty. The flow through this gap entered the depression at a high elevation and with considerable velocities. Therefore, the water level in front of the intake could not be kept high enough to secure tranquile flow. Nevertheless, no significant amounts of ice were carried into the diversion canal. Unfortunately, the structures limit the amount, the pool level can be drawn down during normal flow conditions, which means that high velocity approach can normally not be achieved.

The upper limit of water level was also found to be not as critical in the prototype as in the model. Figure 4 shows the passing of heavy ice concentrations during step burst. Please note how a shear zone developes between the ice in motion and the stationery ice. In the model, the granular plastic shavings interlocked and there was no possibility of a shear zone being formed. Therefore, an ice bridge was always inevitable in the model if the approaching ice began to thicken due to decelerating approach velocities. In the prototype on the other hand this shear zone developed, allowing the ice to move as a thick carpet towards the inlet. It was, therefore, possible to let the approaching ice thicken considerably and pack together into slush ice flocs although that created the danger of possible ice bridge formation upstream from the inlet. Any disturbance of this precarious balance could start an ice bridge and there was also the danger that the ice floes would thicken so much due to the decelerating velocities that they would strand on one of the protruding parts of the ice skimming weir.

The ice skimming weir is a low overflow weir placed on top of the apron wall

Fig. 4 Heavy ice concentration during step-burst. Note the shear zones between moving and stationary ice.

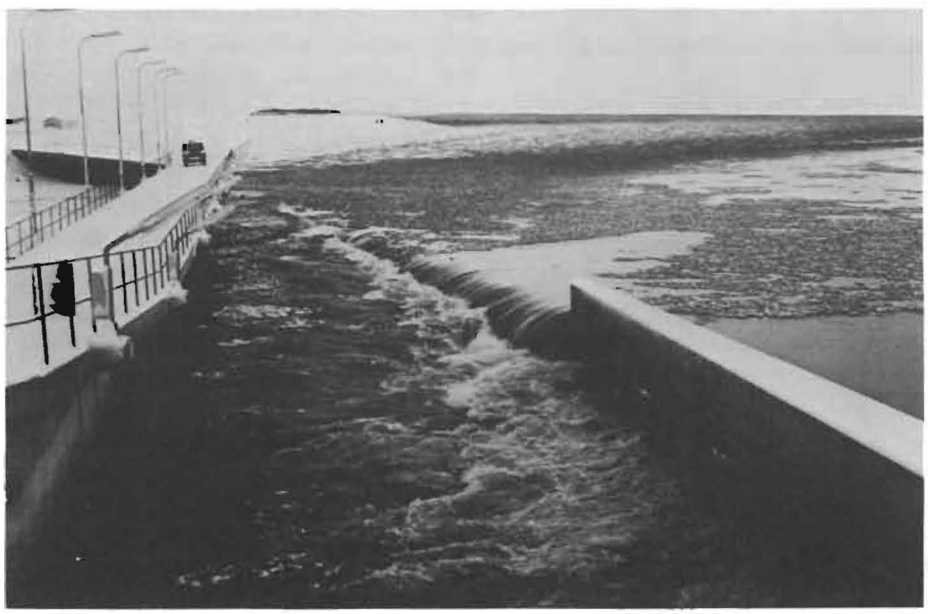

Fig. 5 Normal winter operation

in front of the ice sluice. It is made of 16 sections, each 4 m long. This weir was defective from the beginning and several of the sections fell down while it was impossible to move the others. Last winter all the ice had to be passed over this sawtooth shaped overfall.

The operation last winter can be divided into three periods of rather distinct type of operation. During the initial period the diversion features were operated according to the recommendations of the model studies keeping the water level in front of the inlet relatively low and constant. Figure 5 shows this normal type of operation. This type of operation was carried out successfully until on the evening of November 10 when an ice bridge formed in front of the inlet. This bridge grew into a substantial ice accumulation that filled the river channel for about 1 km upstream. This ice pile up lasted for about 6 hours when a step burst with increased water caused it to start moving and cleared most of it away, leaving only a small ice bridge.

The second phase of the operation started with the collapse of a section of the Bjarnalaekur Canal. The Bjarnalaekur Canal is the principal way of flushing out the ice. For most of its length it is blasted out in solid lava, but in the lowermost reach the canal bottom goes through the lava and into easily erodible materials. During initial testing of the canal, several sections of lava had been undermined and collapsed into the canal partially blocking it. Some of these had been blasted away while others had been left in place forming a low rock dam in the lower reaches of the canal. Sediment had collected in front of this rock dam protecting the easily erodible underlayer. When the amount of flushing water had to be increased due to the ice conditions at the inlet the protective sediment layer was eroded and big blocks of rock fell into the canal practically closing it off. Ice started to accumulate in the canal until by the evening of November 11 it was practically full and ice even beginning to accumulate in the ice sluice above the dam. Attempts to clear it out by increasing the flushing water proved unsuccessful as the difference in water level across the dam was relatively small. In a final attempt all flushing water was cut off and the water level above the inlet allowed to increase somewhat. This made the ice pile in the canal collapse into the ice tunnel. Then both bottom sluices and flap gates were opened and the ice pile in the canal slowly moved down and out of the canal.

The raising of the water level in front of the intake was not without consequences. During that time the ice had accumulated upstream from the ice bridge and all attempts to clear it proved unsuccessful. A big step burst also added to the ice concentration. The ice pile gradually extended further and further upstream forcing the water out of the river channel and up on the left bank and from there over the ungated weir. Up to this time all the water had come to the inlet. Now, however, 75% of the flow was lost over the ungated weir. To increase the flow to the station a small cut was made in the rock jetty. This cut gradually eroded through the jetty and the ice pile between the jetty and the inlet so by midnight on the 12 an open channel had formed all the way to the inlet. (figure 6). As mentioned before the water entered the depression in front of the inlet at a high level and with high velocities and great turbulence even with the highest possible water level in the pond. (figure 6). This flow condition was, therefore, very stable against ice bridge formation and very efficient for passing the slush ice. The disadvantage of this approach condition was, however, shown on January 1 when a big step burst carrying great floes of solid ice came down the river. The big blocks of ice rammed right against the ice skimming weir and finally stopped and filled all the approach channel. This condition prevailed for some period of time or until a period of mild weather melted an approach channel in the ice in the river bed.

The third period of operation started on February 1 when freezing weather sat in again. During this period the approach condition was essentially normal except for the opening in the rock jetty which created an unfavourable cross current in the approach flow. The mode of operation during this period was, however, different from that of the 1st period. In normal operation the water level is kept constant at the level corresponding to the total flow in the river while the amount of flushing water is varied according to the ice discharge in the river. This is the type of operation recommended by the Laboratory and the one that should result in the least amount of water for flushing operation, and, therefore, the best water utilization.

During last winter the water was abundant for the requirements of the station and for ice flushing. The conservation of water was, therefore, not of primary importance and a different mode of operation was tried i.e. the water level was kept at all times as high as possible compatible with the ice discharge in the river and the water requirements of the station. The theory was that when the ice discharge increased suddenly the water level would be brought down to the recommended value and the added water from storage would then be available. Although the theory is basically sound, two things made it very difficult in practice. The first is hydraulic the second human.

The hydraulic factor can be described as follows: During the winter both ice and sand is brought into the diversion canal. The ice accumulates on the surface and the sand on the bottom leaving only a tunnel of a size determined by the average discharge to the station. If the water level is suddenly decreased,

Fig. 6 Flow approaching through the rock jetty. Note the turbulent approach condition and the chunk of slush ice sitting on the ice skimming weir. Photo taken Nov. 13, 1970.

Fig. 7 Ice bridge about to form in front of the inlet.

the water pressure or buyancy supporting the ice is decreased with the danger of a collapse that might reduce substantially the area of the tunnel and thus increase its frictional resistance. After operating with a lower water level for some time, the water will of course erode a new equilibrium tunnel in the ice and sand but until then a substantial head difference will be experienced between the inlet and the Bjarnalaekur Pond. This hydraulic condition makes it rather risky to lower the pool in front of the inlet suddenly.

The human element is equally important. Every station operator feels a sense of security with the reservoir full to capacity. This volume is very valuable in case of an emergency. There is, therefore, a natural reluctance to lower the water level more than absolutely necessary.

The results from this third mode of operation are not favourable. Again and again ice bridges formed because the water level was not lowered fast enough. These bridges were of minor significance and could be cleared away easily after a few days when the ice in the river decreased again. Nevertheless, they are annoying and unnecessary and can and did result in one big ice accumulation upstream. Figures 7 and 8 show the forming of one such ice bridge. Such an ice bridge is removed by raising the water level, causing the bridge to float up and then open all the main spillway gates.

The ice forces and the frequest movements of gates has caused considerable damage to the gates. Figure 9 shows a closeup of the ice skimming weir which has taken the heaviest beating.

CONCLUSIONS

The operation of the Burfell Project ice passing facilities has shown the basic soundness and flexibility of the design. Operational experience has been obtained to deal with various forms of ice conditions in the river. In this paper these various ice conditions and the difficulties they caused has been emphasized. Nevertheless, these troubles have caused very little disturbance in the operation of the power plant, and the reduction in discharge to the station was only a fraction of what had been predicted theoretically. The operation has verified the use of modelling techniques to study sludge ice movement. No effect from undercooled water has been noticed and no effects of freezing is noticeable with respect to the hydraulic characteristics of the sludge ice.

Fig. 8 Ice bridge fully developed. Picture taken 6 hours later than figure 7.

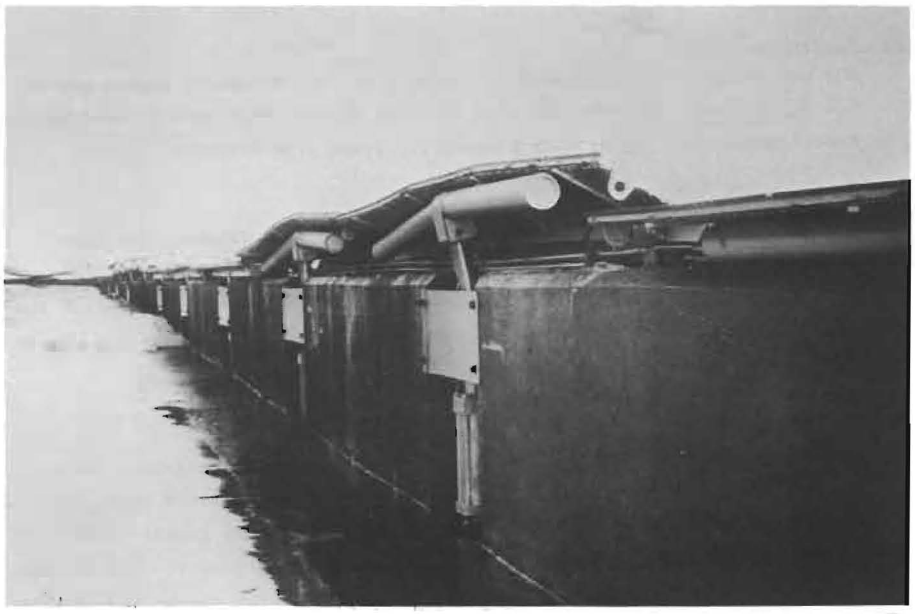

Fig. 9 The ice skimming weir on the apron wall in front of the ice sluice. The ice has destroyed this weir.

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DISCUSSIONS

Dr. B. Michel

How much water depth is needed on the sill of the ice sluice way to prevent ice slush, floes and cakes to stop in front of it? What are the continuity conditions at the approach to prevent ice slush from bridging?

Dr. A. Assur

Several cases of damage of engineering coupments from floating ice were shown in the ural presentation. For the benefit of future design. Would it be possible to publish the forces which were necessary to accomplish the described damage and relate them to the ice condition (thickness and magnitude of floes, approach velocity and similar information).

Dr. G. Sigurdsson

No measurements are available on these points, so I will only give you my considered opinion. Ice slush can be passed with very shallow depth over the sill (say 20 to 30 cm). Floes and cakes will require depths about equal to their own thickness to go over the ice skimming weir. The thicker floes are passed over the main spillway gates where they can be passed with depths of only 60 to 70% of their thickness due to the strong current, and more favourable approach geometry.

The continuity conditions at the approach can be described as follows. The slush ice has to pass over a short section of deep water between the shallow river section and the ice sluice, without thickening too much. The width of the section is approximately constant but the depth increases about 8 to 10 times over a distance of about 150 meters.

It is difficult to estimate exactly the forces that are caused by the ice. But for design purposes one might estimate that sheet ice as heavy as 100 tons could ram into the structures with velocities of about 1 m/sec.

ICE SYMPOSIUM 1970
REYKJAVIK

SOME ASPECTS OF THE DESIGN OF ICE PASSAGE FACILITIES FOR THE BURFELL HYDROELECTRIC PROJECT

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Vice President

Harza Engineering Company

Chicago, Illinois
U.S.A.

A fundamentally new concept in the design of the ice and silt passing facilities was used at Iceland's Burfell Hydroelectric Project. These facilities have operated through one winter season. This operation, while very satisfactory, has, nevertheless, shown the need of some equipment modifications or demonstrated possible improvements thereto for incorporation into similar designs at other locations. Most of these pertain to the regulating gates and are discussed. The design philosophy of the facilities is presented. Means for presenting the sludge ice from reaching the facilities are also presented.

The design and engineering supervision of construction of the Burfell Hydroelectric Project was by Harza Engineering Company International. This design included a fundamentally new concept of ice and silt passing facilities. The design concept of these facilities was to reduce the normally high velocity of the water approaching the river diversion structure by excavation of the riverbed and by some increase in surface levels by spillway flap gate control. This reduction permits the ice to float to the surface, be skimmed into a trough within which it is directed to downstream, then dropped into a high-gradient discharge canal. Gates on the skimming weir and at the downstream end of the trough control the quantity, depth, and velocity of the skimming water. The velocity reduction also permits the bedload to sink to near the excavated riverbed where it is directed into undersluices for release to the common discharge canal. Slide gates at the downstream end control the quantity of water required to move the bedload as well as the quantity of supplemental water required in the discharge canal to sluice both the ice and bedload to downstream. Plans and sections of the diversion structure are shown on Figure 1. Power diversions pass through open ports above the undersluices and under the ice trough and into the diversion canal. Lift gates located in a control structure within that canal regulate the amount of water flowing to the pondage reservoir of the powerplant.

The main spillway with four 20-meter long by 2.5-meter high flap gates is positioned at right angles to the diversion structure and located such that the over-deepened spillway bucket serves as a side runway channel to direct spillway overflow to the discharge canal, with regulation provided by a lift gate. This arrangement provides supplemental or emergency means of passing ice, bedload, and sluicing water.

These ice and silt passing facilities worked extremely well during their first season's operation the winter of 1969-70, even with the handicap of lack of experience in the prototype operation. The maximum professional benefit comes not from negative criticism or the converse, but from the designer presenting, with the benefit of hindsight, possible improvements based on the experiences to date, as is intended hereinafter.

The initial design of the ice and silt passing facilities was based somewhat on previously constructed projects, with the wall of the diversion structure acting as a shear wall, using some of the flow to direct the ice to downstream. The principal changes brought about by the hydraulic model studies were: (1) the addition of a skimming weir and trough paralleling the diversion structure wall to contain the ice and flushing water, plus (2) deepening the spillway bucket and converting it into a side channel to conduct ice, silt, and water into the discharge canal. The tests showed the former to be essential for reasonably satisfactory operation and the latter to be beneficial in some instances.

The ice passing facilities were operated through the winter of 1969-70 when ice flows were more severe than normal and they performed satisfactorily their design function of excluding ice from the power diversions. It can be said that the winter's operations were "as intended" but not always completely "as designed". This differential can be contributed in part to the physical element of design and construction, and in part to the human aspects of the operation. However, it must be kept in mind that the recent operations were on the basis of only about one-half the design diversion discharge required for the ultimate six-unit Burfell Plant. It is appropriate at this time to review the results of this first winter's operations in order to establish, insofar as feasible, any modifications which should be accomplished to the facilities in the near term as well as to establish improvements to be incorporated into future designs of similar facilities.

The operations have not established the optimum angle between the approach current and the ice skimming weir. The design provided this angle to be a rather acute one. However, an ice jam in the river early in the winter resulted in the water approaching the trough at almost a right angle and at moderately high surface velocity for a relatively long period of time, with the facilities continuing to perform their functions in a reasonably satisfactory manner. It is probable that this angle may vary widely, with either an optimum or an adequate one being dependent on local site conditions and design diversion capacity.

The optimum relationship of elevations between reservoir, weir and trough gate crest for ultimate design conditions was not established the first winter, principally because of operations at one-half or less of design diversion requirements. It is rather certain, however, that some control of skimming weir elevation is essential. Moreover, a completely positively controlled weir of heavy construction is preferable to the design provided at Burfell. It is possible that a segmented one of vertical action controlled from underneath would be best, even though probably expensive. The Burfell design did not provide for passing cake ice routinely except over the main spillway, but the human element resulted in passing much of that ice the first winter normally through the ice trough with at least some damage to the skimming gate leafs which were, of course, of relatively light construction. Thus the gate assemblies must be of heavy construction and be capable of being quickly removed from the flow path when heavy cake ice approaches. Such passage of hunks of cake ice through the ice trough may have in addition caused some impact damage to the flap gate, the side channel gate, and the bottom of the stilling basin at the head of the discharge canal.

The recesses for the flap gate operating arms tended to collect some ice which interfered with gate operation and resulted in some bending of a gate arm, possibly because of inadequate care in operation. Reflected heat applied to the ice will prevent this build-up. However, a flush-type installation with the operating mechanism housed inside the pier would be the preferred design.

The slide gate in each of the undersluices moves upward into a cavity when opened. Ice can float upward into this underwater cavity and form a blockage interfering with gate operation. While this apparently did not happen with any of the three gates the first winter, it is considered that a sealed membrane should be provided to exclude the ice, or else a full body gate be provided, although the latter is somewhat expensive.

The undersluices pass a considerable bedload. Thus sand could enter the metal-to-metal seals of the gate to the extent to cause binding and thus interfere with gate operation. A design utilizing more flexible seals of rubber or plastic should reduce this potential hazard and be easier and cheaper to maintain.

Some of the seals on the concrete "corks" filling the access slots in the bottom of the trough and extending to the bulkhead gate slots proved inadequate to prevent their removal by hydraulic pressure a few times. However, an adequate wrap-around design involving a glued attachment of the seal to the frame, which would not be damaged in placement as were the steel stud attachments, should be simple and effective. Venting of the slots is also necessary.

The rather thin skin plates on the normally downstream face of the side runway channel gate and the gates of the control structure were damaged during the winter, probably by churning cake ice. All of these gates are designed to permit unbalanced hydraulic loads on either side. Damage at the control structure gates was probably precipitated by the ice shearing the screwed studs attaching the plates to the gate frame and might not have occurred had the contractor followed the design and welded the plates to the frame. Damage to the skin plates on the side runway channel gate was probably mostly the result of the above-mentioned operation of passing cake ice routinely over the adjacent trough flap gates contrary to the design intent. It is obvious that heavier plates welded to the gate frame are required on the gate faces downstream from the general direction of flow.

Some ice was formed in the cavities between the skin plates of the control structure gates which increased greatly the gate handling weight. While this resulted from the deferral of the construction of the heated gate house on the concept that the control gates would not be used in normal operations until the design diversion was increased, it is probable that most or all of the cavities should be filled with Urethene foam to exclude the water. However, the resultant buoyancy may require the use of some ballast. Heating of the entire gate is an alternative protection, but expensive.

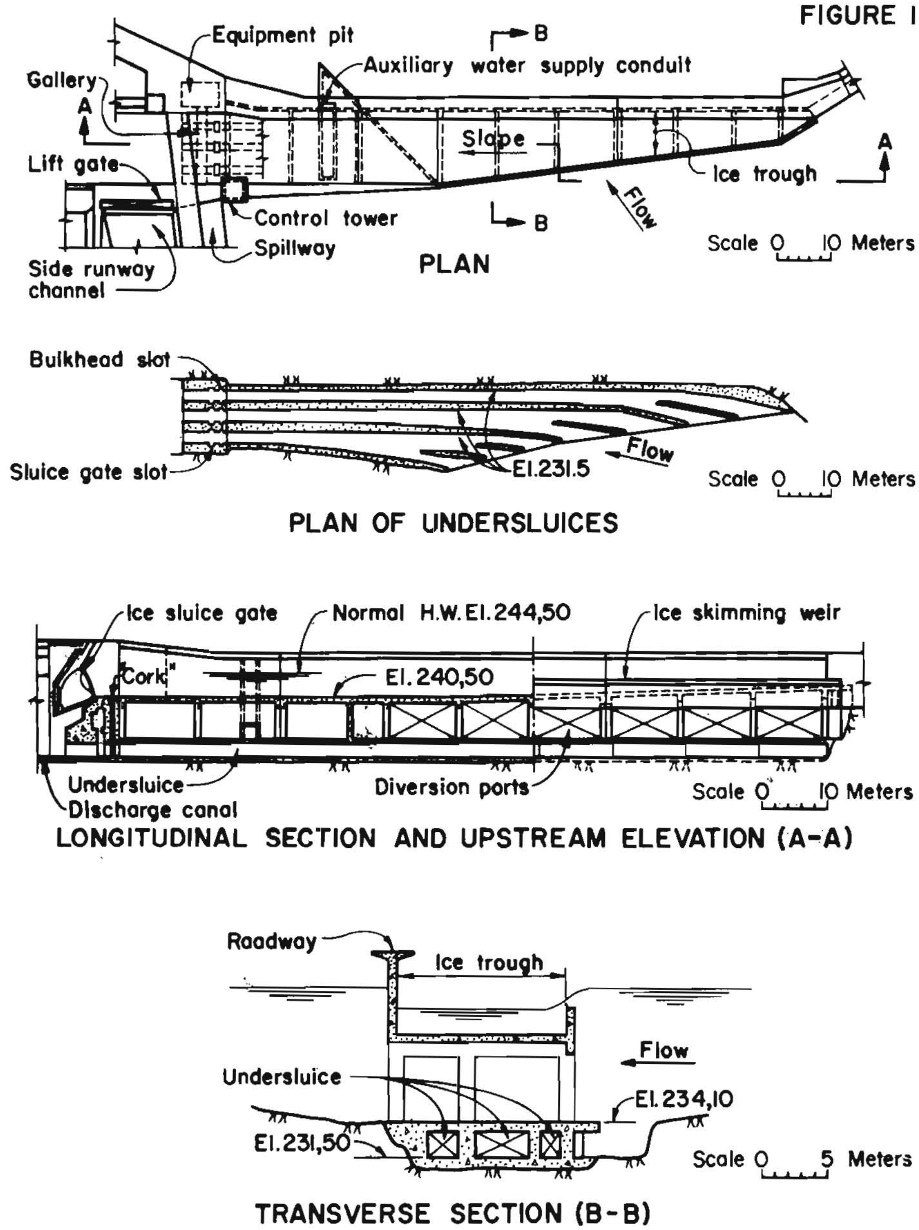
Hydraulic model tests of the overflow gates were made in a hydraulic laboratory for the manufacturer. The nappe splitters were, insofar as feasible, required to be designed to represent extensions of the gate skin plate. This was on the theory that they would be less likely to gather a build-up of ice than the more conventional type of raised splitters, with resultant interference to ice passing. This design, developed in the laboratory, worked very satisfactorily and no vibrations have been noted in any of the gates. However, the design supplied by the manufacturer was too light in construction such that a number were bent from the impact by cake ice. A stronger design is obviously required, but is a simple matter. A few conventional splitters were shown by the model test to be required on the higher flap gates to supplement the normal circumferential design in order to eliminate vibrations under some conditions. The manufacturer's heavier construction of these conventional splitters resisted the ice damage, thus proving the inadequacy of his other construction. Moreover, even these protruding nappe splitters showed no tendency to precipitate extensive ice build-up, but it must be considered that they were in locations of nearly constant flow. Thus, the circumferential design of adequate strength is to be preferred to the extent possible, with the more conventional type used only as necessary to prevent gate vibrations which cannot be prevented by the circumferential design alone.

On several occasions it was observed during the first winter that ice jams, beginning against the upstream side of the diversion structure, formed within a matter of seconds whenever the depth of water over the skimming weir became inadequate to permit the sludge ice to pass without stopping to the trough and irreversible regelation began. Such ice jams have, for the most part, remained until melted by warm water in the river after a thaw. Thus, it is evident that, while ice jams may also be precipitated by an overly high pool level, it is better to err on the high side of skimming depth than on the low.

The reduction in the amount of ice (and of bedload) reaching the diversion structure to be accomplished by upstream constructions was an important part of the original project design which is now being undertaken. These construction features will include, as appropriate:

1. Canalization of the river upstream to reduce the area of exposure to frazil ice formation by such means as permeable weirs, channel narrowing, temporary booms, and other appropriate means.
2. Elimination of rapids and waterfalls which are great ice producers.
3. Installation of snow fences to entrap blowing snow, thus preventing it from entering the open river to form snow slush.
4. Creation of upstream multipurpose projects involving reservoirs of adequate surface area to largely eliminate frazil ice production in that river reach. This is, of course, a long term solution.

FIGURE I



BURFELL HYDROELECTRIC PROJECT
DIVERSION STRUCTURES - PLANS AND SECTIONS

ICE SYMPOSIUM 1970
REYKJAVIK

REGULARISATION DE LA FORMATION DU FRASIL DANS LES BIEFS AVAL DES CENTRALES SITUEES PRES DES EMBOUCHURES

CONTROL OF FRAZIL ICE FORMATION DOWNSTREAM FROM POWER PLANTS SITUATED IN RIVER MOUTHS

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Résumé

Deux procédés hydraulique et thermique de régularisation de la formation du frasil lors du mélange des eaux douce et salée sont considérés. Le procédé hydraulique se réduit à la régularisation de la quantité du frasil par la variation du débit de l'eau douce. Le procédé est basé sur les études de la valeur du "surrefroidissement de concentration" durant le mélange des eaux douce et salée. Pour le procédé thermique on détermine les vitesses de la fonte de glace en différentes conditions thermiques, hydrodynamiques et de concentration et on obtient les relations correspondantes.

Synopsis

Considered are hydraulic and thermal methods for the control of frazil ice formation due to mixing of fresh and salt water. The hydraulic method provides control of frazil ice amount by means of varying the discharge of fresh water. The method is based on the study of the value of concentration supercooling" induced by mixing of fresh and salt water. For application of the thermal method, the velocities of ice melting under different temperature, concentration, and hydrodynamic conditions are established and corresponding critical relationships are obtained.

Introduction

On observe souvent dans les biefs aval des usines hydro-électriques, dans les bassins des usines marémotrices et dans les embouchures à marée le phénomène de la formation du frasil qui donne lieu à la formation des embâcles de frasil engendrant l'élévation du niveau en amont et, par conséquant la diminution de la chute et du rendement de l'usine, aussi bien que l'inondation du pays peuplé.

Les causes de la formation de glace ayant lieu lors de l'intrusion de l'eau salée dans les embouchures des rivières sont tout à fait différentes: refroidissement du fond de la rivière et de la nappe de glace, et surtout, l'apparition du frasil imputable au mélange des eaux de mer et de rivière.

Le volume du frasil peut être régularisé par deux procédés: hydraulique et thermique.

Le procédé hydraulique est employé pour la régularisation de la formation du frasil lors du mélange des eaux douce et salée froides, mais pas surfondues; ce procédé se réduit alors à choisir les débits de l'eau douce et salée de telle manière que le volume du frasil se formant ne dépasse la valeur admissible.

Le procédé thermique consiste à utiliser la chaleur de l'eau pour fondre la glace déjà existante.

Il y a une correlation entre ces deux procédés. Ainsi, le changement du régime hydraulique peut être avantageux pour le procédé thermique: par exemple, l'augmentation de la vitesse de l'écoulement de l'eau chaude accelère la fusion des glaces; les changements du régime thermique influencent les conditions de la régularisation hydraulique: variations des températures provoquent la variation du volume du frasil se produisant à la suite du mélange des eaux.

§1. Régularisation hydraulique

C est lors du mélange des eaux de rivière et de mer à températures (t_1 et t_2) proches aux températures de leur congélation que se forme la plus grande quantité du frasil. Cela peut être expliqué par le fait que dans les conditions indiquées un "surrefroidissement de concentration" du mélange (Δt) a lieu, ainsi la température du mélange ($t_{mél}$) se trouve au-dessous de la température de sa congélation (t_{con}):

$$\Delta t = t_{mél} - t_{con} < 0°C \qquad (1)$$

La température du mélange se détermine par une résolution combinée des équations du bilan de la chaleur et du sel dans celui-ci. La fig.1 donne les résultats du calcul de Δt pour le cas de $t_1 = 0°C$ et $t_2 = t_{con}$ ce qui démontre que le "surrefroidissement de concentration" est la fonction de la relation $n = \frac{G_1}{G_2}$ entre les débits en poids des eaux douce (G_1) et salée (G_2) et de la concentration en poids de l'eau salée ξ_2. Plus est ξ_2, plus est la surfusion de concentration: quant à l'augmentation de n elle implique l'accroissement du "surrefroidissement de concentration" jusqu'à Δt_{max} et ensuite son abaissement. Les résultats donnés sur la fig.1 permettent de trouver la relation $\Delta t_{max} = f(n, \xi_2)$.

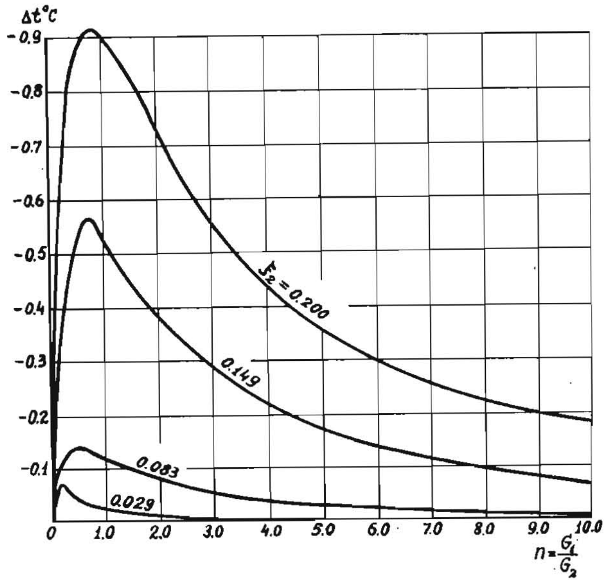

Fig.1.

Si l'on tient compte de la porosité (p = 0.6 – 0.7), le débit du frasil peut être déterminé par:

$$Q_{fr} = \frac{(-\Delta t) \cdot C_{mél} \cdot G_1 \left(1 + \frac{1}{n}\right)}{\rho \, \gamma_{gl} (1 - p)}, \left[\frac{m^3}{h}\right], \qquad (2)$$

où $C_{mél}$–capacité calorifique spécifique du mélange $\frac{J}{kg \cdot grad}$, γ_{gl} = 920 kg/m^3 –poids spécifique de la glace, ρ = 92,8 $\frac{wt \cdot h}{kg}$ –chaleur latente de la formation de glace.

Par exemple, conformément à la figure 1 a ξ_2=0,029 nous avons Δt_{max} =–0,08°C et n =0,1; donc, $\xi_{mél} = \frac{\xi_2}{n+1}$ = 0,0264 et $C_{mél}$=1,12 $\frac{wt \cdot h}{kg \cdot grad}$.

Par conséquent

$$Q_{fr} = \frac{0{,}08 \cdot 1{,}12 \left(1 + \frac{1}{0{,}1}\right)}{92{,}8 \cdot 920 (1 - 0{,}7)} G_1 = 0{,}0384 \cdot 10^{-3} G_1$$

c'est-à-dire près de 4% du débit volumique de l'eau douce.

En pratique on peut régulariser la quantité du frasil se formant lors du mélange des eaux par variation des débits d'eau douce et salée (n) en réglant les débits d'évacuation de l'usine.

§2. Régularisation thermique

Pour ce procédé il faut savoir les vitesses de la fonte de glace en différentes conditions.

C est dans ce but qu'on a effectué les recherches expérimentales sur la fonte des plaquettes de glace (10 x 10 x 2,5 cm) disposées horizontalement dans l'eau et les solutions aqueuses de NaCl dans les conditions de la convection libre et forcée.

L'approximation des résultats relatifs à la vitesse de la fonte de glace à convection libre dans les limites de $t_{con} < t_{mél} < 40^\circ$ et $0 \leq \xi \leq 0,135$ s'effectue à l'aide de l'équation

$$W = 0,11\left(\frac{t_{mél} - t_{con}}{t_{mél} + 273}\right)^{1,7(1+0,9\xi)} \qquad (3)$$

avec W –vitesse de la fonte de glace, cm/sec.

Les recherches sur la fonte à convection forcée ont été accomplies pour trois schémas différents de l'écoulement autour d'un obstacle dans les limites de $t_{con} < t_{mél} < 20^\circ C$, $0 < \xi \leq 0,135$ et $0 < v \leq 0,5\ m/sec.$

Les résultats des expériences sont consignés dans la table ci-dessous:

Table

Schéma	Vitesse de la glace et du courant	Résultats expérimentaux
		8
I	Ecoulement autour d'un obstacle fixe $V_{gl} = 0,\ V_c \neq 0$	$Nu_I = 0,0138\ Pe^{0,8}$
II	La glace s'avance avec le courant $V_{gl} \approx V_c \neq 0$	$Nu_{II} = 0,0693\ Pe^{0,5}$
III	La glace est en mouvement dans le liquide en repos $V_{gl} \neq 0,\ V_c = 0$	$Nu_{III} = 0,0288\ Pe^{0,7}$

où $Nu = \frac{\alpha L}{\lambda}$ –critère de Nusselt; $Pe = \frac{vL}{a}$ –critère de Peclet; α, λ, a –taux de transfert de chaleur eau-glace, de la conductivité thermique et de la diffusibilité thermique; L –dimension déterminante, égale à la relation entre le volume de glace et sa surface.

La plus grande vitesse de la fusion est observée pour le premier schéma, c'est-à-dire pour le cas de l'écoulement autour d'un obstacle fixe, à

$$Nu_I > Nu_{III} > Nu_{II}.$$

ICE SYMPOSIUM 1970
REYKJAVIK

LOCAL HEATING OF WATER FLOW AND FRAZIL PASSAGE THROUGH TURBINES

RECHAUFFAGE LOCAL D UN COURS D EAU ET EVACUATION DU FRAZIL PAR LES TURBINES

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Synopsis

A problem is considered on utilization of groundwater to prevent subcooling of water in water courses. A conclusion is drawn that in some cases frazil ice may be passed through hydropower plant turbines.

Résumé

L'auteur examine le problème de l'utilisation des eaux souterraines pour la lutte contre la surfusion de l'eau dans un cours d'eau. Les recherches ont démontré la possibilité d'évacuer dans un nombre de cas le frazil par les turbines des usines hydroélectriques.

At hydropower stations with a power channel flow velocities exceeding 0.8 – 1.0 m/sec are in most cases determined by the economically optimal value of the channel slope. This condition is unfavourable to ice cover formation, in particular, in the South of the Soviet Union(the Caucasus, Central Asia and South Kazakhstan) . Thus, in winter a frazil transport regime with subcooling is established in open power channels of hydropower plants.

At hydropower stations with a power channel in Transcaucasia and Central Asia for trouble-free winter operation ice chutes are usually arranged in pressure basins for diverting frazil masses from turbines, and electric heating of trash racks in pressure tanks is installed to prevent their freezing-over due to subcooling.

These winter arrangements are highly inefficient economically because of a considerable reduction in power production due to the discharge of frazil and water by-passing the turbine units, (the discharge being 2 to 3 times higher as compared to ordinary discharge) and substantial power consumption for heating trash racks. As a rule, these phenomena occur at a time of peak power demands in power grids.

A different winter operation scheme is feasible because in the majority of cases groundwater is available in the vicinity of the plant headworks, the groundwater temperature ranging from 15° to 18°C, i.e. close to mean annual ambient air temperatures in the region.

As the observed supercooled water temperatures are limited to 0.04–0.06°C below zero in natural conditions, an inflow of about 0.3–0.5% of ground-water of the power channel discharge is sufficient to eliminate subcooling.

This is however valid in case the heat of the inflown groundwater is not lost on the way and through melting of frazil ice. Therefore, on the one hand, the elements of the groundwater supply system at the injection point should be thouroughly heat insulated. On the other hand, the injection of warm groundwater should be effected in small subsurface fluents throughout the channel cross-section. For this purpose an injector designed at the Kazakh Institute of Power Engineering may be employed, consisting of a heat-proof collector across the flow above the water level, equipped with perforated vertical pipes across the whole depth of the flow, with small openings on the upstream side.

To avoid freezing-over of trash racks they should be completely submerged and supported by the skimmer wall located not less than 0.5 m below the water surface.

Under such conditions it is possible to pass frazil ice through the turbine waterways instead of by-passing it. The operation experience obtained at a number of hydropower plants without subcooling (with 80–100 mm vertical spacing of trash racks) confirms that no difficulties arise in passing frazil masses through turbine waterways.

Thus, the availability of warm groundwater and the feasibility of captation of 3-5 l/sec per cubic meter of water discharge in the power channel permit to put into practice this winter operation scheme. The economic effectiveness of the scheme depends on the magnitude of capital investments for the captation and conveyance of warm groundwater to the plant pressure basin.

ICE SYMPOSIUM 1970
REYKJAVIK

CALCULATION OF A FLOW FORMING UNIT FOR PROTECTION OF DOCKS FROM FLOATING ICE-BLOCKS

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When placing ships into floating docks in winter, many difficulties connected with the penetration of drifting ice (together with the ship) into the dock are likely to arise.
To prevent the case, stream forming devices may be efficiently used.

In this work are obtained the dependences allowing to establish rational disposition of flow forming units along the floating dock and their main parametres for protection of slipdeck from floating ice-blocks when towing the ship and lifting the dock up.

Quand on dépose les navires dans les dock flottant en conditions d'hivers des obstacles surgissent à cause des glaces flottantes qui arrivent avec le navires dans les limites du dock flottant.
Pour prévenir çelà on peut employé très éffectivement des formateurs de flots.

Dans le travail on a obtenu la dèpendance qui permet de fixer la position rationelle du flux suivant la longueur du dock flottant et les paramétre principales pour prèvenir l'atteinte de la glace sur l'embarcation pontée au cours de la flottation du dock et du navire.

During the submersion of a floating dock, surrounding broken ice-field is acted upon by:

friction of the flow against the underside of ice sheets, F_1; hydrodynamic pressure load on the under water part of the ice field extremity F_2 ; friction of the air against the upper surface of ice sheets, F_3 ; the force F_4 due to inclination of the free surface; and friction of ice against the dock tower inner walls and protruding parts, F_5 .

The force F_5 can be excluded from consideration for it counteracts both the forces pushing the ice inside the dock and the hydrodynamic influence on the floating ice-sheets due to the artificially created flow.

In view of the above one can calculate the total force value under the influence of which the floating ice enters the dock when the latter is submerged following the formula:

$$F_B = k_1 v_d^2 B L_n + k_2 v_d^2 Beh + k_3 w^2 B L_n + 2\gamma B L_n n \frac{V_n}{L_n^2} , \qquad (1)$$

where:

- V_d – the surface velocity of the flow filling up the space between the dock towrs, m/sec.;
- W – wind velocity assumed equal to 10 m/sec.;
- B – the width of the moving ice field equal to the distance between the inner walls of the dock towers, meters;
- L_n – the length of the ice field moving inside the dock* assumed to be $L_n = 3B$, meters;
- h – the average thickness of ice-floes, in meters;
- k_1 – the coefficient accounting for water friction against the underside of ice-floes and for density of water at 0°C and of the value equal to 0,5 kg.sec^2/ m^4; [1]
- k_2 – coefficient allowing for the cross-section shape of ice-floes and for conditions of water flow round them, as well as water density at 0°C; its value is assumed to be equal to 50 kg.sec^2/m^4; [1]

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* Increase of the ice field length "L_n" over (2,5 + 3,0)B does not cause further increase of the flow dragging force which is due to the fact that the preassure increment is absorbed by the dock towers thanks to the ice floe friction against their inner walls.

k_3 – coefficient allowing for the air friction against the upper ice-floe surface and the air density within temperature range – 20°C to + 10°C, its value being assumed equal to 0, 001–0,002 $kg.sec^2/m^4$;

e – coefficient accounting for the ice-floe extremity portions acted upon by the flow hydrodynamic pressure. With the average broken ice field density of 50%** and the specific weight of ice $\gamma = 920\ kg/m^3$ its value is equal to 0,46;

L_n – the length of the dock as measured along the pontoon deck, in meters;

V_n – average rate of immersion of the dock, in meters per second.

The average rate of water inflow into the inter-tower space of the dock during its immersion may be calculated on the basis of the rate of water flow entering one of the dock ends. The incoming flow surface velocity, V_α, can be most conveniently determined according to the graph of Fig.1 with $\alpha = 90°$.

The dragging force of the flow created by flow forming devices F_{no}, is the total of forces F_1 and F_2 pushing the ice floes out of the dock being immersed. The force F_{no} is to be equal or somewhat exceed the total force of pule-in F_B in order to permit the removing of the ice for the given distance, i.e.

$$F_{no} = KF_B, \qquad (2)$$

where:

K – is the marginal coefficient equal to 1,2.

In view of the above the flow dragging force formula may be written in the form of

$$F_{no} = k_1 v_n^2 BL_n + k_2 v_n^2 Beh = 1,5\ v_n^2 B(B+30h) \qquad (3)$$

The necessary surface flow velocity which the flow forming devices are to assure, may be found from equations (1) and (3), considering the condition of (2):

$$V_n = \sqrt{\frac{0,8(1,2v_\alpha^2 B + 18,4v_\alpha^2 h + 0,24B + 4416Bh\frac{V_n}{L_n})}{B + 30h}} \qquad (4)$$

** As a result of numerous observations during docking of ships it was found that upon the dock immersion and filling up the dock by the broken ice coming from the water surface the ratio of filling up the water surface does not exceed 50%.

The velocity of water outflow from the nozzle of a flow forming device (V_o) may be replaced by the thrust of the flow forming device, that is by the parameter which is simple to determine:

$$P_y = \rho F_p \cdot V_o^2 , \qquad (5)$$

or :

$$V_o = \sqrt{\frac{P_y}{\rho \cdot F_p}} , \qquad (6)$$

where:

P_y - the thrust of the hydromechanical unit of the flow forming device, in kg.

$F_p = \pi R^2$ - the hydraulic cross section of the propeller of the flow forming device, square meters;

R - the propeller radius;

ρ - water density at 0°C, equals 102 $kg.sec^2/m^4$.

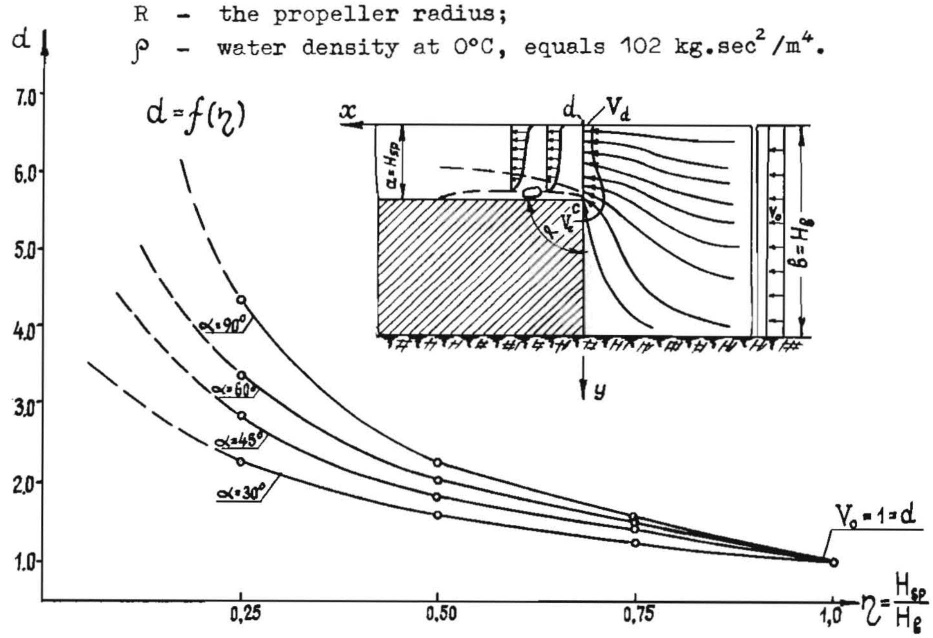

Fig. 1 - The graph of surface flow velocity coefficient (d) variation with the degree of the flow constraint (X) and declivity of an obstacle encountered (α).

As it has been found from numerous thrust measurements during actual testing of different types of flow forming devices over the power range 4,5 to 30 kw the thrust of the hydromechanical unit varies at the rate of 20 to 28 kg per 1kw of the electrical motor power.

As a result of the inverstigation of the laws governing the spreading of submerged hydraulic jets in a limited space (docks) the following relationship between the velocity (V_n) and the thrust of the hydromechanical unit of a flow forming device was found to be :

$$V_n = \frac{1{,}54\sqrt{P_y}}{L - 14{,}3R} \tag{7}$$

On the basis of the relationship for the parameters of a jet injected from the nozzle into the body of the same liquid, described by the equation of liquid motion with variable rate of flow along the duct circuit, one can find that:

$$L_p = K_p\left(\frac{V_o}{V_{cp}} - 1\right) , \tag{8}$$

where:

L_p - the distance from the nozzle outflow section to a given range;

V_o - the mean velocity of outflow;

V_{cp} - the mean velocity of flow at a given cross section;

K_p - an empirical coefficient.

If the surface velocity of the flow, created by the flow forming device within the range, is assumed to be not less than (V_n), the equations (7) and (8) allow to arrange rationally the flow forming devices along the length of the dock.

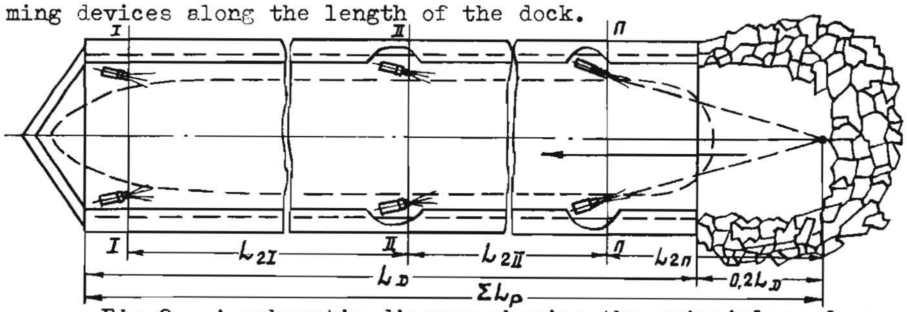

Fig.2 - A schematic diagram showing the principles of positioning of flow forming devices along the length of a floating dock.

The distance between rows of flow forming devices (Fig. 2) with allowance made for the parallel propagation of a pair of streams in passing flow current created by the preceeding rows of flow forming devices, is determined according to the formula:

$$L_2' = m_g'\left(\frac{1{,}54\sqrt{P_y}}{V_n} - 14{,}3\,R'\right) \quad , \qquad (9)$$

where:

m_g is a coefficient of multiplicity of streams depending on the distance between the axis of a pair of parallel streams.

Operation of the ice-guard equipment in existing docks has shown its high efficiency as regards docking of sea-going ships and regular servicing of docks in ice conditions.

Literature:

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Heaps of Ice in Reservoirs (Methods of Calculating). Works of co-ordinating councils on hydrotechnique. Issue 42. L., "Energy", 1968.

ICE SYMPOSIUM 1970
REYKJAVIK

PASSING ICE THROUGH THE STRUCTURES OF HYDRO POWER PLANTS UNDER CONSTRUCTION IN SIBERIA

EVACUATION DE LA GLACE PAR LES OUVRAGES DES AMENAGEMENTS HYDRO-ELECTRIQUES EN CONSTRUCTION EN SIBERIE

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Synopsis

In Siberia under severe climatic conditions, on a number of hydroelectric stations in the erection stage ice was successfully passed in spring through a stretch of the river constricted by cofferdams, through spillway openings, and bottom outlets. A special hydraulic regime was developed for passing ice through bottom outlets upstream and downstream from structures under construction.

Résumé

Dans les conditions climatiques rigoureuses de la Sibérie on a réussi sur un nombre d'aménagements hydro-électriques en construction à évacuer au printemps la glace par le tronçon rétréci à cause des batardeaux, par les pertuis de la partie déversante du barrage, ainsi que par les orifices de fond. Pour l'évacuation de la glace par les orifices de fond on a mis au point un régime hydraulique spécial dans les biefs amont et aval des ouvrages en construction.

Hydroelectric development now in progress in the eastern regions of the USSR enriches our experience in building structures under Siberia's severe climatic conditions.

Conditions of spring ice drift at the following construction stages are considered:

a) ice drift on a river stretch constricted by first-stage cofferdams;

b) passing ice through spillway openings, while the channel is constricted by the second-stage cofferdams;

c) passing ice through bottom outlets.

Used are field observation data on passing ice during the construction period at the following power plants: the Ust-Kamenogorsk and Buchtarma plants on the Irtish river, the Novosibirsk plant on the Ob river, the Vilyui plant on the Vilyui river and the Krasnoyarsk plant on the Yenisei river.

When ice was passing a river stretch with first-stage cofferdams, the channel constriction ranged from 0.30 to 0.54. At the site of the Vilyui plant ice passed through a construction trench 18 m wide, with the river constricted to about 180 m wide. Channel constriction through first-stage cofferdams exerted generally little effect on the break-up of the ice cover and on spring ice drift.

On the river stretch downstream from the cofferdams the break-up often occurred 1-3 days ahead of break-up in the upper pool and no ice gorges were noted therefore in the vicinity of cofferdams.

Formation of an ice gorge took place on the Irtish river within a few kilometers downstream from the cofferdam. The above formation resulted in the water stage rise in the vicinity of the plant structures. The magnitude of the rise did not exceed those due to ice gorges on unregulated rivers.

During ice pushes considerable masses of ice several meters high heaped up on the upstream cofferdams at some sites. The differences in the water level that develop in the vicinity of the structures (for instance, at the beginning of the cofferdams) greatly affect the destruction of ice fields. During the ice drift period ice floated along the constricted portion of the river more or less calmly.

Thanks to higher current velocities in the proximity to the cofferdams, as compared to those in the upstream stretch of the river, there occurred no rise in ice drift density despite the reduction of the channel width.

As ice passed through the spillway openings the channel was constricted to 0.75-0.82 by the second-stage cofferdams and the spillway piers. The span between the piers amounted to 18-24 m. An appreciable constriction of the channel by hydraulic structures resulted in retardment of ice drift start upstream. Retardment of ice break-up in the proximity to the dam was accompanied by the development of appreciable ice gorges at the end of the backwater zone. Those ice gorges broke up into more or less small ice floes. In the upstream pool the ice cover lost its strength. At the beginning of the ice drift the dimensions of ice fields approaching the plant structures considerably exceeded the width of the spillway spans. The ice fields generally broke up in zones of concentrated water level differences upstream of the openings in the dam under construction (for instance, at the channel constriction at the beginning of cofferdams and in front of the dam) . An increase of current velocities in front of the unfinished dam was accompanied by dispersal of ice floes, which when striking against the piers got broken in-to pieces commensurate to the width of the openings in the dam under construction. Through the openings the ice floes often passed in the vertical position.

After the break up of ice gorges upstream a heavy mass of ice approached the dam, which mass consisted of comparatively small ice floes heaped up in several layers 3–4 m thick. Specific ice discharges through the dam amounted to 2.5–5.5 cu.m/sec instead of the usual discharges of 0.5–2.5 cu.m/sec for the unregulated river.

After the break of ice gorges increased water discharges (a "break wave") passed through the unfinished dam. The increase of water discharge and of outlet roughness combined with the decrease of the water cross-section due to ice resulted at the time of ice drift in increasing the water level difference between the upstream and the downstream pools. For some power plants this difference amounted from 2 or 3 up to 4 m.

Ice passed through the unfinished dam openings almost without delay, but the ice drift was of a turbulent nature. Through narrow but deep openings ice passed easier (though with higher specific discharges) than through wide but shallow openings.

At the Krasnoyarsk station ice passed through 18 bottom outlets during two years. The outlets were 6 by 12 m. All the openings were fitted with slide gates. At the time of ice drift the second-stage cofferdams allowed to keep the water level more than 20 m higher as compared to the winter level.

With the turbulent nature of ice drift, considerable ice discharge, enormous ice block sizes and high ice strength the passage of ice through bottom outlets presented a serious problem. Only ice that had lost its strength could pass through small bottom openings.

To weaken the ice strength by the warm runoff and solar radiation ice was temporarily accumulated in the upstream pool. This delay was achieved through current velocity control due to gate operation, current velocities not exceeding 0.5 m/sec being maintained. At the time when the water level rose there was an anchored ice field 25–35 km in length in front of the dam. A great ice gorge composed of ice floes coming from the upper reaches formed at the end of the backwater zone of the dam.

After water temperature rose above zero speedy thawing of ice began owing to warm runoff. The ice thawing was most intensive in the upper part of the storage reservoir at the ice gorge.

Mass drifting of ice started in 6 –8 days, as the ice strength was reduced to 20–40 t/sq.m and the entraining force of the current was breaking up the weakened ice cover. At first great ice fields approached the dam and broke easily to pieces at the impact with the dam piers. The broken ice was drawn into the bottom outlets. Later masses of small broken ice cakes, heaped up in several layers, floated continuosly toward the dam and easily passed through the bottom outlets. Some strong ice cakes came to a stop in front of the dam, broke slowly and were drawn with difficulty into the bottom outlets.

Next year as water discharges rose quickly, the passing of the ice through the spillway was begun early. Being of considerable strength the ice passed through the bottom outlets with difficulty and delays of several minutes in front of the dam. Ice dived in the bottom outlets that were located at a depth of 8 m from the water surface. From 30% to 40% of the accumulated ice was moved downstream, the rest of it melted in the upstream pool. Arrangements provided in the project design enabled to pass the ice drift trouble-free through the partly built structures.

ICE SYMPOSIUM 1970
REYKJAVIK

CONDITION FOR JAMLESS ICE PASSING THROUGH UNFINISHED DAM WITH LOW SILL.

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In this work are considered the problems of passing the ice through an unfinished dam with a low sill.

Criterial condition is found for drifting of ice in head race of hydropower station being built. Critical magnitude for ratio of dimensions of a span to the dimension of the piece of ice is found for passing separate pieces of ice. Pieces of ice which have dimensions larger than criterial ones are taken with the flow to the tail-water.

On a examiné les questions du passage de la glace par le barrage inachevé à seuil bas.

On a trouvé la condition aux limites de l'évacuation de glace à l'amont de l'usine hydro-électrique en construction. Pour faire passer la glace on a trouvé la valeur critique du rapport entre les dimensions d'un pertuis et du glacon. Les glacons de dimensions critiques sont entrainés par le courant vers le bief aval.

When constructing a hydroelectric station including a concrete dam, the first, after damming the river bed, flood is usually let through an unfinished dam with a sill laid at bottom level or slightly above it.

As calculations and accumulated experience show, with so low a sill, at the entrance to the contraction, formed by dikes of the second turn, there appears water plane drop Z , which greatly exceeds value

$$Z_{min} = 0,037 \sqrt{\frac{hR}{\gamma_o}} \qquad (1)$$

(h - ice depth; R - bending strength of ice covering; γ_o - water volume weight), required to break ice fields.*
That is why from large floes when they enter the contraction there periodically separate sheets with a mean size along the water flow

$$d = 11 \sqrt[4]{\frac{Rh^3}{\gamma_o}} \qquad (2)$$

which then, during the movement along the contracted bed, break down into ice blocks of length $c = (1,0 + 1,3)d$

Pattern of ice run in the contracted bed and unfinished dam operation, when letting ice blocks, formed as a result of ice field breaking, through, largely depends upon the relation between its span width b and ice block size d . With the relations $\frac{b}{d}$, exceeding some critical value

$$\frac{b_*}{d} = 0,1 \sqrt{\frac{gR}{\gamma_o v^2}} \qquad (3)$$

(v - mean flow speed before dam; g - gravitational acceleration) separate blocks of ice which came to the dam are immediately carried off by the stream to the aft bay.

With the values $\frac{b}{d}$, lower than critical ones,ice blocks stop in front of the dam what leads to pilling up of ice masses at the head-

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* D.F.Panfilov, Breaking of Ice Fields under the Influence of Local Changes of Water Plane, "Gydrotechnicheskoje Stroitelstvo" ("Hydraulic Engineering Construction"), 12, 1965

race, to their compaction by the stream, and,as a result of it, to the creation of thrust transferred to the banks. With an insufficient span width for the given particular conditions ice pillings up are not capable to overcome the resistance of the dam piers and of ice-carying route banks even in case they spread throughout the whole water plane between the upstream dike cross section and the dam. In other words, there exists the second critical width of a span for which,as applied to the case, when the longitudinal dike is parallel to the bank, the following fomula was derived previously : **

$$b_{min} = \frac{1}{n+1}\left\{\frac{c_{np}}{v}\sqrt{\frac{2\xi n b_o h k R}{\gamma_o(1-\eta)\left[\eta_2 + \frac{\gamma h}{\gamma_o H}(1-\varepsilon)\right]}} - n b_o\right\} \qquad (4)$$

Since for carrying out practical calculations by formulas (3) and (4), it is necessary to know ice depth and strength for the moment of ice run, besides the main characteristics of the water flow and the dam, the investigation was continued, and as a result of theoretical analysis and handling available experimental data there was derived a criterium inequality determining the conditions for ice run coming at the headrace of the hydro-electric project:

$$\frac{ko}{Fz} \leqslant 3,1\left(1+0,1\frac{H_o}{h}\right) \quad , \qquad (5)$$

where $ko = \frac{R}{\gamma_o B_o}$; $Fz = \frac{v_o^2}{gH_o}$; B_o , H_o and v_o - width, mean depth and mean speed of the stream before the hydroelectric project.

When the bed is contracted along its width for 50-70% as a first approximation we may assume $\frac{h}{H} = 0,1$ and $B_o v_o = B v$ (B - bed width contracted by dikes). That is why, taking into

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** D.F.Panfilov, Calculation of Dam Crest for Letting Ice Through, "Gydrotekhnicheskoje Stroitelstvo" ("Hydraulic Engineering Construction"), 7,1968.

consideration formulas (2) and (5) and noted during experiments deviations of maximum values d from average ones (up to 25%), it is possible to derive the following approximate relation instead of expression (3):

$$\frac{b_*}{B} = 0{,}95\sqrt{\frac{v_o^2}{gB_o}} \tag{6}$$

Finally, solving together expressions (4) and (5) and substituting numerical values of coefficients characterizing the interaction of water-ice flow with the dam piers and the bed, after some simplification not inflicting serious errors, condition for jamless passing of compacted ice masses between low sill dam blocks is obtained:

$$\frac{b_o}{b_1} = 0{,}41k\frac{B_o}{B}(1-\eta) \tag{7}$$

In this inequality $\eta = exp(-0{,}3\frac{L}{B})$; L - distance from contraction entrance to dam; B - overall width of dam block front equal to contracted bed width; b_o - pier thickness; $b_1 = b + b_0$ - span width between piers' axes; k - coefficient allowing for contact density of ice mass with pier front edges (varies linearly from k = 0.6 at b_o = 3m to k = 1 at b_o = 10m). In as much as the numerical coefficients of formulas (6) and (7) are obtained as a result of few experiments, they should be refined in future.

The analysis of inequality (7) results in an important and, so far as it is known, unusual conclusion : the necessity of letting ice through does not limit the choice of degree of river bed contraction by dikes. In other words, ice can be let through an unfinished dam with low sill at any relations $\frac{B}{B_o}$; it is only necessary to determine correctly the width of its separate spans and the elevation of the dam blocks.

EXPERIENCE WITH ICE PROBLEMS IN PASVIK RIVER

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SYNOPSIS

The fundamental importance of supercooled water in the creation of ice troubles is reviewed in connection with a description of ice conditions in Pasvik River and experience with measures to reduce these troubles.

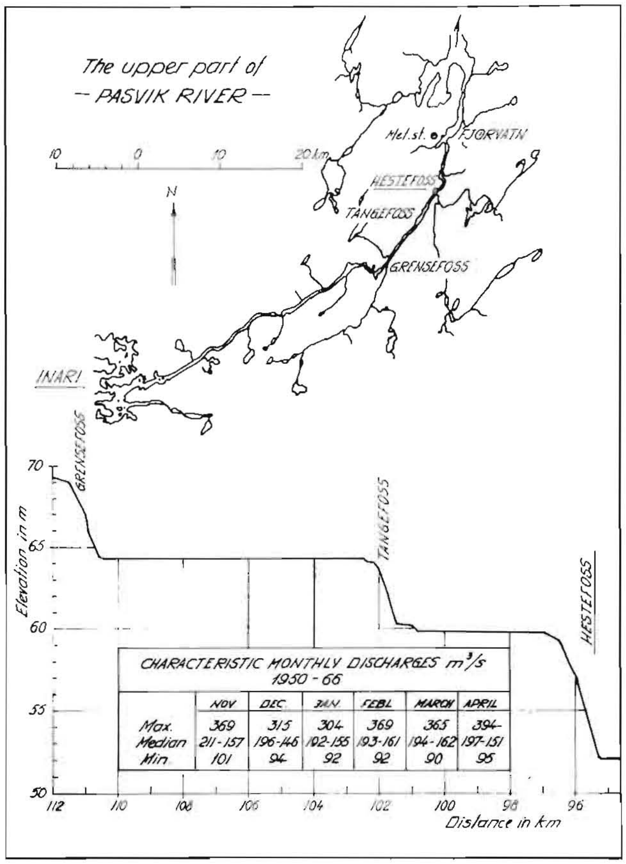
RÉSUMÉ

L'importance fondamentale de l'eau surfondue pour les conditions adverses crées par le glace dans lacs et rivières sont exposés avec une description des conditions de glace dans la rivière Pasvik-elv. Experiences des moyens employée pour reducer ces troubles sont aussi traitées.

1. HYDROLOGICAL AND METEOROLOGICAL DATA RELEVANT TO THE ICE CONDITIONS

The Pasvik river basin is located in the northern part of Norway, at the boundary with the Soviet and Finland, catchment area 20890 km^2. The river flows from Inari lake, area 1800 km^2, altitude 120 m.

In fig. 1 is given the size of drainage area and the longitudinal section of the river between Inari and Fjorlake, 95-112 km.



CHARACTERISTIC MONTHLY DISCHARGES m^3/s 1950 - 66	NOV	DEC	JAN	FEBL	MARCH	APRIL
Max.	369	315	304	369	365	394
Median	211-157	196-146	192-155	193-161	194-162	197-151
Min	101	94	92	92	90	95

Fig. 1 The upper part of Pasvik river.

The flow from Inari is normally regulated within the limit of 120-240 m^3/s, as an average of 24 hours. An extension of this limit to 80-240 m^3/s is permitted under certain conditions.

The duration of the winter and its intensity is illustrated by the following table based on measurements at the Pasvik meteorological station.

Winter characteristics from observations at Pasvik meteorological station 1963-70

Winter	a	b	c	d	e
1962-63			9	-25,5	25/4
63-64	23/10	16/11	11	-29,3	5/5
64-65	2/11	7/11	19	-26,8	15/5
65-66	8/10	25/10	24	-37,0	10/5
66-67	13/10	18/10	10	-25,2	5/5
67-68	8/10	18/10	17	-35,2	15/5
68-69	23/9	10/11	19	-30,5	30/4
69-70	23/10	1/11			
Median	13/10	1/11	17	-29,3	5/5

a - first day of frost
b - ice formation
c - number of pentades with mean temperature colder than -10°C
d - min. mean temperature °C
e - last day of frost

Graphical illustration of pentade means of temperature and pentade sums of precipitation for the 3 last winters are given in fig. 2

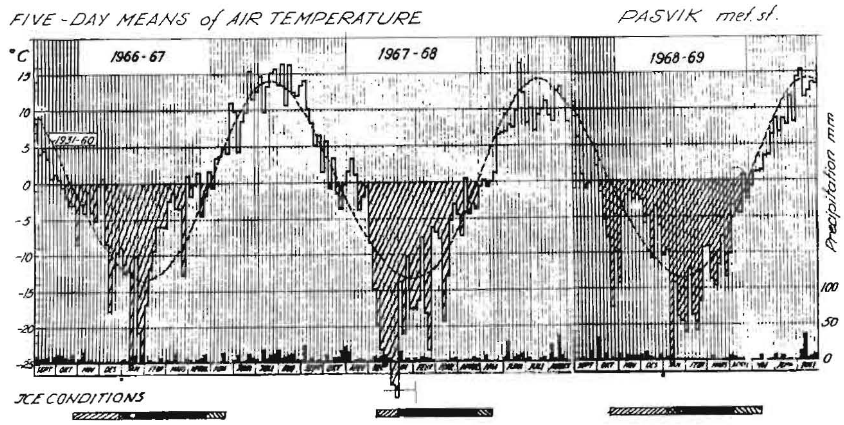


Fig. 2 Five day means of air temperature at Pasvik.

2. ICE CONDITIONS BEFORE WINTER 1967-68

In autumn, after the river temperature has fallen to 0°C, shore ice appears along the river banks. In quiet sections of the river this shore ice grows from both banks out into the middle of the stream, unites and forms sheet-ice. The river is divided into open and frozen sections.

In the open parts of the river ice formation is much more complicated. The supercooling of a free water surface is one of the most important factors, but there are also critical values of water velocity and water temperature which are of importance for the formation of ice cover in rapids and waterfalls.

When the turbulent water stream has been cooled down to 0°C, the heat loss from a free water surface will cause a supercooling of a very thin water film which incessantly is being renewed during the turbulent motion of the water masses. A supercooled element of the surface film will move in an irregular way through the water and may just as often sweep the bottom as be moving along the surface. On the way down the supercooling of the element will decrease and a slight supercooling of the surrounding water will result. The consequence of this is formation of frazil and bottom ice (underwater ice). Frazil ice may not only be floating in the surface stream. In turbulent water frazil ice may be carried with the water stream in any depth of the river. This combination of frazil ice floating in slightly supercooled water with incessantly newformed supercooled water film elements whirling down and gradually "dying" represents a most potent factor in the ice formation in rapid rivers. It may be called "active frazil ice", in contrast to frazil ice floating in water which is not supercooled and might be classified as "passive frazil ice".

Sketch fig. 3a illustrates the formation of an ice dam in a rapid.

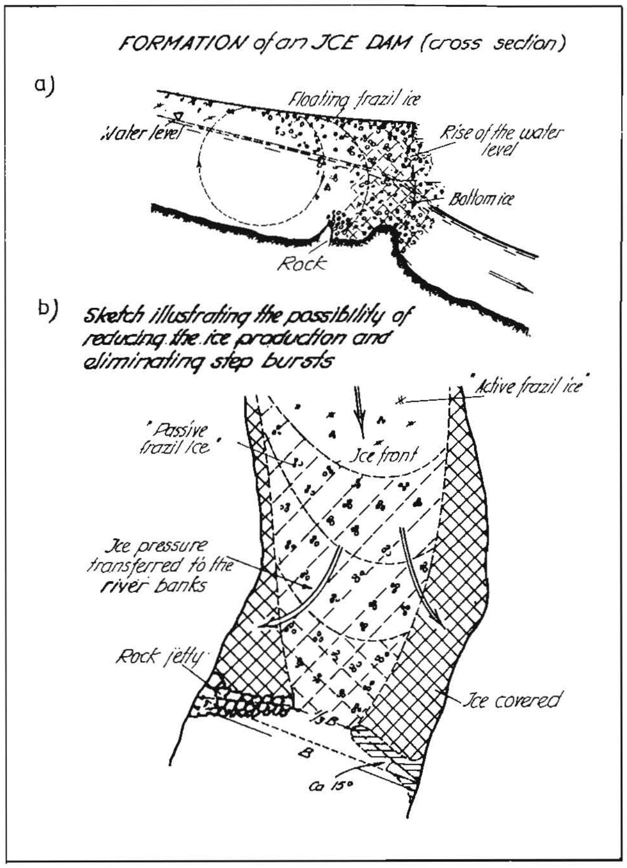


Fig. 3 Formation of ice dams.

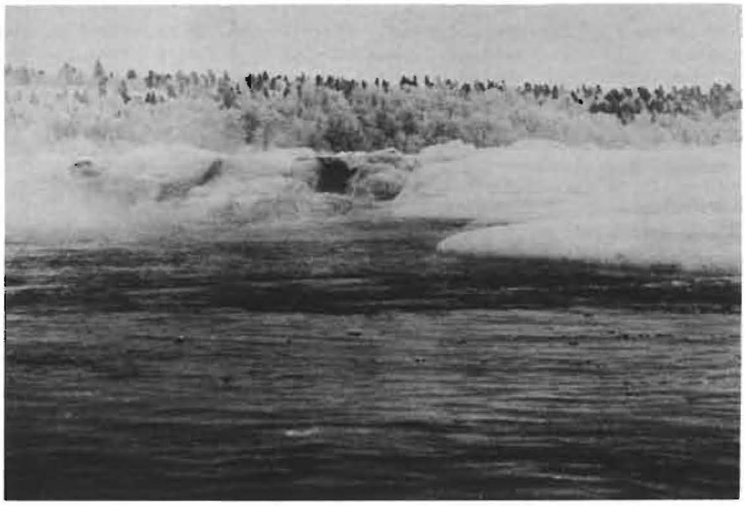


Fig. 4 Ice Dam in Pasvik River at Hestefoss January 1967
Damming height about 3 m.

Fig. 5 Accumulation of pack-ice at Bailey bridge.
About 60 cm thick ice blocks.

Fig. 4 shows a typical ice dam built up by the formation of bottom and frazil ice in Pasvik river.

The bottom ice is very loose, but the dam reduces the water velocity in the basin behind it and a cover of ice may be formed. This stops the supercooling and dynamic ice formation. If the development goes on without interruption, the flow will concentrate in a narrow main channel. The supply of heat from the fall energy is a contributing cause to this. In this way nature stabilizes the ice conditions in rapids.
A bottom ice dam is most often poorly fastened to the river bed. If some changes occur, for example variation of water flow or weather changes, such an ice dam may burst and start an ice-drift. Such a winter ice flow will bring serious trouble.
Fig. 5 shows an accumulation of pack-ice masses at a Bailey bridge after an ice flow in Hestefoss on the 16. January 1967.

3. PRESENT EXPERIENCE ON REDUCING THE ICE TROUBLE

The production of frazil and bottom ice is directly proportional to the cooling surface i.e. the area of open water.

The heat balance of ice formation in rivers, after Devik, is presented by the following table.

Heat loss from a water surface of 0°C measured in kcal/daa, sec, by clear sky, N=0

t_a °C / V m/s	0	-1	-5	-10	-15	-20	-25
1	27	30	43	57	70	82	94
5	37	46	83	124	162	197	232
10	47	64	125	195	260	321	381
15	57	79	164	261	350	434	516
20	66	95	202	324	436	543	647
25	75	110	238	383	518	645	769
30	83	125	272	431	597	745	889

Correction table for claudiness, scale 0-1

N	0,1	0,2	0,3	0,4	0,5	0,6	0,7	0,8	0,9	1,0
Corr	0	-1	-2	-3	-5	-7	-9	-12	-15	-18

Humidity = 75 % Global radiation = 0 V = wind velocity in 2m height

The most important tasks will be to prevent and reduce such open water surface and production of underwater ice.

By discussion with K/S, A/S Norelektro & Co and firma Pasvikkraft the following programme has been accomplished for this problem:

1. To lay out timber booms and booms made of plastic tube, just above the first rapid of Hestefoss, increasing shore ice production.
2. Narrowing the river at Hestefoss rapids with stone fences (groins), see fig. 3b, to develop the basis for artificial ice bridges and stabilize ice dams.

a. Ice conditions during the winter 1967-68, fig. 6

The formation of ice went on very propitiously and by the end of December the open part of the river was considerably reduced.

On the 2. January a very strong frazil and bottom ice production was observed. The water level at the upper groins had risen about 1.5 m.

On the 4. January air temperature was -36 °C and a dense curtain of frost smoke covered the bottom of the valley. The fog subdued the cooling of the water very effectively.

The 7. January air temperature was -27 °C and a very strong ice production on the open river sections was observed. The water flowed over the groins.

On the 9 of January ice conditions had stabilized.

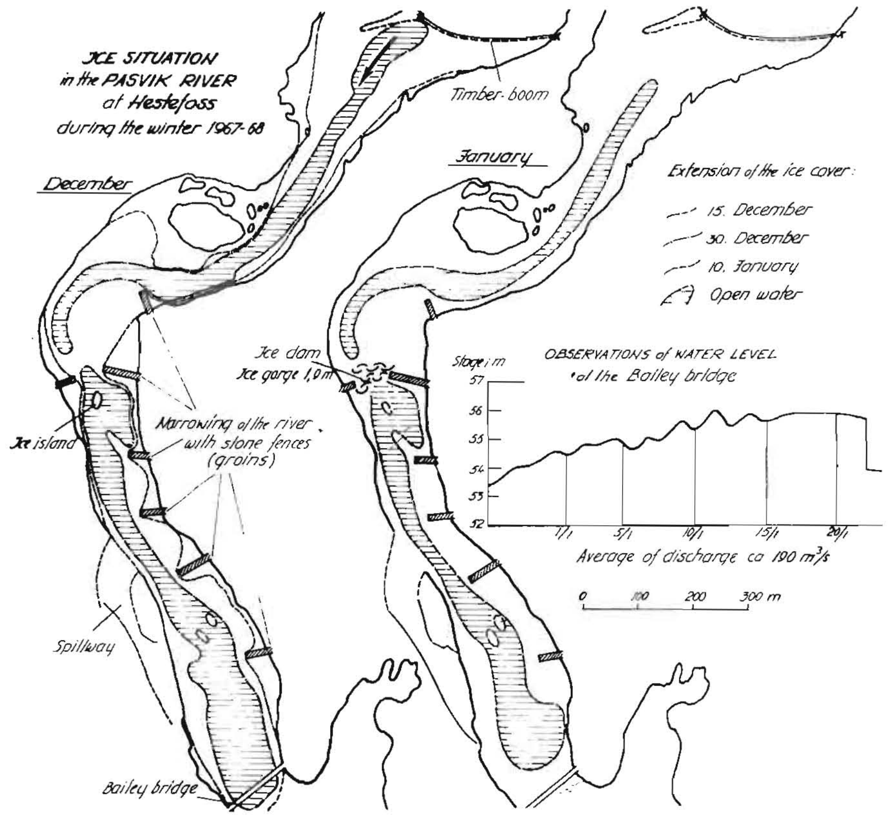


Fig. 6 Ice conditions in the Pasvik River at Hestefoss during the winter 1967-68.

The dams were rock jetties, for practical reasons build on shallow parts of the Norwegian river bank. They were ca. 4 m wide, and extended ca. 1,2 m above water level.

b. Ice conditions during the winter 1968-69, fig. 7

The discharge was held continously betwgen 153-158 m^3/s until the middle of February, afterwards it increased to 165 m^3/s.

The booms were laid out 17. and 18. October. The main boom above the flood-gate was laid out 11. November. The plastic tubes were separated in water-proofed bulkheads by joining them with plugs of wood in 3-4 places.

Stone fences (groins) as before.

The upper booms above the rapids worked rather well, especially the first one. Already in November most of the river above the booms was ice covered.

The boom above the flood-gate had to be strenghtened. The formation of ice first began from the left side. Water velocity was about 1 m/s and development of sheet ice was stopped. At the stone fences in the rapids a stable ice dam was formed which endured through the whole winter.

In the course of January an ice dam, about 3 m, grew up at the entrance of the flood-gate, see rise in water level at the water gauge 1, fig. 7.

Fig. 8 shows the arrangement of the plastic-boom installed before the spillway.

Booms made of plastic are cheaper and easier to lay out. The disadvantage is that the plastic-tube is very smooth. It is advantageous to twist hemprope round the tubes and if necessary, pierce twigs in the twisting.

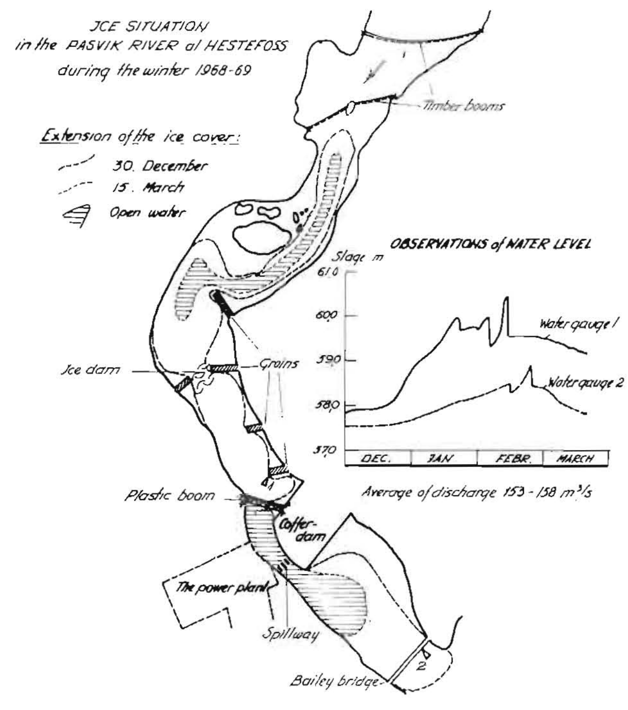


Fig. 7 Ice conditions in the Pasvik River at Hestefoss during the winter 1968-69.

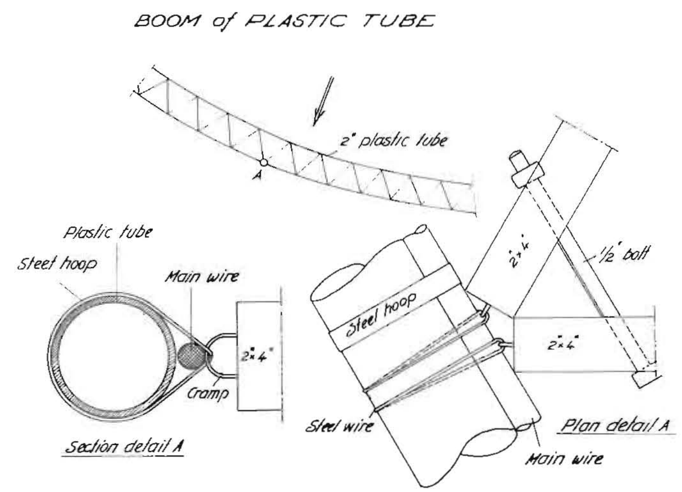


Fig. 8 Details of plastic tube boom arrangement.

4. CONCLUSIONS

Some basic principles will clearly emerge of the nature of ice formation in running water.

The production of ice is directly proportional to the cooling surface - the area of the open river. The most important preventing work which must be done is to reduce production of frazil and bottom ice. It might be advisable to accelerate the formation of an ice-cover by use of different booms or groins as mentioned before. An ice layer may then start growing upstream or the groins may initiate formation of steady ice dams in rapids, secure slow water flow and promote the formation of an ice cover.

To establish reservoirs in the river can stop the step bursts and prevent the formation of ice bridges at critical places.

Our experience shows that the time necessarry to build up an ice-dam fairly easy may be decreased by artificial means. To keep the dams in place during winter is more complicated. In cold stabile winters nature herself helps with this work. In unstable winters flooding may break down the ice-dams.

Discussion by CARSTENS

on paper by KANAVIN 4 - 7

Mr. Kanavin's paper describes a new and extremely interesting type of ice control structure. An iceproducing rapids is converted into a series of ice-covered pools by means of constrictions. These constrictions are inexpensive rock jetties that extend about one third of the width of the river from each bank, leaving a gap in the middle. Here the flow velocities are high enough for anchor ice to form, and so an ice dam builds up from the bed.

In the stable winter climate of the Pasvik River this combination of man-made and natural dam has proved sufficiently strong to survice two winter seasons. The jetties were slightly damaged by the spring floods, but were easily repaired.

ICE SYMPOSIUM 1970
REYKJAVIK

FORMATION OF ICE SHEET IN DOWNSTREAM POOLS OF HYDROELECTRIC STATIONS AND ITS CONTROL

FORMATION DES GLACES À L'AVAL DES AMÉNAGEMENTS HYDROÉLECTRIQUES ET CONTRÔLE DE CELLES-CI

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On rivers in areas with severe climatic conditions great quantities of frazil occur in the autumn and in the winter, this frazil contracting the river cross-section when accumulating beneath the ice cover. Ice jams are formed in the spring after the ice break-up on rivers with thick ice cover causing floods.

Large storage reservoirs of hydroelectric stations and rational flow control during rivers freeze-up and ice break-up influence radically the thermal and ice conditions of rivers downstream of hydroelectric stations and diminish adverse effects of ice which can occure on non-regulated rivers.

Dans les cours d'eau de régions a climat rude, on observe, en automne et en hiver, de grandes masses de glace de fond, lesquelles, s'accumulant sous la glace de couverture, font réduire la section hydraulique. Au printemps, dans les cours d'eau à couverture de glace épaisse, se forment, lors de débâcles, des amoncellements de glaces provoquant des inondations.

La création de grandes retenues auprès d'usines hydroélectriques et l'utilisation rationelle des débits de cours d'eau en temps de gelées et de débâcles, font changer radicalement le régime thermique et celui de glaces de cours d'eau à l'aval des ouvrages et réduire l'effet défavorable des glaces observé sur des cours d'eau à écoulement naturel.

The hydropower development in the east and north-east regions of the Soviet Union, where the rivers are characterized by a complicated ice regime, necessitates widening the scope of ice thermal studies.

Creation of reservoirs of the hydroelectric stations essentially changed the thermal conditions in the downstream pools and, hence, the ice regime of river flows downstream of the structures.

Under natural-flow conditions the ice sheet on the rivers is formed either due to gradual widening of shore ice and its further jointing or due to advancing the ice edge upstream of the ice bar formed in a certain place.

Complete freezing due to jointing the shore ice strips is usually observed on lowland rivers and takes place on large sections of the river during a short period of time, especially when the air temperature lowers considerably.

Long rivers with abundant flow and a considerable heat reserve are freezing usually due to advancing the ice edge upstream. Especially this is characteristic of effluent rivers. The most of these rivers are characteristic for frazil ice formation that makes their ice conditions more complicated.(I)

The construction of hydroelectric stations on the rivers and creation of reservoirs results in changing the downstream pool conditions which become similar to those of effluent rivers. In this case a large frazil-ice-forming stretch of the river is cut off and the frazil ice and ice in full or in part are accumulated in the reservoir. In the close proximity of the hydroelectric station an air hole in the ice cover is formed which does not freeze over during the whole winter period and which area is conditioned by the depth of the reservoir, its thermal conditions and hydraulic conditions in the downstream pool. The ice conditions of the lower reaches of the river change respectively.

With air temperature below zero and increasing heat emission from the water surface a zone of zero temperature establishes at some distance from the hydroelectric project. Downstream of this boundary frazil ice usually originates. Flowing downstream and increasing in volume this frazil ice produces the material for advancing the ice edge upstream.

During the winter period, as the water mass in the storage reservoirs becomes cool, the boundary of zero temperatures and the ice edge are approaching the hydroelectric station until the heat inflow from the reservoir exceeds the heat emission from the water surface in the open part of the downstream pool.

As the ice edge is advancing upstream, a part of frazil ice masses moving from the upper reaches of the river is carried away underneath the ice cover with result that the water cross-section is plugged with frazil ice, the roughness coefficient of the lower surface of ice increases, the backwater from ice in front of the ice edge is created and the water level rises.

The larger the water discharge in the river, the slopes of the water surface and flow velocity, the higher the water level. This rise of the water level can be also observed on non-regulated rivers. So, on the Angara River near the town of Irkutsk 5 m rise of water level was observed runder natural flow conditions during complete freezing.

Especially large rises of water level caused by frazil ice can be observed downstream of the rapids where the water fall is highly concentrated.

On the Angara River, for example, downstream of the Padun Rapid, the water level after complete freezing was rising up to 7 m, and downstream of the Yershov Rapid - up to 9 m.

Small rises of water level were observed on the Volga, the Nieman, the Daugava and other rivers.

Quite different situation is created in the downstream pools after hydroelectric stations are constructed on the rivers.

Due to seasonal run-off control the discharges increase considerably that causes the increase of the stream velocities, the increase of the stream turbulence and formation of a large amount of frazil ice and, in case of complete freezing, the increase of the amount of frazil ice in the river channel and subsequent rises of water level.

This is proved by the results of field observation carried out in the downstream pools of a number of hydroelectric stations.

On the Angara River, in particular, a I2 m rise of water level was recorded downstream of the Yershov Rapid when the complete freezing was set-in under the unfavourable temperature conditions (air temperature rise), when the "frazil ice's rises" of level are usually higher due to weak regulation of frazil ice masses.

In case of higher discharge the value of the frazil ice's rise of level can be oalculated by the formula (2,3) derived on the assumption of equal disoharges in the open channel Q_s = at Z_s and during freez-up Q_w = at Z_w = $Z_s + \Delta Z$

$$\Delta Z = H_{summer}\left\{\left(\frac{n_{channel\,2}}{n_{channel\,1}}\right)^{3/5}\left[\left(\frac{n_{ice}}{n_{channel}}\right)+1\right]^{2/5}\left(\frac{J_{summer}}{J_{winter}}\right)^{3/10}\left(\frac{B_{summer\,1}}{B_{winter}}\right)^{3/5}-\frac{B_{summer}}{B_{winter}}\right\}+h_{ice},$$

where ΔZ	– the value of "frazil ice's rise" of level	with open river channel and water level = = Z summer.
$H_{summer\ I}$	– mean depth	
$H_{channel\ I}$	– river channel roughness coefficient	
$J_{summer\ I}$	– water surface slope	
$B_{summer\ I}$	– river width	
$n_{channel\ 2}$	– river ohannel roughness coefficient at Z_{winter}	
B_{winter}	– river width	with complete freezing and water level in the ice well Z_{winter}
J_{winter}	– slope	
n_{ice}	– roughness coefficient of the ice lower surface	

This formula, checked at a number of hydroelectric stations, gave satisfactory results. The above in view, when planning the operation of a hydroelectric station provision should be made for the creating the optimum conditions of releases to the downstream pool.

In case ice formations appear on the river in autumn it is expedient (to an extent the power consumers permit it) to decrease releases to the downstream poll. It will decrease the flow velocities and frazil ice formation and accelerate the complete freezing. Owing to less amount of frazil ice in the river channel and smaller thickness of the ice cover, the downstream water levels will be lower resulting in a less deoreasing of the head at the hydroelectric station. As the ice cover of the river becomes stronger, the discharges of releases can be increased little by little up to normal ones.

Wide variations of discharges through a hydroelectric station with daily pondage adversely affect the ice conditions as they can cause breaking-up the ice cover, ice movement, plugging the river channel with ice and, as a result, decreasing the head.

Following these advices, it is possible to create the safe conditions for the downstream strechts.

With spring coming, when the heat emission from the water surface becomes less than the heat inflow from the storage reservoir, the boundary of zero temperatures and the ice edge begin to move downstream of the structure. The larger the heat inflow, i.e. the larger the water discharges, the faster this moving. Therefore, the water releases to the downstream pool can be increased to drive away the ice edge as fast as possible. The releases are to be increased within reasonable limits to keep from a sudden rise of the water level under the ice cover.

Formation of air holes in the ice cover in the downstream pools of hydroelectric stations during the winter period and rapid retreat of the ice edge from structures prior to the beginning of a spring flood are the positive factors which better the ice conditions in the downstream pools.

Due to these factors ice difficulties have been eliminated on the Angara River near Irkutsk after the Irkutsk Hydroelectric Station was constructed; on the Nieman River near the town of Kaunass after the construction of the Kaunass Hydroelectric Station; on the Daugava near the town of Yaunelgava and in the Kegums Reservoir after the construction of the Plyavinas Hydroelectric Station and on the other rivers.

Construction of hydroelectric stations with reservoirs of large storage capacities is a very essential engineering measure minimizing the damages caused by the unfavourable ice conditions provided that the run-off and the ice thermal processes are controlled judiciously.

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ICE SYMPOSIUM 1970
REYKJAVIK

ICE CONDITIONS IN RESERVOIRS OF PUMPED--STORAGE POWER PLANTS

RÉGIMES DES GLACES DES RÉSERVOIRES DES USINES D'ACCUMULATION

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Synopsis

The paper deals with the investigations carried out at the VNIIG to elucidate ice conditions characteristic for the upper and lower reservoirs, the inlet channel, the intake and the pressure galleries of pumped-storage power plants operating under severe climatic conditions. The scope of studies and experimental procedures are described. Some investigation results are given for the cases of the Kiev and Zagorsk pumped-storage power plants.

Résumé

Le rapport concerne les recherches du VNIIG ayant pour but d'étudier les particularités du régime des glaçes des réservoirs supérieur et inférieur, du canal d'amenée, des prises d'eau et des conduites sous pression des usines d'accumulation en exploitation dans les conditions climatiques sévères. On décrit les essais et les méthodes employées: quelques résultats obtenus pour les cas des usines d'accumulation de Kiev et Zagorsk sont donnés.

In connection with designing and constructing pumped-storage power plants in the U.S.S.R. where climatic conditions are varied and rather rigorous, the VNIIG is carrying out research into the ice regime of the reservoirs of this type of power plants.

Wide and abrupt daily variations in the water level in the reservoirs as well as reversion of the direction of water flow through the waterway system are responsible for specific ice conditions in the upper and lower reservoirs and over the whole waterway in the vicinity of a pumped-storage power plant.

Information on ice regime in the reservoirs of pumped-storage power plants is rather scarce /R.1/. Therefore the VNIIG and some other research bodies /R.2/ are engaged in studying ice conditions characteristic for the upper and lower reservoirs and for the whole waterway system of pumped-storage power plants as well as in predicting ice troubles which may occur near these power plants.

At the VNIIG the investigations are conducted:

a) on models in the Laboratory;

b) in an experimental basin at an open testing area;

c) downstream from some hydroelectric power plants with daily variations in the water level from 3 to 4 m;

d) by calculations and theoretical studies using coefficients obtained from laboratory and field investigation results.

Laboratory studies on the ice regime in such reservoirs were performed in a model basin (scaled 1 : 60 vertically) of the Kiev pumped-storage power plant /R.3/. The variation in the water level of the basin during its drawdown and filling is 6 m. The internal slope 1 : 4.5 of the basin is protected with reinforced concrete slabs. The internal slopes 1 : 5 and 1 : 7 of the inlet channel are also lined with reinforced concrete slabs.

The strength of ice used in the experiments was close to that of model ice. A low-strength ice cover was obtained by altering the salinity of the water supplied to the model /R.4/. The drawdown and filling regime in the experimental basin corresponded to the operating conditions of the prototype power plant.

Outdoor tests were carried out in a rectangular experimental basin 10 x 15 x 5 m. One of the sides of the basin with a slope of 1 : 3 was lined with reinforced concrete slabs. The depth of loss or gain in storage was 1.75 - 2.0m, which corresponded to 1/3 - 1/4 of the actual value of drawdown at the Zagorsk pumped-storage power plant.

The ice cover on the water surface of the experimental basin formed under natural conditions. The thickness of the ice cover formed on the surface of the basin during one operating cycle varied from 0.5 to 7.0 cm.

The drawdown and filling regime of the experimental basin corresponded to the operating conditions of the Zagorsk power plant.

Field investigations associated with the problem in hand were carried out downstream from a hydroelectric power plant in a ship channel and near the river banks downstream from this channel.

The bank of the channel (1 : 3) had a reinforced concrete lining. The bank 1 : 3 and 1 : 5 of the adjacent stretch of the river was covered with crushed stone. Downstream from this stretch where the river banks were unprotected the steepness of the slopes varied from 1 : 2 to 1 : 7.

The variation in the downstream water level of this hydroelectric power plant reached 3 m in 24 hours.

The studies performed resulted in establishing the following:

1.The ice cover in a reservoir, except the strip along the slopes, will follow the water surface during the drawdown and filling of the reservoir.

2.The ice cover formed during each cycle at the reservoir surface in the strip near the slopes will break up and deposit on the slope, as the water level drops, and will eventually freeze to the slope.

Drawdown-filling cycles in winter will result in freezing-over of the slope, the thickness of the ice layer being likely to reach 4-5 m or more upon a long period of frost and with a large depth of drawdown. The porosity of such ice accumulations is 0.3-0.4, the average value being about 0.2. The porosity of ice depends to some extent on the depth of the layer of broken ice lying on the slope.

3.During winter thaws and in spring ice masses will slip from the slope, if the concrete slope is steeper than 1 : 3.5 - 1 : 3.

4.Heavy wind waves on the open water surface and frosts (autumn-winter transition) may cause supercooling of water and formation in the reservoir of frazil ice which may penetrate the intakes.

5.In inlet and outlet channels with flow velocities exceeding 0.5 m/sec broken ice may move into the intake not only after the ice cover is broken in the channel itself, but also in the portion of the reservoir adjacent to the channel.

6.If metal pressure pipelines are not heat-insulated, ice may form on the internal surfaces of the pipes.

7.Periodical wetting of metal and other surfaces of the structures within the zone of varying water level enhances the probability of their freezing-over. In designing pumped-storage power plants to be erected in the areas with severe climatic conditions major emphasis must be placed on the investigation of ice regime in the reservoirs and over the whole waterway system of pumped-storage power plants as well as on the development of adequate measures ensuring trouble-free operation of the power plant in winter.

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ICE SYMPOSIUM 1970
REYKJAVIK

CALCULATION OF FLOW VELOCITY IN THE OPERATIVE ZONE OF PNEUMATIC INSTALLATIONS USED FOR ICE-FIGHTING

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In this work are given analytical dependences for calculating the velocities of circulating flows originating in the basins with stagnant waters due to the working pneumatic installations.

Dans le rapport on détermine les dépendances analytiques pour le calcul des vitesses des courants circulaires, dans les reservoirs d'eau non courante pendant le travail des installations pneumatiques.

In basins with stagnant of nearly stagnant waters pneumatic installations are applied for ice-fighting using the deepwater heat. In operation of such installations air bubbles drag along the surrounding liquid. Suction of water takes place in the near-bottom region while at the surface the water jets spread out forming closed circulatory currents. The resulting flows can be considered either three-dimensional or two-dimensional, depending on the installation design. For determination of basic performance characteristics of the installations (rate of air flow, nozzle-to-nozzle of pipe-to-pipe distance, etc.) it is necessary to ascertain the velocity field in the circulation zone. The given pertains to the latter problem.

Analysis of the experimental data shows the advisability of dividing the entire flow zone into three regions (Fig.1):

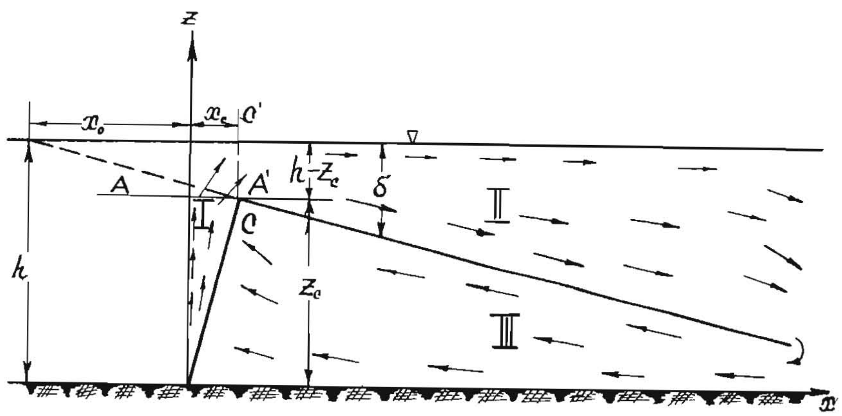

Fig 1.

Fig.1. - Flow pattern in the zone of operation of a pneumatic installation.

I - region of ascending flow where water lifted by air bubbles; *II* - region of spreading of jets; at the subsurface region the water lifted to a hight of about 0,8h , spreads outwardly from the vertical axis; *III* - region of the counter flow resulting from suction of water to the air discharge region.

Region I: By its properties and structure the upward flow of air-water mixture is similar to a free turbulents flow, or stream. Its peculiarity is in that the velocity along the axis over a major part of travel remains reasonably constant which is different from the case of ordinary jets.

The following formula have been derived for velocities in an air-water jet, based on the second pattern of L.Prandtle and along the lines of the theory of water ascension caused by air bubbles, developed by I.M.Konovalov:

for a two-dimensional:

$$W_m = 1{,}38 \sqrt[3]{\frac{q_o g (2P_o + \gamma h)}{P_o}} \tag{1}$$

$$\frac{W}{W_m} = e^{-1{,}07 \cdot \frac{\sigma^2 x^2}{z^2}} \tag{2}$$

for an axisymmetrical jet:

$$W_m = 2{,}27 \sqrt[3]{\frac{q_o g (2P_o + \gamma h)}{P_o h}} \tag{3}$$

$$\frac{W}{W_m} = e^{-0{,}517 \cdot \frac{\sigma^2 x^2}{z^2}} \tag{4}$$

where: W is the velocity of flow at any given point of a jet, W_m is the velocity at the jet axis. "q_o" is the rate of air flow at the initial cross-section; "P_o" - pressure at the water surface (equal to the atmospheric pressure); "h" is the depth; "γ" -specific gravity of water; and "σ" is non-dimensional empirical constant (σ = 6.55 for a two-dimensional jet and σ = 12.2 for an axisymmetrical jet).

Region II: Experiments show that the flow in Region II can be schematically represented in the form of a two-dimensional, fan-shaped semi-jet spreading out from an imaginary source and being delimited, in the plane of symmetry, by a free surface. In Fig. 2 the general profile of a fan-shaped jet is matched against experimental data, for illustrative purposes. In order to determine the flow velocities one can, therefore, make use of the turbulent flow theory, bearing in mind, however, that the values of the experimental constants in respective formula would be different. If, for instsnce, the Guertler's formula

$$\frac{u}{u_m} = 1 - th^2 \xi \quad \text{, where} \quad \xi = \frac{\sigma (h - z)}{x}$$

is used for determination of the horizontal velocity components, then for a two-dimensional jet σ = 8.2 and for a fan-shaped jet σ =14.2. In the result of the experimental datum processing the following values of the ratio of width of the initial region of jets $h - z_c$ to the total depth h, were obtained:

for a two-dimensional jet: $h - z_c = 0{,}22h$;

for a fan-shaped jet: $h - z_c = 0{,}17h$.

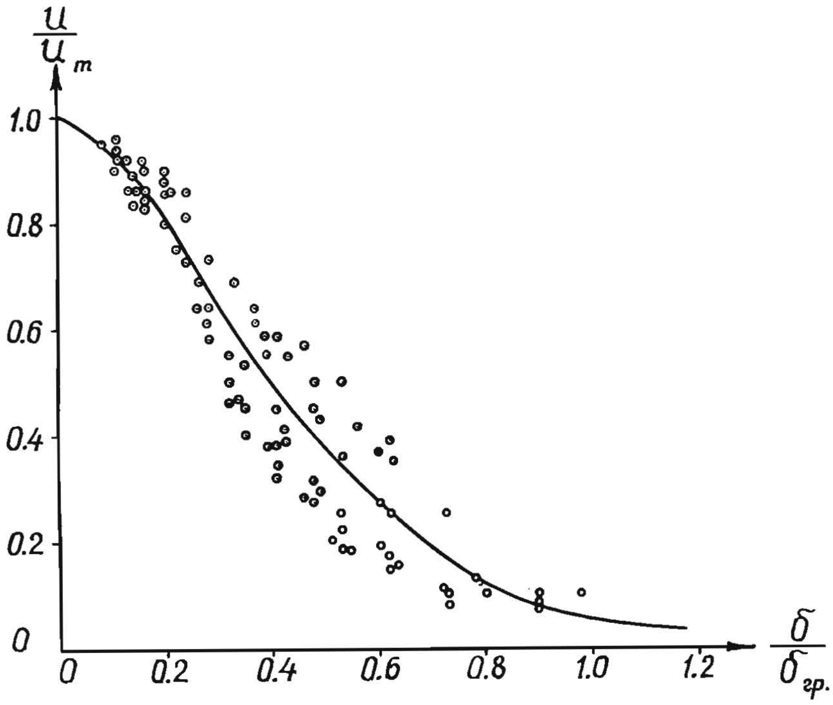

Fig.2. - General profile of a fan-shaped jet.

—— theoretical curve ;

⊙ experimental data.

The following expressions were obtained for the axial velocity of U_m horizontal jets, based on the turbulent flow theory and on the above experimental data:

in the case of two-dimensional jet:

$$U_m = 0{,}92\, U_{m_c} \cdot \sqrt{\frac{h}{x - x_o}} \qquad (5)$$

$$x_o = -0{,}64h$$

in the case of fan-shaped jet:

$$U_m = 1{,}14\, U_{m_c} \frac{h}{x - x_o} \qquad (6)$$

$$x_o = -0{,}97h$$

Here "u_{m_c}" is the flow velocity at the jet axis and at the initial cross-section. One can calculate the values of u_{m_c} if one assumes that the overall rate of flow of a vertical jet equals, in section A-A', the rate of flow of a horizontal jet. Then, using an equation $Q_{верт.с.} = Q_{гор.с.}$, one finally obtains:

for a two-dimensional jet:

$$u_m = 0{,}88\, W_m \cdot \sqrt{\frac{h}{0{,}64h + x}} \qquad (7)$$

for a fan-shaped jet:

$$u_m = 0{,}39\, W_m \cdot \frac{h}{0{,}97h + x} \qquad (8)$$

The above relations hold for a distance of only about (3 to 4)h and then the velocity values decrease abruptly with the resultant decay of jets.

<u>Region III</u>. Usually it is sufficient to know the mean velocities in the given region. Since the rates of flow in Regions II and III are equal and the cross-section area of Region III is $\omega' = \Omega - \omega$ (here, Ω and ω are the cross-section areas of the entire circulation zone and of Region III, respectively), the expressions for mean velocities are, then:

in the case of two-dimensional flow:

$$u'_{cp.} = 0{,}82\, W_m \cdot \frac{\sqrt{h(0{,}64h + x)}}{3{,}2h - x}$$

in the case of three-dimensional flow:

$$u'_{cp.} = 0{,}18\, W_m \cdot \frac{h}{5{,}7h - x}$$

The derived formula can be used for calculating velocities versus the rate of air flow and the circulation zone depth. Comparison of the theoretical and experimental data proved to be satisfactory.

As distinct from ordinary turbulent streams described by Turner, Taylor and others, we consider in this article the water ascension caused by air bubbles. In this case the velocity along the axis of stream remains approximately constant. This is confirmed by the data of laboratory and prototipe experiments.

ICE SYMPOSIUM 1970
REYKJAVIK

AN EXPERIMENTAL STUDY ON PREVENTION OF ICE-COVER FORMATION IN RESERVOIRS BY MEANS OF AIR-BLOW METHOD

ÉTUDES EXPERIMENTALES SUR LES METHODES DESTINEES A EMPÊCHER LA SURFACE DE L'EAU DE SE GELER DANS LES RÉSERVOIRS AU MOYEN DE JETS D'AIR

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Eiichi Koike, Civil Engineer   Civil Engineering Department   Japan

In the instance of ice-covered reservoirs in wintertime, it is necessary to keep the upstream side of the spillway gates in an ice-free state so as to keep them functional in case of a flood emergency, and a useful measure to this end is to blow air into the water so as to prevent the surface from forming an ice-cover. This paper describes the results of the experiments conducted at a few reservoirs of medium or small scale concerning the relation among air temperature, blow air volume and ice-free area width; it forms the first of a series of experiments to be conducted in the future to establish a general rule in this respect.

Dans un réservoir dont la surface gèle en hiver, il est nécessaire de garder libre la surface de retenue, côté amont des vannes des evacuateurs de crues pour maintenir leurs fonctionnements en cas de crues, et un moyen très efficace consiste à souffler de l'air dan l'eau afin d'empêcher la surface de geler. Dans la présente note, les auteurs traitent de quelques résultats d'expérience sur des réservoirs de capacité petite et moyenne donnants les relations entre la température atomosphérique, le volume d'air soufflé et la largeur de la surface d'eau fondue. Ces expériences sont les premières d'une série d'essais prévus et destinés à établir une loi génerale dans ce domain.

It is generally supposed that the width of the ice-free area in reservoirs is related to many factors such as the volume of the air blown into the water, water temprature, air temperature, wind, earth temperature and so on, but the authors took air temperature and the volume of air blow as the most dominating factors, and during last winter (1969-70) conducted <u>in situ</u> experiments in the hydropower reservoirs at Iwashimizu, Iwamatsu and Iwachishi, Hokkaido, Japan.

After a series of expriments, it has been concluded that the air blow of 100 - 145 litres per minute will suffice to keep the upstrem side of the spillway gates in an ice-free state at these reservoirs, which is believed reasonably economical for the purpose.

Spillway Gates at the Iwachishi Reservoir.

1. <u>Measurement at the Iwachishi Reservoir</u>.

The Authors conducted experiments on the relation between air volume and ice-free area width at the spillway gates of the Iwachishi Reservoir, the results being as shown in Fig. 1. The volume of air blown into the water was at first 300 l/min. per gate, then was gradually diminished to 100 l/min.

The test was carried out at gates No.2 and No.3, each 10.8 m wide and 9.1 m high. Air was blown in at the depth of 8.8 m, when the water temperature ranged $+0.4°C \sim +0.6°C$ at the depth of 9m.
The maximum ice-cover was 0.6 m thick before the test. Air was supplied finally at the rate of 145 l/min. per gate (13.4 l/min. per metre of gate width), and at 100 l/min. per gate (9.3 l/min./m).

When two curves of air temperature and ice-free area width are given, the cross-correlative function between them is shown in the form of :

$$R_{yx}(\tau) = \lim_{T \to \infty} \frac{1}{T} \int_0^T x(t) \cdot y(t+\tau)\, dt$$

where $R_{yx}(\tau)$: cross-correlative function between the variants $x(t)$ and $y(t)$,

$x(t)$: irregular curve of air temperature ---- the variant at time (t),

$y(t+\tau)$: irregular curve of ice-free area width --- the variant at the point apart from time (t) by τ

T : total time of observation.

The results of the correlation computations are given in Fig. 1. From Fig. 1 is seen that the cross-correlative function for gate No.3 is greater than that for gate No.2, and this may be explained by the smaller supply of air at gate No.3.

2. Measurement at the Iwamatsu Reservoir.

The same cross-correlation functions were obtained by the experiment at Iwamatsu, another reservoir under different climatic conditions. The gates were all of the same dimensions : 8 m wide and 6.2 m high. Air blow into the water was set at 6 m below water surface, water temperature was +0.3°C~+0.6°C at the -6 m depth, and the maximum thickness of ice-cover was 0.55 m before air blow. The supply of air blow was chosen at 9.9 l/min. per metre of gate width.

In Fig. 2 is shown the correlation obtained by the experiment, all figures in the chart are expressed as the average for seven gates.

From the results of the experiment we learned that

(1). Air was supplied at the same rate as at Iwachishi, but the ice-free width decreased by one metre.

(2). A time lag of about three days is seen between the peak of air temperature and that of the ice-free area width.

(3). The curve shapes of air temperature and ice-free area width are not similar; this may be explained by the influence of climatic characteristics at the reservoir, such as cumulative air temperature and water temperature.

3. Measurement at the Iwashimizu Reservoir.

Special measurements were conducted at another reservoir at Iwashimizu, chiefly to clarify the water temperature distribution before and after the air blow, the results being as shown in Fig. 3. From the measurements were learned the following facts:

1). A temperature rise of 0.3°C was measured on the water surface after the air blow: this might show the upward movement of the lower water layer caused by the air blow.

The temperature went down after the stop of the air blow, and it went back to exactly the previous state by three hours after the stop of the air blow.

2). When the water temperature distribution diagrams before and after the air blow were overlapped, a turning point was perceived to exist nearly at the central part; temperature of the upper and lower water layers turned over at this point.

3). The water velocity caused by air blow was measured as shown in Fig.3(b). The average velocity was 0.27 m/sec. in the upward direction, while the surface average velocity was 0.17 m/sec. in the horizontal (upstream) direction.

Through these experimental measurements, many interesting facts were learned by the authors, who are to continue this experimental study in the coming winters at other reservoirs under a variety of conditions in order to establish a general quantitative rule on selecting an adequate air blow volume to prevent ice cover in reservoirs.

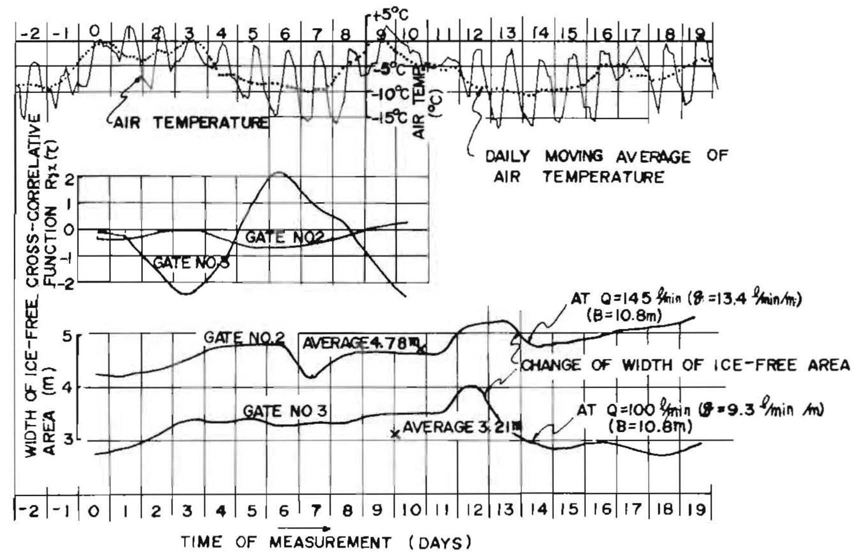


FIG.1 RELATION BETWEEN AIR TEMPERATURE AND ICE-FREE AREA WIDTH (IWACHISHI RESERVOIR)

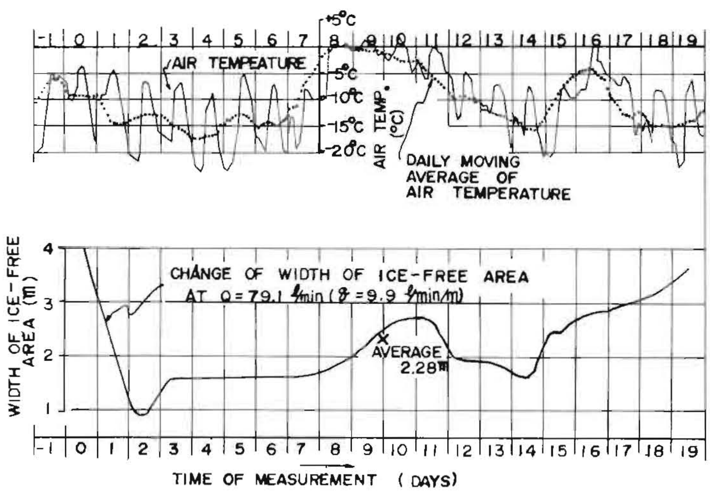
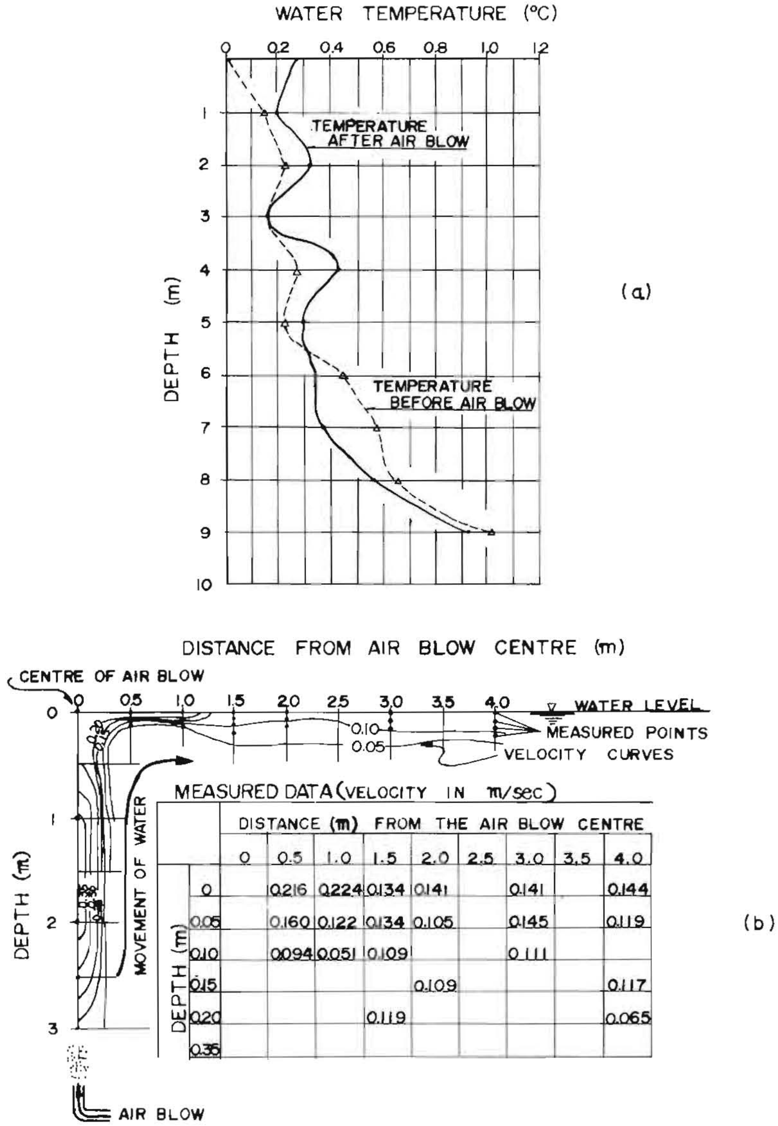


FIG. 2 RELATION BETWEEN AIR TEMPERATURE AND ICE-FREE AREA WIDTH (IWAMATSU RESERVOIR)



MEASURED DATA (VELOCITY IN m/sec)

DEPTH (m)	DISTANCE (m) FROM THE AIR BLOW CENTRE 0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
0		0.216	0.224	0.134	0.141		0.141		0.144
0.05		0.160	0.122	0.134	0.105		0.145		0.119
0.10		0.094	0.051	0.109			0.111		
0.15					0.109				0.117
0.20				0.119					0.065
0.35									

FIG.3 WATER TEMPERATURE DISTRIBUTION (IWASHIMIZU RESERVOIR)

ICE SYMPOSIUM 1970
REYKJAVIK

SCALE MODELS FOR INVESTIGATION OF ICE PHENOMENA

MODÈLES RÉDUITS DES ÉTUDES CONCERNANT LES PHÉNOMÈNES DE GLACE

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Research Institute for Water Resources
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Hydraulic model experiments may be resolved into two groups, using actual ice in a cooled hall or performed with ice substituents. In both cases difficulties are bound to arise in the determination of scales. The effect of distortion is investigated on the basis of the non-dimensional relation of Pariset, Hausser and Gagnon[1] concerning the stability of the ice cover. Distorted models do not yield numerically correct values.

Des études sur modèles réduits peuvent être effectuées: ou par l'utilisation de la glace naturelle dans un hall d'essai refroidi, ou bien par l'utilisation des substituants de glace. Des difficultés à surmonter s'élèvent dans tous les deux cas à la détermination des échelles du modèle. L'effet de la distorsion du modèle est étudié sur la base des rapports présentés sous forme adimensionnelle de Pariset, Hausser et Gagnon, concernant la stabilité de la couverture de glace. Les modèles distordus ne peuvent pas fournir des valeurs numériquement correctes.

Hydraulic experiments may be resolved into two broad groups, namely these using actual ice in a cooled hall and those performed with ice substituents.

In both cases difficulties are bound to arise in the determination of scales. Of course for studies into the process of ice formation, the use of natural ice and cooled water is inevitable. Suitable large laboratory installations are available at very few institutions only. Among these the ice laboratory at Leningrad should be mentioned first. Michel in Canada attempted to simulate natural conditions most closely by building the experimental flume outdoors for studying frazil ice formation. The small experimental flume at Trondheim has been installed at the refrigeration laboratory of the university. The recirculation of icy water and ice floes presents special difficulties. In the case of frazil ice even the heating of the recirculating ice floes special conveyer belts have been developed - mostly in the Soviet Union. Difficulties are also encountered in conserving the dimensions, and preventing damages of ice floes.

For this reason, in experiments where the objective is to study ice-run conditions, or to design structures for passing ice, artificial ice is used as for example by Z. Hanko in Hungary for the model studies on the Nagymaros Barrage.

When the ice cover is studied for stability and jamming, then in keeping with the equation of Pariset, Hausser and Gagnon, the identity of

$$\frac{Bv^2}{C^2H^2} \qquad / \text{ or the equivalent } \qquad \frac{Q^2}{BC^2H^4} \qquad /$$

must be ensured in the model and prototype. This implies that the scale factors are required to meet the condition

$$\frac{\lambda_B \lambda_v^2}{\lambda_C^2 \lambda_H^2} = 1$$

As is well known

$$\lambda_v = \lambda_C \lambda_R^{\frac{1}{2}} \lambda_y^{\frac{1}{2}} \lambda_x^{-\frac{1}{2}}$$

After rearrangement this becomes

$$\lambda_C = \frac{\lambda_v \lambda_x^{\frac{1}{2}}}{\lambda_R^{\frac{1}{2}} \lambda_y^{\frac{1}{2}}}$$

and further

$$\lambda_B = \lambda_x \qquad \text{and} \qquad \lambda_H = \lambda_y$$

where

λ_x is the horizontal, and

λ_y is the vertical scale factor.

Introducing these accordingly

$$\frac{\lambda_v^2 \lambda_x}{\lambda_y^2} \; \frac{\lambda_R \; \lambda_y}{\lambda_v^2 \; \lambda_x} = \frac{\lambda_R}{\lambda_y} = 1$$

The relationship is consequently invalid, unless the condition $\lambda_R = \lambda_y$ is satisfied, which in turn is impossible, unless the condition $\lambda_x = \lambda_y$ is also met, i.e., the model is an undistorted one[2]. No quantitative conclusions can thus be arrived at from tests in distorted models on the stability of ice covers and the formation of ice jams, or at least checks must be run to see whether the positions relative to the bell-curve of the model and prototype are identical, or not.

Ice jamming and the stability of the ice cover are concerned, distorted models do not yield numerically correct values. Even before qualitative conclusions can be arrived at, it is necessary to check - with the help of diagram of Pariset and Hausser - whether the points characterizing model and prototype conditions lie in the same field, or not.

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ICE SYMPOSIUM 1970
REYKJAVIK

ICE MODELING IN HYDRAULIC ENGINEERING

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SYNOPSIS

This is a short summary of laws of similitude applied to problems of floating ice acting on structure where hydromechanical similitude has to be taken into account. We also discuss briefly the latest developments concerning the artificial material that can simulate ice on models.

RESUME

Il s'agit d'une courte revue des lois de similitude appliquées aux problèmes de l'interaction des glaces flottantes avec les ouvrages de génie où il faut aussi tenir compte de la similitude hydromécanique. Nous discutons aussi brièvement les derniers progrès concernant le développement de matériaux artificiels pour simuler la glace sur les modèles.

INTRODUCTION

More than twenty years ago the first model of floating ice[1] was built to study a problem of river ice jamming. It was conceived along standard practices in hydraulic engineering with the Froude scale and was followed by many others of the same type. The similitude was limited mainly to representing hydrodynamical conditions in order to obtain water levels.

In the past few years new problems have come up where forces caused by ice acting on structures have to be determined. Modeling techniques have evolved to cope with this problem but because of the difficulty in finding a material that simulates the breaking of ice, the laws of similitude are being questioned[2, 3].

So we think it would be in order now to state the simple laws of similitude for modeling floating ice while giving briefly the latest developments concerning the model material itself.

HYDRODYNAMICAL SIMILITUDE

Models where the conditions of internal collapse of the floating ice are not represented have been widely used. The ice is usually made of individual pieces, considered unbreakable, and only their hydrodynamic behavior is simulated.

A model is geometrically and cinematically similar if for corresponding points at corresponding times, we have:

$$\begin{aligned} \ell &= \lambda\ \ell m & t &= t_* tm \\ v &= v_* vm & \lambda &= v_* t_* \\ a &= a_* am & \lambda &= a_* t_*^2 \end{aligned} \tag{1}$$

where ℓ, t, v and a are the distances, times, velocities and accelerations on the prototype with subscripts m for model values. The corresponding scales are λ, t_*, v_* and a_* .

To obtain dynamical similitude it is required that there be a unique ratio of all forces in the model and prototype:

$$F = F_* Fm \tag{2}$$

The forces of gravity on water, ice and other structures have the form:

$$F = \rho\ g\ \ell^3 \tag{3}$$

If water is used in the model, this leads to:

$$f_* = \lambda^3 \tag{4}$$

The forces of inertia and water friction have respectively the form:

$$F = \rho \ell^3\ {}^{dv}/dt, \qquad T = C(^{\varepsilon}/\ell,\ \mathrm{Re})\ \rho\ \frac{V^2 \ell^2}{2} \tag{5}$$

For rough turbulent flows in the model as in nature, where Reynolds members Re are high enough, both these conditions lead to:

$$F_* = \lambda^2\ v_*^2 \tag{6}$$

It can readily be seen from (4) and (6) that full dynamical similitude is the Froude similitude:

$$v_* = \sqrt{\lambda} \qquad t_* = \sqrt{\lambda}$$

$$a_* = 1 \qquad F_* = \lambda^3$$

Under those conditions, the roughness parameter ε in (5) has to be reduced to the geometrical scale. In the same manner the friction factor of ice on other ice or other surfaces has to be identical in model and prototype.

Many ice models have been built with the Froude similitude, giving successful results[4, 5, 6], the ice itself being represented by paraffin, wood or polyethylene pieces. We will not elaborate on this well known technique.

COMPLETE HYDRO-MECHANICAL SIMILITUDE

If brittle failure of the floating ice sheet of thickness h is considered for the prototype and the model and if this failure can be predicted from the elastic theory, the following supplementary forces have to be taken into account:

Shear failure forces:

$$T = \tau o \ h\ell \ C_1 \left(\frac{a}{\ell}, \frac{b}{\ell}, \frac{c}{\ell}, \nu \right) \tag{8}$$

Crushing failure forces:

$$N = \sigma c \ h\ell \ C_2 \left(\frac{a}{\ell}, \frac{b}{\ell}, \frac{c}{\ell}, \nu \right) \tag{9}$$

Flexural failure forces:

$$P = \sigma o \ h^2 \ C_3 \left(\frac{a}{L}, \frac{b}{L}, \frac{c}{L}, \nu \right) \tag{10}$$

where T, N and P are the forces producing failure; τo, σc and σo the shear, compressive and flexural strength of ice; ℓ, a, b, c, representative lengths corresponding to conditions of loading C_1, C_2, C_3 arbitrary functions; ν Poisson's ratio and L a characteristic length called the radius relative stiffness given by:

$$L = \sqrt[4]{\frac{E \ h^3}{12 \ \rho g \ (1-\nu^2)}} \tag{11}$$

E: Modulus of elasticity of ice.

If we require complete similitude to be attained and particularly the correct geometrical representation of the loading forces relative to the ice, given implicitely in functions C_1, C_2 and C_3, we get from relations (4) (8) (9) (10) (11) extra conditions of similitude:

$$\left.\begin{aligned} \tau o_* &= \sigma c_* = \sigma o_* = \lambda \\ L_* &= \lambda \text{ and } E_* = \lambda \ (1-\nu^2)_* \end{aligned}\right\} \tag{12}$$

Equations (7) and (12) gives complete hydro-mechanical similitude.

A few models have been built in recent years where only some of the requirements of (12) could be obtained, usually a correct value of the σ_o, scale. One technique, developed by the Arctic and Antarctic Institute of the USSR[7] uses weakened ice produced by quick freezing of brine. The flexural strength is reduced to scale but Young's modulus, the shear and compressive strengths are not reduced to the same scale. This gives a distortion in the results which is hard to appreciate. The same can be said of many materials to simulate ice based on wax and paraffin weakened by various additives. Very recently two breakthroughs[8,9] have been made in developing model ice that would permit in most cases full hydromechanical similitude at reasonable scales.

CONCLUSIONS

The techniques of simulation of floating ice has improved very rapidly in the last few years. The theory itself is straitforward and simple but the big difficulty has been to develop a material that would simulate to a small enough scale all the mechanical properties of ice.

Such a material has been obtained recently and this will improve considerably our ability to solve the very complex problems of ice action on hydraulic and marine structures.

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DISCUSSION

Dr. J. F. Kennedy: Director, Iowa Inst. of Hydraulic Research, U.S.A.

It is indeed interesting to learn more details about the artificial ice developed by Professor Michel. The fixed strength properties reported in one of his slides, together with the strength scaling relation, given by his Equation 12, impose a more or less constant geometric scaling ratio, λ. This could be a severe limitation when modelling large prototype structures. It is to be hoped that there will soon be developed a material whose strength can be readily varied over a moderately wide range.

Dr. A. Assur: Chief Scientist, CRREL, U.S.A.

The oral presentation stressed among other things the effect of the angle of internal friction for ice accumulations. What are the best values to assume and how have these been documented?

C. R. Neill: Research Council of Alberta, Canada.

Concerning the new model ice material mentioned by the author, are the specifications for this material freely available, or is it a patented material?

DISCUSSION

P. J. Dix: British Hovercraft Corp., England.

The approach to scaling the icebreaking mechanism described by Dr. Michel is good and typifies the approach being adopted in several research establishments.

It seems unlikely, however, that a completely accurate ice model in which all properties are scaled precisely can at this stage be obtained. We are therefore left to model in the best possible way and attempt to compare the data with full scale. Despite the fact that meaningful model tests on ships in open water have been carried out since 1870 it is still not possible to obtain a complete model of the situation, so how long must we give for tests in ice?

The approach described by Edwards in his paper is the one to be adopted i.e. if it agrees with full scale then thescaling cannot be too far adrift.

It is considered important to note Dr. Michel's statement that only a brittle failure can at this time be represented. With particularly low rates of loading as may be experienced with ice drifting past structures, this is approaching the ductile regime and consequently presents different problems.

CLOSURE

B. Michel

I have not elaborate too much on the characteristics of the model ice we have developed in our laboratory as this was not the objective of this paper. But I can say a few words on it following discussion by Dr. Kennedy and Mr. Neill. First of all the values of the different strength characteristics (σ_c, σ_o, E) can be varied at will from their minimum values up to the design values that may be used in nature. They can also be raised one relative to the other in a more limited way by changing some components of the chemical mixture. But we have found that it is impossible to make model ice with σb smaller than 2 psi. that would hold itself up and be workable in a model study. So this is the value that gives the smallest possible model (σb = 2.5 psi. to represent a given type of ice) and usually the most economic one. Of course any scale that gives a higher σb model value correspond to model ice that is easier to make and manipulate. This material is not patented but is available at our laboratory.

Dr. Assur has put up one important question in ice similitude where the model technique is better than nature. We can easily measure the angle of internal friction of model ice and adjust it. Unfortunately, I know of no measurement made in nature for this important parameter. In the computation of the equilibrium and stability of ice jams in rivers I have used[10] the value of 25° and I found that this factor had little importance for these computations for values between 15° and 30°. This might not be the case for other phenomena of ice accumulation.

Mr. Dix is bringing up the whole philosophy of ice modeling. He is saying that we still cannot fully simulate simple hydraulic phenomena so it is certainly not possible to do it with ice and the only approach is the pragmatic one of modeling that agrees with full scale data.

This reasoning is certainly correct when two conditions are respected; the first one is that you know what is being measured in nature and the second one is that you are modeling the same phenomenon on the model.

The question of ice modeling makes me think of modeling in fluvial hydraulics. If you go to nature and make measurements you obtain results of sediment load and bed movements only for the very special combination of events occurring in nature. You get qualitative understanding of the phenomena and a first statistical interpretation of the data not extrapolable to any other river or any other case. If you take the problem you are studying in the lab. where you can control successively all the parameters you obtain a large understanding of the physics of the phenomenon with a quantitative formulation of the interaction of the parameters. The model is then a more useful tool than the measurements in nature. It is the same with ice. If you go to nature you get overall results where a large number of parameters (only a few ones are usually measured) combined in only one particular set of circumstances. You can define qualitatively the phenomenon and get a few measurements of their overall effects. But if you understand these phenomena and know how to model them you can study systematically the effect of each parameter and quantify it better than you will ever do in nature.

Modeling that agrees with full scale data can be done in a number of ways that do not necessarily represents the phenomenon occurring in nature. This has happened quite often in fluvial hydraulics and a few times in ice modeling. A typical theoretical case is modeling a jam and the water level rise in a river. You can do it by having an unconsolidated ice jam. You can do it by having a too strong ice sheet on the model. You can do it by having a too weak ice sheet on the model but by putting a very heavy suspended ice load in the flow. All pragmatic results in each case can be adjusted to be right but only one simulation would represent the natural phenomenon. Extrapolation with modified river conditions would lead in the other two cases to very erroneous results.

Ice modeling is one of the most powerful tool to make our field progress but under the condition that it is used properly for what it is intended to do with its laws very well understood.

10- Michel, B. (1968)- "Thrust by an unconsolidated ice cover on a boom" National Research Council, Canada. Tech. Mem. No. 92, pp. 163-170.

DISCUSSION RECEIVED AFTERWARDS

J. L. Haydock: Acres Consulting Services Ltd, Canada.

I should like to comment on the general problem of ice modelling hydro-mechanical similitude with particular reference to the physical properties of synthetic model ice.

Professor Michel opens the section of his paper on hydro-mechanical similitude with two qualifications.

1- "If brittle failure is considered"

2- "If this failure can be predicted from the elastic theory".

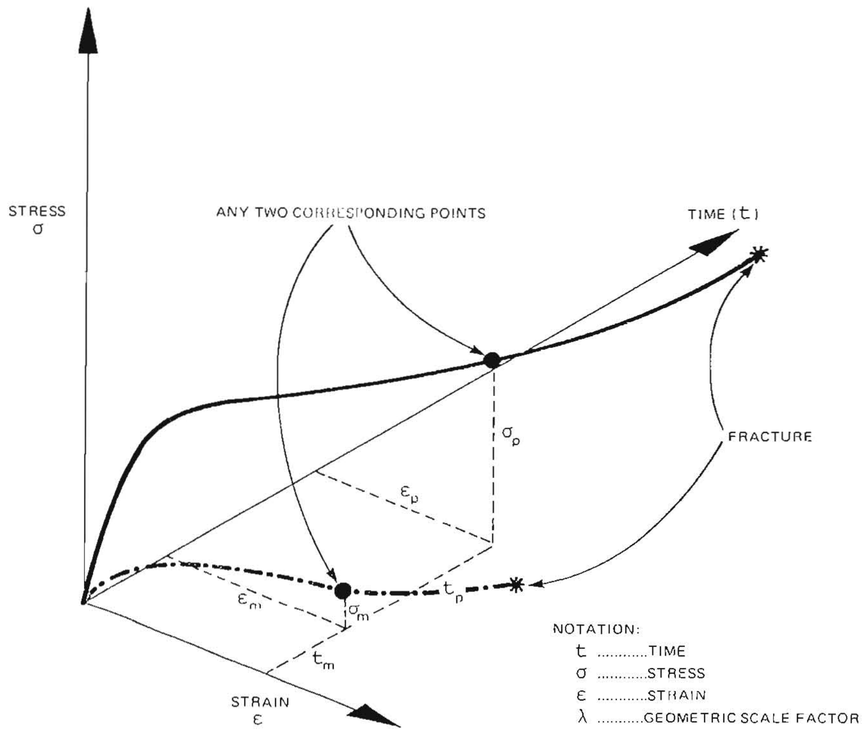
These imply linear elastic stress-strain behaviour, together with ultimate strength, and stress-deformation characteristics which are independent of rate of loading and duration of load. Though these assumptions will often be justifiable for cases of rapid loading, the question naturally arises--what would the properties of model material have to be to cope with other types of problem in which the non-linearities and time-dependent load-deformation and ultimate strength characteristics of ice and ice-snow mixtures cannot be ignored.

The complete similitude analysis is lengthy but it was carried out some 6 years ago by the Canadian consulting engineering firm of H. G. Acres & Company Limited prior to a series of hydro-mechanical ice model tests executed as a part of design studies for the crossing of Northumberland Strait between the Canadian mainland and Prince Edward Island. The results of this similitude study can, however, be summarized quite briefly.

For complete similitude of static, quasi-static and dynamic behaviour in a single model we must have, in conjunction with Froude scaling, a synthetic model ice which has the properties shown on the accompanying sketch.

The full line is a more or less arbitrarily drawn form of stress-strain-time characteristic for the prototype ice or ice-snow mixture. The stress-strain-time paths for the model material and the real ice, when tested under similar loading systems, say unconfined compression, must be related to each other by the set of linear transformations shown on the sketch for all corresponding

GENERALISED STRESS - STRAIN - TIME CHARACTERISTICS FOR MODEL ICE IN FROUDE TYPE MODEL



STRESS - STRAIN - TIME PATHS UNDER SIMILAR LOADING SYSTEMS:

———— PROTOTYPE ICE

—·—·— MODEL ICE

MODEL - PROTOTYPE TRANSFORMATION RELATIONSHIPS·

1) TIME $t_m = t_p/\sqrt{\lambda}$

2) STRESS $\sigma_m = \sigma_p/\lambda$

3) STRAIN $\varepsilon_m = \varepsilon_p$

FOR ALL CORRESPONDING POINTS ON MODEL AND PROTOTYPE STRESS - STRAIN - TIME PATHS INCLUDING FRACTURE.

points on the two paths up to and including failure. The hatched curve shows the required characteristics for the model ice material derived in this way.

When time-dependency is neglected, the paths become two-dimensional in the stress-strain plane and relation 1) on the sketch becomes irrelevant. With the additional assumption that the stress-strain behaviour is linear, relationships 2) and 3) on the sketch yield the similitude criteria indicated by Professor Michel in his paper as a special case.

The extent to which the complete requirements can be met in a single synthetic material governs the effectiveness of the simulation in the most general case. If the requirements can be satisfied entirely, reproduction of all the types of hydro-mechanical behaviour in a single model will be virtually complete. Creep phenomena, plastic failure, "brittle" failure, hysteresis, propagation of compression, shear and flexural waves, will all be satisfactorily simulated. Similarity fails only for flexural waves propagated across the ice sheet at speeds approaching that of sound in water (about 1 mile per second) but the type of shock wave problem to which this would relate is encountered only in very special cases.

The general similitude requirements for synthetic ice in the Froude type model are, of course, a great deal easier to state than to achieve, and in any practical problem many compromises must be made. It is, however, desirable to be aware of the general requirements so that a specific modelling problem can be attacked with a knowledge of the approximations it is necessary to make, and so that research work to develop better and more adaptable materials can proceed toward well defined goals.

It is interesting to note that structural engineers concerned with arch dam design and development have achieved remarkable success over the years in evolving model materials simulating, to definite scales, the non-linear and time-dependent properties of concrete and rock for use in static and dynamic model tests of arch dams. These studies include simulation of failure modes and determination of failure load factors. The success achieved in this field must, I think, be very largely attributed to a thorough understanding of the general similitude requirements for such models and intensive development work on model materials.

In spite of recent developments in hydro-mechanical modelling of ice and refinement of model ice materials there is still a great need for further progress and I trust that this discussion will help to indicate the opportunities which remain to be exploited.

CLOSURE

B. Michel

I have said in the oral presentation of this short paper, limited to four pages, that there was interest in the future of ice modeling to simulate the non-elastic behaviour of ice in the ductile range. This is certainly a challenge and the approach described by Mr. Haydock is correct. We have done extensive testing on fresh water ice in our laboratory and the arbitrary curve shown by Mr. Haydock correspond to plastic flow of ice under very small rates of deformation. A few problems only are of practical interest in this domain like the problem of thermal thrust of an expanding ice sheet and the problem of internal pressures set up by wind and water friction forces. These problems are certainly not of the types to be treated by model studies. Another area of interest would be in the range of ductile behaviour of ice. But again this is limited to rates of interaction of ice pieces with structures less than a few tenths of a foot per second. The very large domain of testing will still remain in the elastic range where we have brittle behaviour both of the ice and its modeling material.

ICE SYMPOSIUM 1970
REYKJAVIK

MODELLING THE MOTION OF SHIPS THROUGH POLAR ICE FIELDS USING UNCONSTRAINED, SELF-PROPELLED MODELS

Roderick Y. Edwards, Jr.* — ARCTEC, Inc., Washington, D.C., U.S.A.
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President

Synopsis

The desirability of performing self-propelled model tests of icebreaking ships is outlined. Scaling relationships for modelling icebreaker performance are discussed briefly. Model test techniques employed by the U.S. Coast Guard at the Naval Undersea Research and Development Center are reviewed. The results of the tests of models of the Wind class icebreaker and a new hull form are presented. Correlation between model and full scale is demonstrated by comparing the results of full scale and model trials of the Wind class icebreaker. The model test results revealed theoretically predicted differences in absolute icebreaking resistances and also showed effects of hull form variations.

All assertions or opinions contained herein are the private views of the writers and are not to be construed as official or reflecting the views of the Commandant or the United States Coast Guard at large.

*Formerly LCDR., U.S. Coast Guard

INTRODUCTION

In 1964 the United States Coast Guard began feasibility studies for the design of a polar icebreaker to replace the Wind class icebreakers. Little information on the powering requirements for such ships was available at that time. A series of model and full scale trials were therefore conducted in conjunction with analytical work to find a suitable technique for predicting the powering requirements for these ships. One technique is the use of a model sea ice, the properties of which permit the application of scaling laws suggested by Nogid [5] and Kashteljan et al. [1]. Model tests have the advantage of permitting a wide variation of the parameters of test ships without the financial risk associated with testing unusual ideas on a large scale. Usually, ship model tests are conducted by towing the ship with a carriage and measuring the towing force and speed. In towed model tests in ice the propulsive force cannot be applied to the model in a completely realistic way. Self propelled tests permit realistic behavior of the model during the icebreaking process and provide useful information on the propeller-ice interaction phenomena.

Scaling Laws

The icebreaking problem may be characterized as the interaction of a propelled body floating on a free surface with an elastic plate on a hydrostatic foundation. Several important differential equations govern the elastic, hydro-dynamic, inertial and gravitational forces which arise during ice interactions. To insure the proper relationship between the model and full scale behavior, these differential equations, in dimensionless form, must be the same for both the model and the prototype. Nogid [5] and Kashteljan, Poznyak and Ryvlin, [1] have given an excellent development of these scaling relationships. They show that the use of a material and a model ship which satisfies the constraints listed below will provide a suitable model of the dynamic interaction between a ship and a floating ice sheet

$$h_m = 1/\lambda \cdot h_p \tag{1}$$
$$\sigma_m = 1/\lambda \cdot \sigma_p \tag{2}$$
$$E_m = 1/\lambda \cdot E_p \tag{3}$$
$$\rho_{im} = \rho_{ip} \tag{4}$$
$$L_m = 1/\lambda L_p \tag{5}$$
$$M_m = 1/\lambda^3 \cdot M_p \tag{6}$$
$$I_m = 1/\lambda^5 \cdot I_p \tag{7}$$
$$V_m = 1/\lambda^{1/2} \cdot V_p \tag{8}$$

where h, σ, E and ρi are the ice thickness, ice strength, elastic modulus of the ice and mass density of the ice respectively and L, M, I, V are the linear dimension, mass, mass moment of inertia and velocity of the vessel respectively. The subscripts m and p refer to model and prototype (full scale) respectively. Lewis and Edwards [2] have arrived at the same conclusion using dimensional analysis (Pi Theorem) and have corroborated their results with full scale data, model test data in fresh water ice and model test data in sea ice. They have proposed the following continuous ice resistance relationship.

$$R/\sigma h^2 = C_0 + C_1 \rho_i g B/\sigma + C_2 \rho_i B V^2/\sigma h \qquad (9)$$

It may be seen that this equation implies the same scaling laws as equations (1) through (8).

Model Test Techniques

The important components of self propelled icebreaker model experiments are a carefully prepared model ice sheet, a well calibrated and properly balanced model and an accurate method for measuring ship speed, propeller speed, and propeller thrust.

Ice Sheet. - The ice sheets were formed on the surface of a sea water pool by lowering the air temperature in the room containing the pool. Best results were obtained using a freezing air temperature of -10°F. When the desired ice thickness was obtained the air temperature was raised to 10°F. The experiments were conducted at this "holding" temperature. Strength of the ice was obtained from in-situ cantilever beam tests. The elastic modulus was obtained by measuring sheet deflection under various loads. At the end of each continuous or ramming run, the ice thickness was measured carefully along the track of the model.

Model Ship. - An accurate scale model (1/36) of the prototype hulls was constructed. A series of towing tests were performed in open water on each model. The result of these tests was an array of available model towing force as a function of propeller and model speed. This data was subsequently programmed on a computer so force available for icebreaking could be retrieved accurately from measured values of propeller and model speed. The model was carefully ballasted and dynamically calibrated such that the radius of gyration of the model about the midship axis is $1/\lambda$ times the radius of gyration of the full scale ship.

Model Measurements. - The self propelled model requires an umbilical cord (see Fig. 1) to provide power to the model engines. Measurements of motor speed, torque and 6 components of hull acceleration were removed through this umbilical cord. Great care was taken to insure that it would not influence the behavior of the model. The velocity was measured using the arrangement shown in Figure 2. The system was carefully designed to minimize forces on the model. The outputs of all the sensors were recorded on a light pen oscillograph.

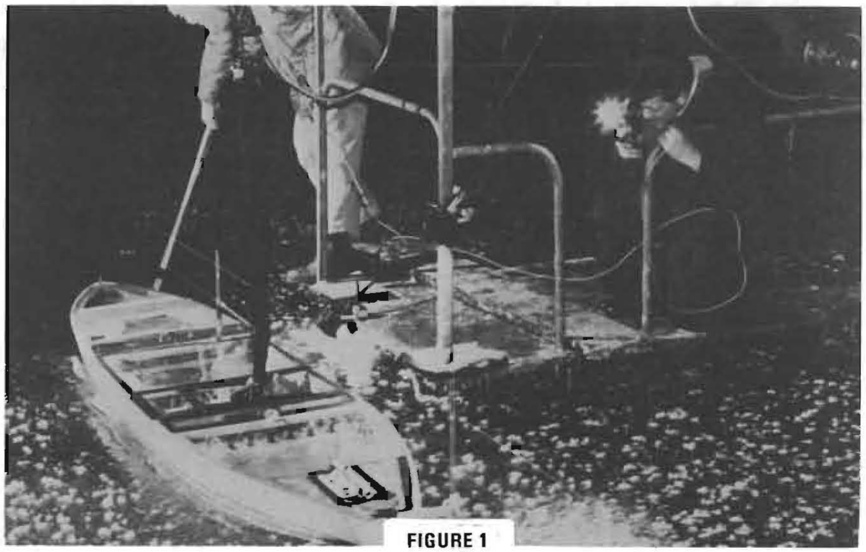

FIGURE 1

PHOTOGRAPH OF A 1/36 SCALE MODEL OF AN ICEBREAKER PROCEEDING THROUGH 3.0 CM OF SEA ICE. (MAN IN BACKGROUND IS TAKING UNDERWATER PICTURES OF THE SCREWS; MAN IN FOREGROUND IS SHOOTING SURFACE MOTION PICTURES; THIRD MAN CONTROLS THE CARRIAGE SPEED KEEPING THE UMBILICAL CORD VERTICAL OVER THE MODEL.)

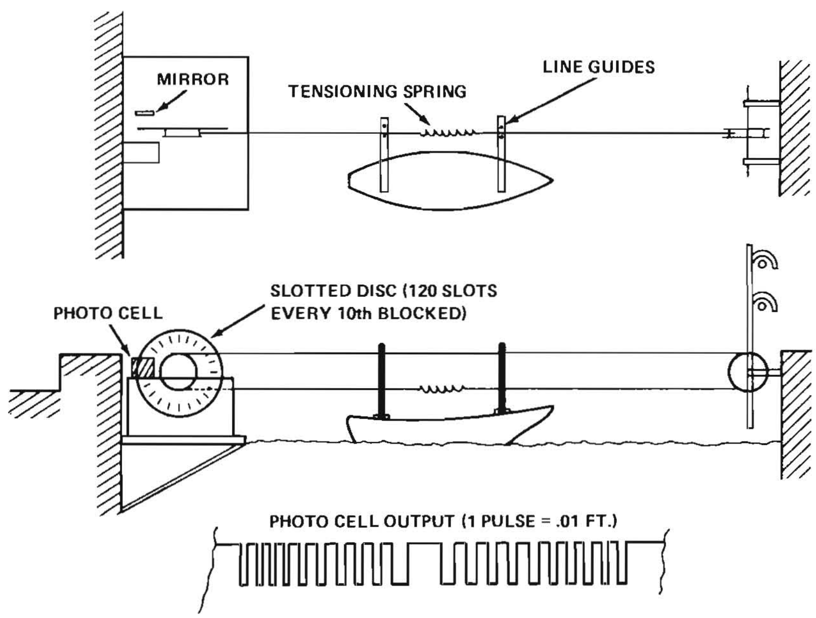

FIGURE 2

SKETCH OF MODEL VELOCITY AND DISTANCE RECORDING SYSTEM

Description Of A Continuous Mode Icebreaking Run. - Prior to each run, values were obtained for elastic modulus and strength. In-situ cantilever beam breaking tests were used to obtain the strength of the ice sheet. The forces which caused the beam to fail and the cantilever dimensions were measured. Strength was computed as follows:

$$\sigma = \frac{6P_oL}{bh^2} \tag{10}$$

where σ, P_o, L, b, and h are the failure strength of the ice, the load required to cause failure, the cantilever length, breadth and thickness respectively. Elastic modulus is obtained by determining the characteristic length of the ice sheet.

$$\ell_c = \sqrt[4]{\frac{Eh^3}{12\rho_w g(1-\nu^2)}} \tag{11}$$

where ℓ = characteristic length, ν= Poisson's ratio (approximately .33), ρ_w = the mass density of the fluid. The characteristic length may be obtained from the solution to the deflection of the infinite ice sheet prepared by Nevel [3].

$$\frac{W\rho_w gX^2}{P_o} = -\ 1/2\pi \frac{X^2}{\ell^2} \text{kei} \left(\frac{X}{\ell}\right) \tag{12}$$

where W, P_o, X, kei are deflection of the ice sheet, load, distance from load to deflection gauge and a Kelvin function tabulated by Nevel [3] respectively. Nevel [4] has tabulated the righthand side of (12) in terms of $\frac{X}{\ell_c}$. For each data point the lefthand side is a constant, hence X/ℓ_c may be obtained by a table search and interpolation. ℓ_c and subsequently E may then be computed.

While the strength measurements were being conducted, model sensors were calibrated and the model prepared for the test. After being placed in a slot in the ice sheet, power was applied to the propellers and the model was permitted to progress unconstrained, except in the athwartships direction, for five or six model lengths. Underwater motion pictures of the propeller area were taken when possible. Surface pictures were also taken to provide an opportunity to evaluate the similarity between model behavior and full scale behavior visually. At the conclusion of the run, ice thickness was recorded at one meter intervals along the track.

Data Reduction

The model velocity and shaft speed readings were obtained from the oscillograph records at one meter intervals. The readings were combined with the ice thickness data and recorded on computer cards. Each run was processed using a program which computed available thrust from model and propeller speeds. The program printed and plotted out the values of resistance (available thrust), ice thickness, shaft speed and model velocity at every meter. Average values of resistance, ice thickness, and model speed were computed over those regions for which the plots indicated steady state conditions (See Figure 3). Each average value of ice thickness, velocity and resistance, constituted a data point. Using this data, combined with ice strength, ice density (assumed constant at 1.79 $lb\text{-}sec^2/ft^4$) and model beam, the dimensionless coefficients listed below were formed.

$N_\nabla = \rho_i gh/\sigma$	= dimensionless volumetric force	(13)
$N_I = \rho_i V^2/\sigma$	= dimensionless inertial force	(14)
$R' = R/\sigma h^2$	= dimensionless resistance	(15)
$B' = B/h$	= dimensionless beam	(16)

where R, ρ_i, σ, h, B, V, g are respectively resistance, mass density of the ice, ice thickness, ship beam, ship speed and the gravitational constant (in compatable units). These dimensionless numbers, their application and development are discussed at length by Lewis and Edwards [2].

RESULTS

Tests were performed on a 1/36 scale model of the Wind class icebreaker and on a model of one of the candidate hull designs for the new U. S. Coast Guard icebreaker (referred to as the M-13). The raw data was converted to the dimensionless numbers (13),(14),(15) and (16). These "dimensionless data points" were fitted to equation (9) and the three unknown constants determined using regression analysis. The resulting dimensionless equation for the Wind class model data and the results of the regression analysis of the full scale trial data of a Wind class vessel are plotted in Figure 4. The dimensionless equations are shown below for the Wind class and M-13 model data

$$R' = 0.838 + 5.382\ B'N_\nabla + 4.211\ B'N_I \quad (17)\ \text{WIND}$$

$$R' = 0.323 + 6.606\ B'N_\nabla + 3.246\ B'N_I \quad (18)\ \text{M-13}$$

The complete results of the regression analysis associated with these equations are shown in Tables 1 and 2. Equations (17) and (18) are plotted in model units in Figure 5 along with the raw data from the two model tests. The equations are plotted for the full scale conditions in Figure 6. A comparison between the two hull forms is shown in Figure 7 where the ratio of equation (17) to equation (18) for equal values of beam is plotted. Reference to Figure 7 indicates the small relative superiority of the M-13 hull form over the Wind class hull form with regard to continuous icebreaking capability. Abbreviated forebody lines drawings for the M-13 and Wind class are superimposed on Figure 7.

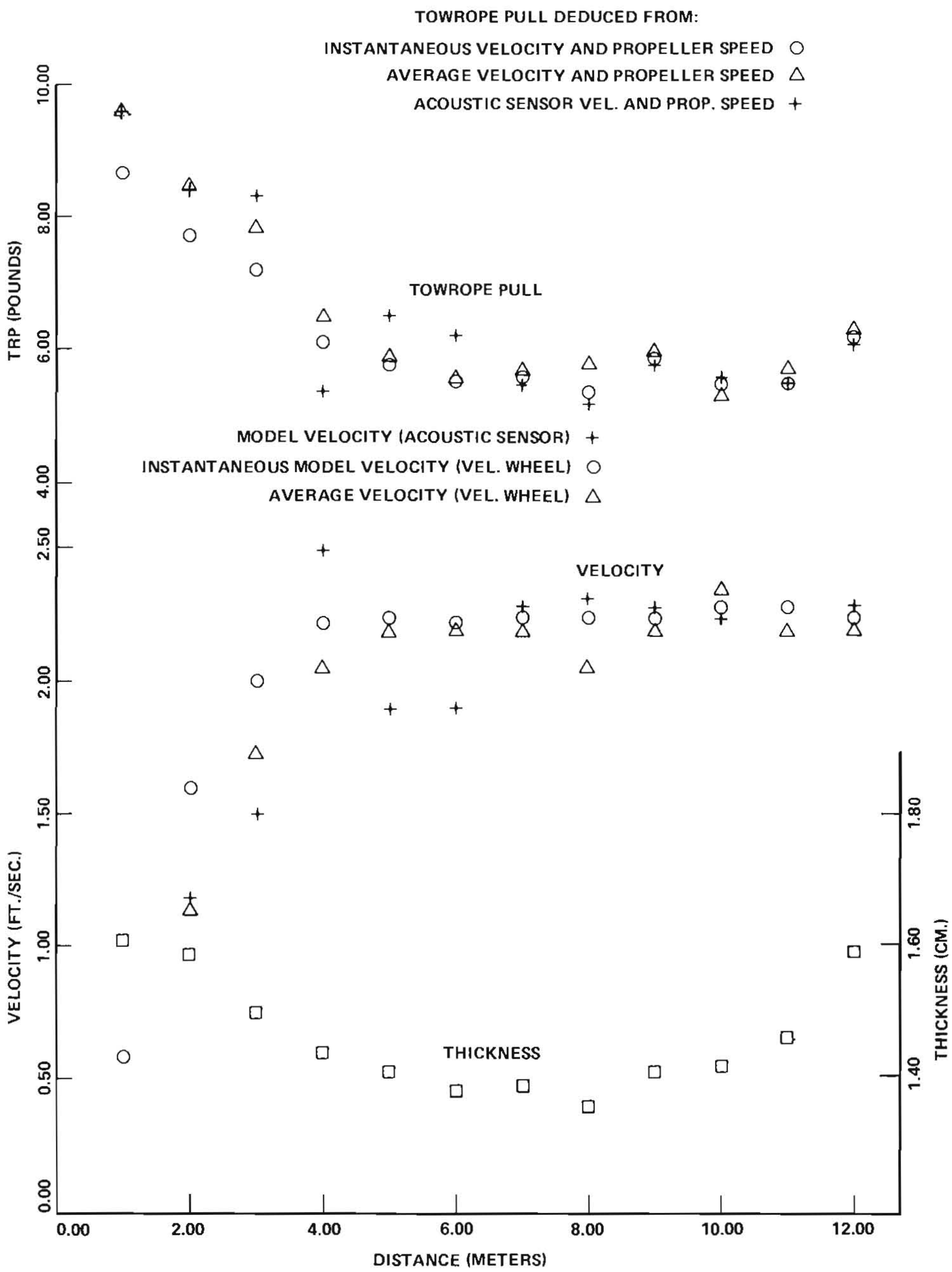

FIGURE 3

PLOT OF ICE THICKNESS, MODEL VELOCITY AND RESISTANCE (TOW ROPE PULL) vs DISTANCE (PLOTTED OUTPUT OF THE FIRST STAGE DATA REDUCTION PROGRAM)

TABLE 1. SUMMARY OF REGRESSION AND VARIANCE ANALYSIS OF WIND CLASS DIMENSIONLESS MODEL CONTINUOUS ICEBREAKING DATA

VARIABLE NAME	MEAN	STANDARD DEVIATION	CORRELATION X VS Y	REGRESSION COEFFICIENT	STD. ERROR OF REG. COEF.	COMPUTED T VALUE
$100(\frac{B}{h})\ \frac{\rho i g h}{\sigma}$	22.80616	13.84108	0.77605	0.05382	0.01086	4.95360
$100(\frac{B}{h})\ \frac{\rho i V^2}{\sigma}$	13.35272	18.66083	0.78551	0.04211	0.00806	5.22492
DEPENDENT						
1	2.62812	1.59744				

INTERCEPT 0.83815

MULTIPLE CORRELATION 0.86510

STD. ERROR OF ESTIMATE 0.81814

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES	F VALUE
ATTRIBUTABLE TO REGRESSION	2	93.57888	46.78944	69.90150
DEVIATION FROM REGRESSION	47	31.46004	0.66936	
TOTAL	49	125.03892		

TABLE 2. SUMMARY OF REGRESSION AND VARIANCE ANALYSIS OF M-13 DIMENSIONLESS MODEL CONTINUOUS ICEBREAKING DATA

VARIABLE NAME	MEAN	STANDARD DEVIATION	CORRELATION X VS Y	REGRESSION COEFFICIENT	STD. ERROR OF REG. COEF.	COMPUTED T VALUE
$\frac{B}{h}\frac{\rho igh}{\sigma}$	0.17606	0.06693	0.80245	6.60578	0.76792	8.60208
$\frac{B}{h}\frac{\rho iV^2}{\sigma}$	0.14067	0.18246	0.87446	3.24264	0.28170	11.51073
DEPENDENT						
1	1.94245	0.95068				

INTERCEPT	0.32322
MULTIPLE CORRELATION	0.95782
STD. ERROR OF ESTIMATE	0.27993

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES	F VALUE
ATRRIBUTABLE TO REGRESSION	2	34.82563	17.41281	222.22750
DEVIATION FROM REGRESSION	40	3.13423	0.07835	
TOTAL	42	37.95986		

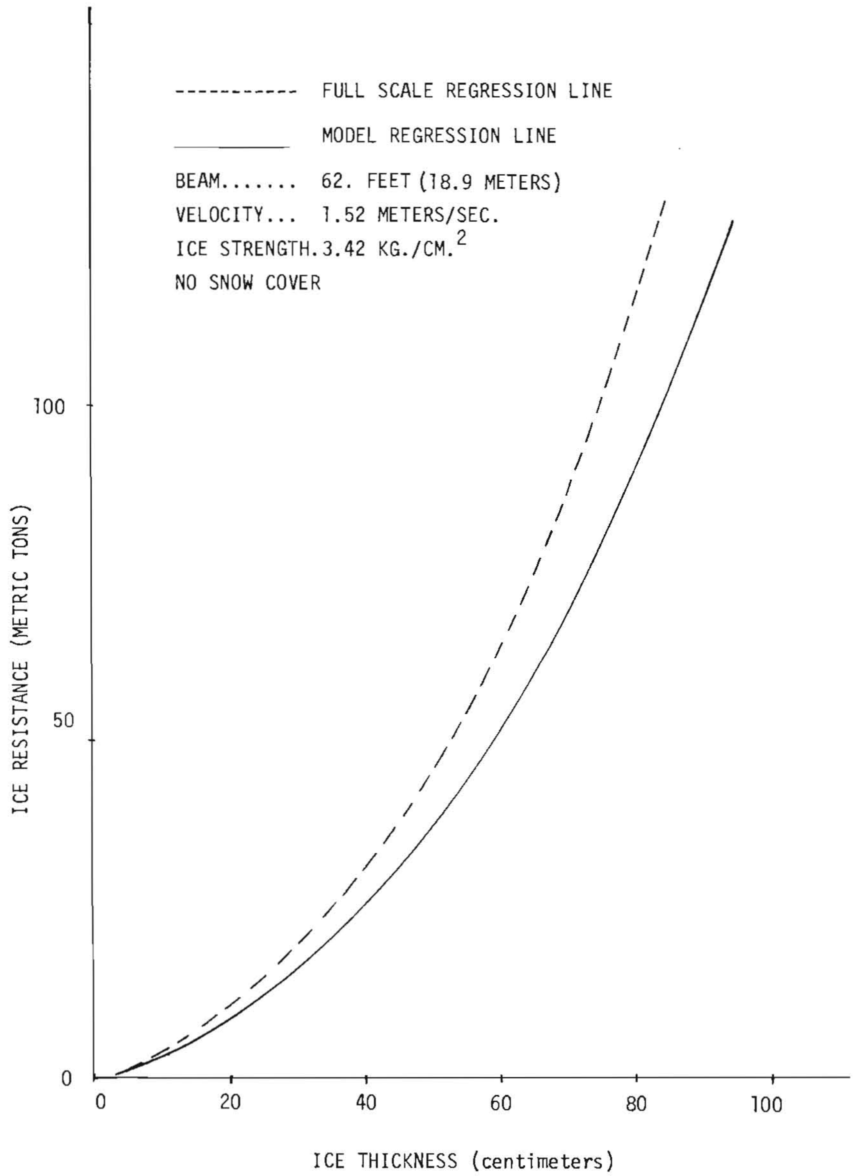

FIGURE 4.
Plot of Wind class model and full scale dimensionless regression equations in full scale units.

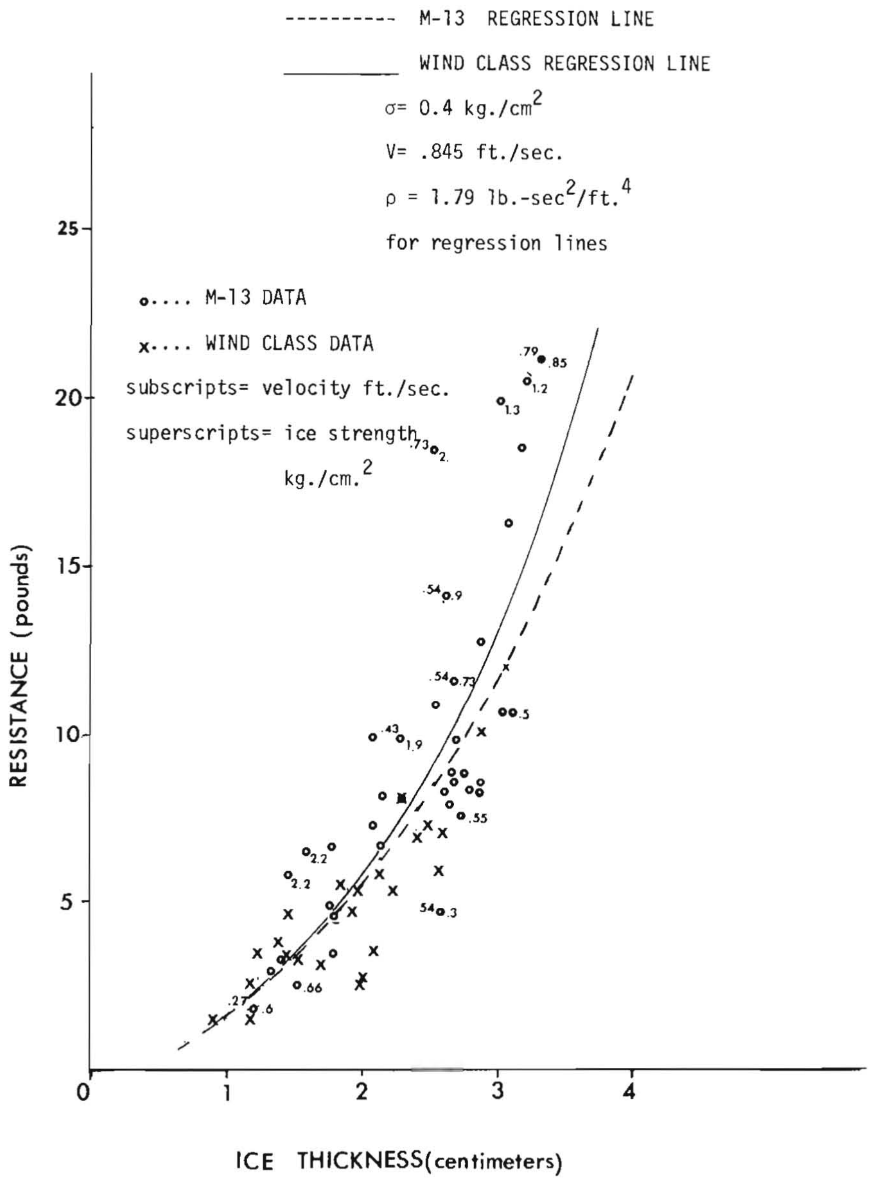

FIGURE 5.
Plot of raw data and results of regression analysis (model units) for Wind class and M-13 models.

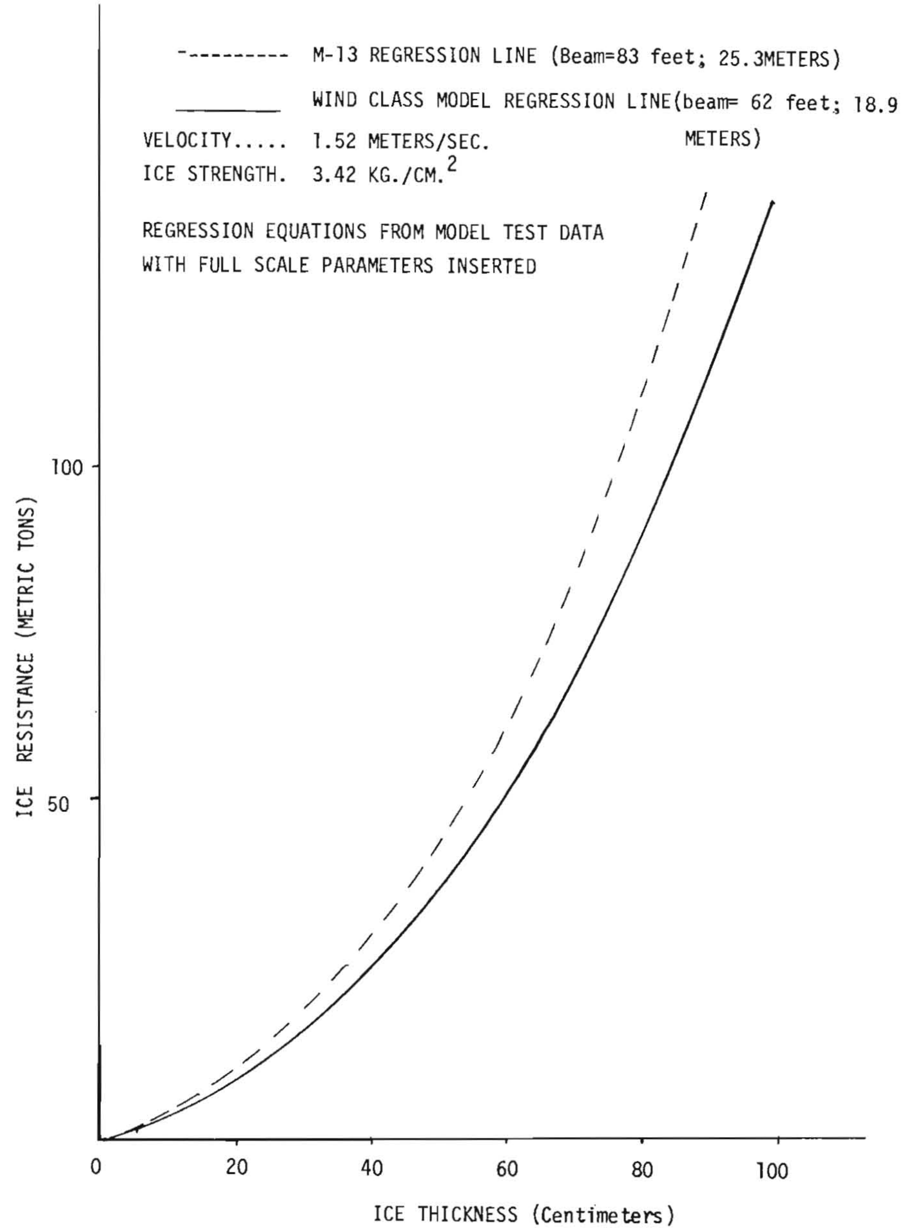

FIGURE 6.
Plot of the results of dimensionless regression analysis (full scale units) for M-13 and Wind class.

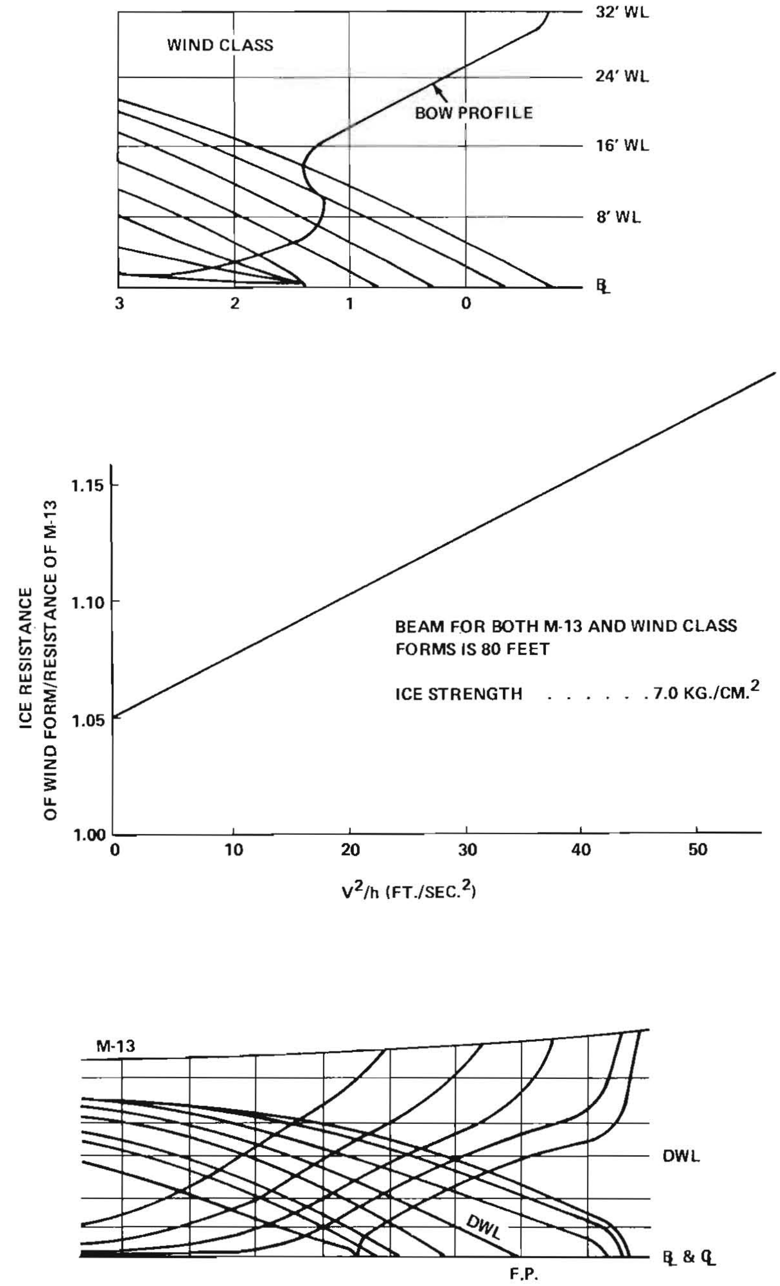

FIGURE 7

PLOT OF THE RATIO OF WIND CLASS ICE RESISTANCE TO M-13 RESISTANCE FOR A FULL SCALE BEAM OF 80 FEET vs V^2/h (ABBREVIATED FOREBODY LINES SHOWN FOR WINO CLASS AND M-13)

CONCLUSIONS

The comparison   Wind class model and full scale test results demonstrates the validity of sea ice model tests. The nondimensional approach has been used to remove the effects of different model beams. It also eliminates the problem of ice strength varying from test to test. After removal of these two effects only the effect of different hull shape and sampling and experimental errors remain to account for the difference in resultant coefficients of the dimensionless resistance equations (17) and (18). It is concluded that changes incorporated in the forebody shape of the M-13 produce a distinct improvement in continuous icebreaking performance over the Wind class.

ACKNOWLEDGEMENTS

The authors wish to recognize the members of the U. S. Coast Guard, Naval Undersea Research and Development Center and The Cold Regions Research and Engineering Laboratory whose work over the past few years in icebreaker research has contributed to the data base necessary for this paper.

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DISCUSSION by P.J. Dix (British Hovercraft Corporation)
on paper by R.Y. Edwards & J.W. Lewis (Ref. 4.14) (Modelling the Motion of Ship Through Polar Ice Fields, etc.)

The authors have neglected to mention under their section on scaling laws that a representative coefficient of friction is required between the model hull and the ice. To use the authors' subscript $\mu_m = \mu_p$.

Since the elastic modulus E is dependent on rate of application of load it is considered important that the experiments carried out to measure deflection are at a correctly scaled rate. The same may also be true for the measurements of ice strength.

The solution to the deflection of the infinite ice sheet prepared by Nevel assumes a compliant base, i.e., the water. However, in the case of the fracture of an ice sheet by an icebreaker where the breaking occurs relatively quickly it may be important to consider the virtual mass or inertia of the water.

It would be interesting to see the values of E obtained and what variation in E occurs as the strength varies. It seems unlikely that the same scale will apply to both strength and E.

The authors are to be congratulated on carrying out model tests in ice which show a good correlation with the limited full scale data which is available.

DISCUSSION by Dr. Bernard Michel (Universite Laval)
on paper by Edwards and Lewis (4-14)

The authors have been pioneers in icebreaker model tests and they were the first ones to use weakened brine ice in a towing tank on the American continent. This is certainly a worthwhile achievement in a very difficult field of ice modeling. There is little doubt that this technique is very useful to understand and quantify in the first approximation the behavior of different icebreaking bows.

The technique however fails on a number of points to be a perfect tool for research. This may be quite acceptable to answer a number of questions like the ramming one, but may be needed for more complex icebreaking operations.

One of the areas of imprecision of the technique is lack of complete similitude of all the forces and corresponding parameters that are intervening in the icebreaking process. The more obvious one is the Young's Modulus. The ice is weakened with brine and the scale is chosen in function of the flexural strength value. The Young's Modulus is then measured and assumed to be of related value in nature. There is no liberty to set it to predetermined values for various types of natural ice. Less obvious, but important processes that occur in the icebreaking operation are not scaled down. The crushing of ice near full beam of the ship, the part of crushing strength in flexural failure and the secondary shear failures of primary broken ice pieces. These phenomena may be neglected in a ramming operation but they cannot be so in continuous icebreaking when most energy is spent on the dynamic process of moving these pieces around. Another problem of similitude with real ice is the fact that at low rates of loading on a model, ice may be ductile but it will be brittle at corresponding high rates in nature.

Another weakness of this method is the low reproductibility and rather large scatter of results. This is caused by the extreme difficulty of making ice of uniform texture in a big tank and from one test to the other. The primary ice layer is very sensitive to conditions of formation including purity of water (foreign substances). Furthermore, there is a large variation of strength within the few top inches of brine ice. Ice samples with this technique may reveal large variation on mechanical properties in the same test and from one test to the other. This may explain the dispersion in the results.

As a first approximation the technique seems to give acceptable results as can be seen from figure 4 and has been used successfully by the authors.

DISCUSSION by Dr. A. Assur (Cold Regions Research and Engineering Lab) on paper by R.Y. Edwards, Jr. and J.W. Lewis

A considerable controversy exists between various schools of thought engaged in ice modeling on how to properly scale the strength properties of ice. The use of artificial materials has been suggested to achieve proper scaling of properties. In the case of icebreakers these difficulties could be alleviated if the effect of strength upon performance would be known conceptually. By adjusting for the effect of strength, the other resistance components could be modelled. According to Russian studies the effect of strength is less significant than previously assumed. Such an approach would not produce complete similitude but could be tried nontheless.

In analyzing resistance versus ice thickness it helps to eliminate the velocity effect first. Fig. 5 shows both. What is the best resistance versus velocity relationship for constant thickness?

AUTHOR'S REPLY TO DISCUSSIONS OF

PAPER 4-14 BY R.Y. EDWARDS, JR. AND J.W. LEWIS

Mr. P.J. Dix - British Hovercraft Corporation

Mr. Dix's observation concerning the coefficient of friction is correct. The equality of the friction coefficient is a necessary condition for proper modelling. The elastic modulus of ice is dependent upon the rate of load as is the ice strength. In our experiments we loaded the ice sheet with handheld force gauges and attempted to do so at a rate which we felt was typical of the model ship's rate of loading the ice. We agree that much better control of these measurements should be exercised in the future. Our observations indicate that σ and E are related linearly. It has been difficult to achieve precise scaling of both σ and E to date. We are continuing to investigate ice formation techniques to bring the two variables into a relationship which will permit the simultaneous scaling of both.

Dr. Bernard Michel - Universite Laval

We thank Dr. Michel for his comments. We agree that the modelling procedures we have developed and used are not perfect tools for research at this time. We do feel, however, that the technique provides solutions to engineering problems. We have observed that correlation between model test and full scale data is good in the continuous icebreaking mode.

With regard to load rate, the rate at which the model loads the ice sheet is proportional to the model speed which in a Froude scaled system is $1/\lambda^{1/2}$ times the full scale speed, however it appears that even at model speeds of 0.5 ft/sec, the model sea ice fails in a brittle mode. On one occasion when the model ran at approximately 0.1 ft/sec, extremely plastic ice behavior was observed. Nonetheless, the ice resistance data obtained from that run did not exhibit any noticeable variance from data obtained in the higher speed runs when the ice failed in the elastic mode.

Finally, with regard to the "scatter" of our data, may I suggest that the statistical parameters listed in Tables 1 and 2 indicate rather good correlation between the curves and the data. Later dimensionless analysis of this data, Ref. [2] indicates that it exhibits little scatter. Perhaps the fact that the raw data shown in Fig. 5 is plotted in one dimension but is a function of four variables gives the impression of large "scatter."

Dr. Andrew Assur - Cold Regions Research and Engineering Laboratory

The authors agree with Dr. Assur's practical approach to modelling ship behavior in ice. Our test experience with the U.S. Coast Guard and our current laboratory work point to the relatively small part strength plays in the behavior of a ship in ice. It would seem that one could accept a small error in ice strength, scale elastic modulus correctly and make small corrections to the test results.

To answer Dr. Assur's question about velocity effect, we suggest that the resistance appears to vary as the square of the velocity.

ICE SYMPOSIUM 1970
REYKJAVIK

MODELLING OF ICE TRANSPORT

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Trondheim, Norway

The physics of ice/water flows, in particular the set of forces at work in such a mixture, is briefly outlined.

Similitude requirements are derived, and model laws discussed. Finally, some problems of practical model testing are mentioned.

THE PHYSICAL BACKGROUND

Two Phases - The transport of ice by flowing water is a two-phase flow, and it is helpful to have this in mind and stick to a conceptual model in which the ice represents the particulate phase, no matter how wide the range of particle size. The forces at work are

a) fluid to particle forces
b) fluid to boundary "
c) particle to particle "
d) particle to boundary "

For low concentration of solids only forces of class a and b above are interesting. As the concentration of solids increases, the class c and d forces become increasingly important.

Two States - Heat exchange gives rise to changes of state. In the simplest case such changes may be represented by sources and sinks to provide the correct mass flux. However, in many cases a much more important effect of changes of state is the increase during freezing and decrease during thawing of forces of class c and d.

LAWS OF SIMILITUDE

Fluid to Particle Forces - A set of 7 forces are recognized in newtonian mechanics. In steady flow the inertia forces is balanced by the four forces of gravity, drag, lift and pressure gradient. In unsteady flow two more forces appear, one due to the added mass of the particle, and the other due to its previous history (Basset force).

A Froude model, which is based on similitude for the two first of these forces, inertia and gravity, also happens to reproduce correctly drag and pressure gradient forces, except for viscosity-dominated phenomena with certain round shapes.

The remaining steady flow force is the lift force, or more precisely, the lift due to turbulence (the lift due to the mean flow is subject only to the restriction mentioned above for the drag). A correct simulation of lift forces is necessary to secure a correct concentration distribution of ice particles suspended below the free surface. The governing parameter, sometimes referred to as the Rouse number, is

$$z = \frac{w}{\kappa u_*} \qquad (1)$$

where w is the fall velocity of the particle (negative for ice) and κu_* is a rough measure of the turbulence velocity (κ is the von Karman constant, u_* is the friction velocity).

Therefore, all of the fluid to particle forces are reproduced with good accuracy in a Froude model when 1) the particles are sharp-edged, as are indeed most ice forms, and 2) the Reynolds number is sufficiently high to produce Rouse numbers of the right order of magnitude.

Fluid to Boundary Forces - The same restrictions that apply to modelling of single phase open channel flows, apply equally for ice/water flows. A sufficiently high Reynolds number must be secured to reproduce observed or anticipated energy grade lines. At the same time the correct mode of flow (subcritical or supercritical) must be obtained, and, finally, the velocity distribution should not be significantly distorted.

In open channel models these requirements are very often met by distorting the geometry and exaggerating the vertical length scale compared with the horizontal length scale. This practice, while always introducing scale effects, nevertheless has proved extremely useful because it gives sufficiently accurate results at a low cost.

Particle to Particle Forces - For an ice bridge to form of particles floating on the surface, shear forces must be transmitted from the individual particles to its neighbours, and eventually to the solid boundary.

The theories of soil mechanics have been used to estimate the strength properties of an ice bridge. (DEVIK, O. and E. KANAVIN, 1965, PARISET et al 1966).

The shear stress τ is assumed to follow Coulomb's law

$$\tau = c + \sigma \tan \phi \qquad (2)$$

where c is the cohesion, σ the normal stress and tgϕ the friction angle of the granular material.

While the empirical equation (2) has been turned into a very useful analytical tool for soils due to the huge amounts of data available, we are only in the beginning as far as ice is concerned. Accepting (2) as a working hypothesis, similitude requires that the ratio $\frac{\tau}{\rho V^2}$, where V is a characteristic velocity, remains invariant. For a Froude model this means that the scale ratio of shear strength τ_r equals the length scale ratio l_r, or

$$c_r = (\sigma \tan \phi)_r = l_r \qquad (3)$$

If the normal stress σ is correctly reproduced, i.e. $\sigma_r = l_r$, (3) gives

$$(\tan \phi)_r = 1 \qquad (4)$$

Particle to Boundary Forces - If the boundary is simply a failure plane in an ice jam, the strength properties may be considered the same for forces of class c and class d.

If the boundary is of a different material, for instance a concrete wall or a river bed, (2) may still give a useful description of the shear with a proper choice of coefficients.

MODEL STUDIES

Reduced scale model studies have been reported for various ice/water flows, for instance

1. Breakup of ice jams (MICHEL 1965)
2. Formation of ice jams (PARISET et al 1966)
3. Discharge capacity of free surface flows (CARSTENS 1968)

Most of these studies are based on the practical approach of the professional model tester. His method is first to build and operate a physical model so as to reproduce certain observed phenomena according to a model law of his choice. Then he proceeds by extrapolating on the model beyond the range of verified performance. This in turn widens his experience when the modelled structure has been completed and the laboratory extrapolation can be checked.

While the accuracy of the model can be estimated within its verified range, the uncertainty introduced by extrapolations is not known, and usually the results are reported with some qualifying reservations.

Fig. 1 shows the model at the Technical University of Norway of the Burfell diversion, Thjorsá, Iceland, during a test with polyethylene shavings to simulate frazil slush.

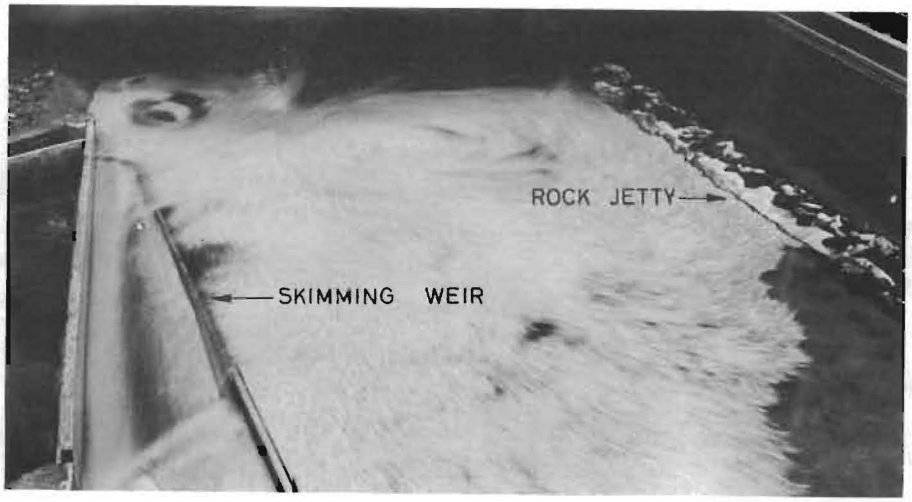

Fig. 1 1:40 model of the Burfell diversion

Fig. 2 shows the prototype in operation with actual slush in the river.

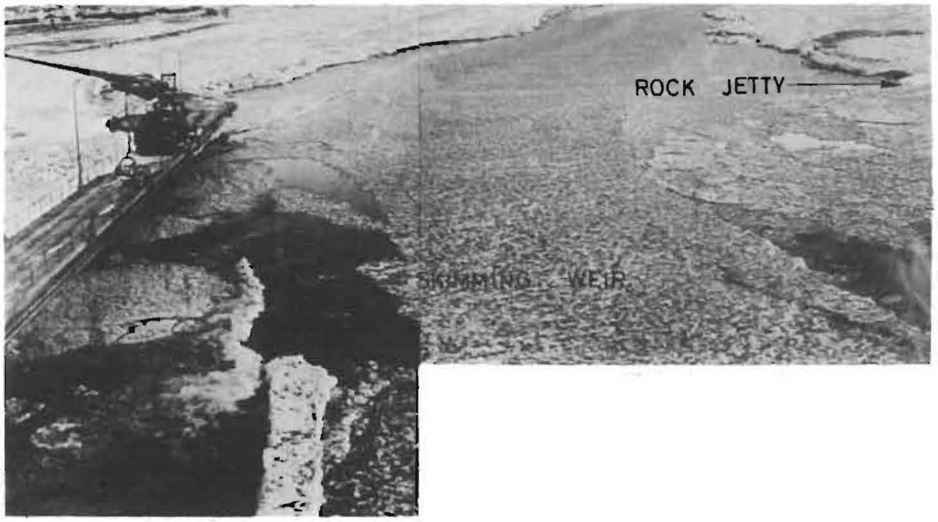

Fig. 2 The Burfell diversion, Thjorsá

Comparing the two photos, one can conclude that quite a few details were predicted on the model, which had a length scale 1:40.

Fig. 3 Shear zones in prototype frazil slush, Thjorsá at Burfell

On the other hand, the shear zones that developed in the slush carpet upstream of the diversion as seen in Fig. 3, were not predicted by the model. Shear failures in the model slush were only observed along separation lines, where strong velocity gradients existed. Whether this lack of similitude was caused by too high internal shear strength (class c forces) or too low boundary shear strength (class d forces) in the model, is not known at present.

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DISCUSSION

AUTHOR'S COMMENT: In presenting the paper I showed some slides from the model study of the Burfell diversion and made some comments on these tests and the prototype experience of last winter. The questions are concerned with this example.

G. SIGURDSSON: CARSTENS mentioned that low water levels at the inlet at Burfell would result in more frequent forming of ice bridges than high water levels. This is not correct. Of the 12 or 14 ice bridges that were formed at the inlet last winter one was caused by too low water levels or more correctly too little flushing water, two were caused by a step-burst carrying very large pieces of broken up sheet ice that stopped at the inlet in spite of high water levels. All the other bridges were formed because of too high water levels. Low water levels reduce greatly the likelyhood of ice bridge formation as well as making it easier to get rid of bridge should it form.

Answer: What I had in mind when I said that lower water levels would cause more frequent bridging, was the risk of occasional ice floes with a draft exceeding the water depth over the skimming weir. Such floes would stop against the weir, act as bridge piers and facilitate bridging.

On the other hand, the thickening of the ice carpet which results from too high water levels will also cause ice bridges, as we have just seen on your slides.

The question of the optimum water level is in my opinion a statistical one that can only be answered when experience has given the frequencies with which different ice forms occur. The record of last winter is biased, because you were operating most of the time at high water levels.

S. RIST: My questions are concerning the intake at Burfell. As SIGURDSSON mentioned in his lecture they had to cut a gap in the rockfill jetty in November last year (1969). The first question is: Did anything unexpected happen? Did the river choke at the upper end of the rockfill jetty when the ice was growing active, so it did not occur as single particles? Or is it more favourable hydraulic conditions to let the water flow direct towards the ice sluice? The third, would you advice to fill the gap again or open it more to devoid turbulence?

Answer: Step bursts on the Thjorsá are of course not unexpected, but they were not included in the model tests.

The model worked better without the jetty, however the model ice was not sheared the way the river ice is, and did not form relatively narrow approach channels through stagnant ice fields as the river does.

I would not fill the gap in the jetty. What I would like to do, is to extend the jetty to the left bank.

S. RIST: When the water is entering from the wide river bed into the narrowness by the rockfill jetty the concentration of the slush on the surface increases and the icebridges will form more easily. As you mentioned to extend the rockfill jetty to the east bank, then we are running against another problem it will not be easy in floods to use gateless spillway for excess water and ice floes.

Answer: I agree that by extending the jetty we face a risk of raising flood levels. Therefore I would build the jetty of an easily erodible material so that a raising flood would wash it out before peaking.

ICE SYMPOSIUM 1970
REYKJAVIK

THE FAILURE OF ICE

LA RUPTURE DE LA GLACE

Dr. L. W. Gold, Research Officer, Division of Building Research,
National Research Council of Canada, Ottawa, Canada.

Information is presented on the cracking activity that occurs in columnar-grained ice during creep and constant rate of strain tests, when the load is applied perpendicular to the long direction of the grains. This cracking activity is responsible for the occurrence of tertiary creep or failure in uniaxial compression tests when the stress exceeds about 12 bars. The behaviour of ice around structures is discussed with reference to the results of the laboratory studies.

On présente des données sur la fissuration de la glace à structure colonnaire au cours d'essais de fluage et de déformations à taux constant lorsque la charge est appliquée perpendiculairement à la longueur des grains. La fissuration est responsable du fluage tertiaire ou de la rupture au cours des essais sous compression uniaxiale lorsque les contraintes excèdent 12 bars. On discute du comportement de la glace autour des structures en rapport avec les résultats obtenus en laboratoire.

The determination of the maximum force that ice can exert against a structure requires knowledge of the strength of ice under various conditions of loading. Failure of ice, under most design loads of interest to engineers, is due to a breakdown of its structure by the formation of cracks. A series of observations have been carried out on the factors controlling crack formation in ice, and the role that it plays in deformation and failure (1, 2, 3). These observations were made on columnar-grained ice, subjected to simple compression in the direction perpendicular to the long direction of the grains. There was a marked tendency for the crystallographic basal planes to be parallel to the long direction of each grain. This note presents some of the results of this study that are of significance for both laboratory and field investigations of ice pressures.

CRACK FORMATION DURING CREEP

Cracks were observed to form in ice when the constant compressive stress was greater than about 6 bars. The cracks were long and narrow, and usually associated with only one or two grains. The plane of the cracks tended to be parallel to the applied stress, with the long direction in the long direction of the grains. When the stress was less than about 10 bars, the cracking activity was confined mainly to the primary stage of creep.

A marked change in creep behaviour was observed over the stress range 10 to 12 bars. The secondary creep stage developed if the stress was less than 10 bars. If the stress was greater than about 12 bars, the tertiary or accelerating creep-rate stage developed directly from the primary stage within a creep strain of 0.25 per cent. Cracking activity for these loads was continuous in the tertiary stage, and many specimens developed fault zones approximately parallel to the planes of maximum shear. The cracking activity appeared to concentrate within the fault zones, as shown in figure 1.

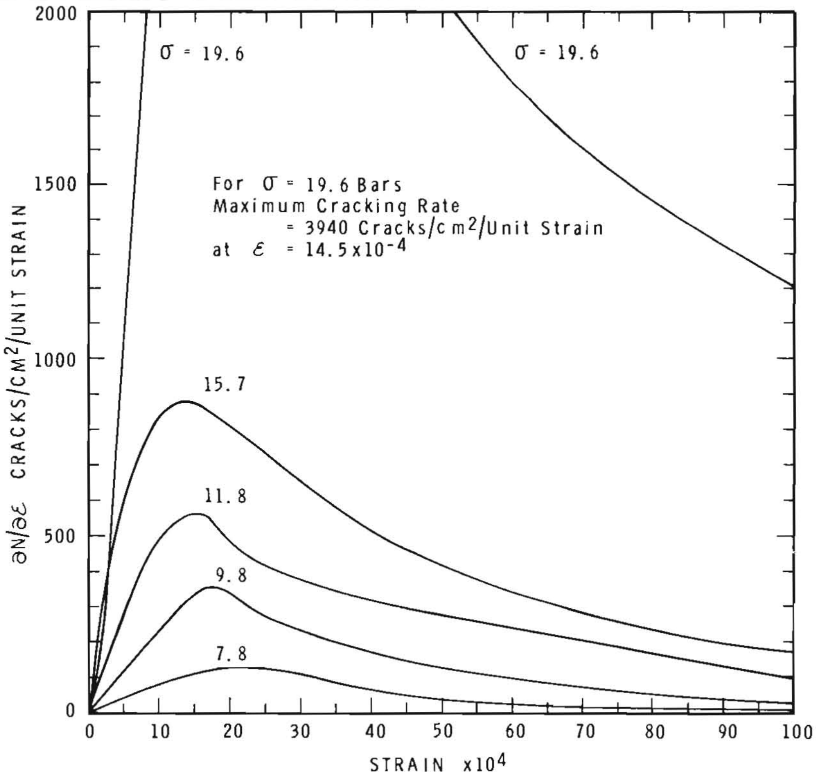
The strain dependence of the average rate of cracking is shown in figure 2 for tests carried out at various stresses at a temperature of -9.5 ± 0.5°C. The corresponding average creep behaviour is shown in figure 3. Similar cracking activity and creep behaviour was observed at temperatures of -4.8, .-14.8 and -31.0°C.

The observations indicated that the onset of the tertiary stage of creep for stress greater than 12 bars, was due to the breakdown of the structure by cracking activity. The crack density observed at the beginning of the secondary creep stage for stress of 10 bars, was about 1.4 per cm^2. It was about 1.6

1

Fig. 1. Crack formation in columnar-grained ice deformed 1 per cent; stress = 19.6 bars. Temperature = -9.5°C.

Fig. 2. Strain dependence of cracking rate, $\frac{\partial N}{\partial \varepsilon}$, for given stress σ, bars. Temperature = -9.5°C.



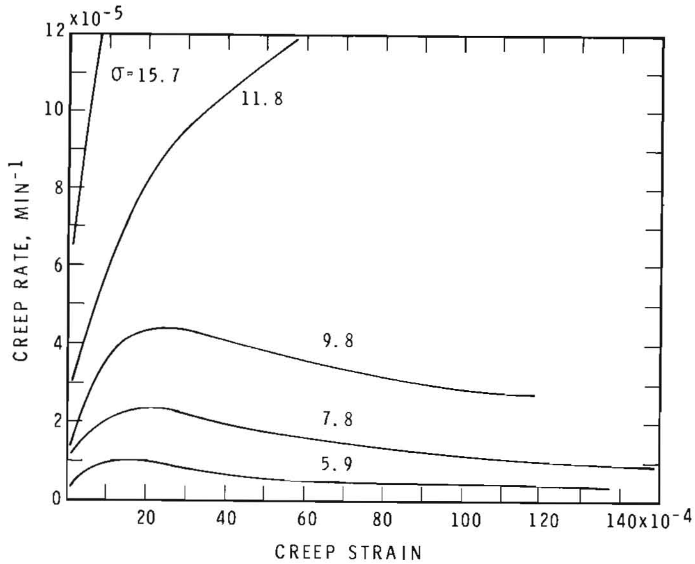


Fig. 3. Strain dependence of average creep rate for given stress. Temperature = -9.5°C. (Note the transition in behaviour in the stress range of 10 to 12 bars.

per cm^2 at the onset of the tertiary stage for stress of 11.8 bars , and about 0.5 per cm^2 for the same condition when the stress was 15.7 bars.

CRACK FORMATION DURING CONSTANT RATE OF STRAIN

Observations have also been made on the behaviour of columnar-grained ice under conditions of constant rate of cross-head movement of the testing machine. As in the constant-load tests, loads were applied perpendicular to the long direction of the grains. The stress developed had a maximum in the range of strain of 5×10^{-4} to 15×10^{-4} (i.e. an upper yield stress). A second upper yield stress was observed for strain rates in the same range as those associated with constant compressive loads of less than 10 bars The second maximum in the stress occurred in the strain range of 1 to 2 per cent; this range is about the same as that associated with the transition of the secondary creep stage to the tertiary stage for constant compressive loads.

The strain rates that were applied covered the range of 1.2×10^{-7} sec^{-1} to 1.7×10^{-4} sec^{-1}. A ductile-to-brittle transition was observed for strain rates of about 8.3×10^{-5} sec^{-1}. The yield stress in this range of strain rate was 70 to 100 bars.

Crack formation occurred when the strain rate exceeded about 10^{-7} sec^{-1}, a rate that is about equal to that associated with a constant compressive stress of 6 bars. There was very considerable cracking activity prior to and during yield for strain rates giving an upper yield stress greater than 12 bars.

Cracking activity prior to yield in the constant strain rate tests, and prior to the tertiary stage in the creep tests, appeared to be distributed relatively uniformly throughout the specimens. Yield in the constant-strain rate tests appeared to be associated with the development of fault zones in the same way as for the tertiary stage during creep (see figure 1). These observations demonstrated that the structural instability associated with failure was induced by the breakdown of the structure by cracking activity. This fact has implications for the failure behaviour of ice around structures, and the use of laboratory results on the strength of ice for predicting the maximum forces that can occur.

IMPLICATIONS FOR LABORATORY AND FIELD TESTS.

Almost all information concerning the strength of ice has been obtained from unconfined compression tests. The behaviour of columnar-grained ice for this condition of loading is such as to cause the strain during deformation and failure to be primarily perpendicular to the long direction of the grains. This strain behaviour would be inhibited under most field conditions because of the biaxial stress state that would be induced. Such a biaxial stress would also inhibit the type of cracking activity causing failure in the case of unconfined compression. For the case of complete lateral constraint, for example, crushing failure would require crack formation perpendicular as well as parallel to the long axis of the columnar-grains if the failed ice is to be displaced to the upper and bottom surfaces of the ice cover. Crushing strengths for complete lateral constraint would be greater than those observed in unconfined compression.

There appears to be little evidence that the relatively large stresses required to fail ice in the laboratory are induced near structures. From a design point of view, however, the possibility of such large stresses under some field conditions cannot be ruled out at this time. Ice covers would probably develop their maximum load on a structure when moving relative to it at a speed causing a rate of strain in the ductile-to-brittle transition range. A serious load condition might develop, for example, at an isolated structure frozen into the ice cover, if the cover is suddenly subjected to a significant lateral displacement.

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Discussion by B. Ross on paper by L.W. Gold.

Are the cracks observed in the tests noted to occur in the central regions of the specimens or are they initiated at the loading heads of the test machine?

Reply to question by B. Ross.

The cracks that are formed prior to the onset of the failure condition are uniformly distributed throughout the central region of the specimen. It was clear that the constraints imposed by the loading platens affected the distribution of the cracks near the ends of the specimens. Some evidence was also obtained of an edge effect (i.e. the crack density was smaller near the edge than in the central region). The observations on the carcking activity were made over the central portion of each specimen where the cracking activity appeared to be uniform.

Discussion by G. Frankenstein on paper by L.W. Gold.

Did the cracks occur along the Grain Boundries?

Reply to question by G. Frankenstein.

We observed both transcrystalline and grain boundary cracks. The relative proportion of these two types of cracks appears to depend upon the strain rate and associated stress.

Discussion by S. Hanagud on paper by L.W. Gold.

In your paper you mentioned that cracks are initiated under tensile loading at a stress below the level at which complete failure would take place. I would like to know the stress at which these cracks are initiated are their size. Is there any effect of these cracks?

Also I shall appreciate if you can elaborate on the failure stress under stream rate.

Reply to question by S. Hanagud.

In tensile tests, cracks were observed to form for stress equal to 4 kg/cm^2. These cracks were small, usually less than 1 mm by 1 cm in area. Some cracks were observed to degenerate into a plane of small cavities with continued deformation. The cracks did not appear to participate in the final failure event, although this occurred so rapidly that it was not possible to state positively that failure was not due to the propagation of a pre-existing crack.

All of our tests at constant rate of strain have been carried out in compression. In the ductile range of behaviour, the maximum yield stress increased with rate of strain according to a power law relationship. This dependence was found to be the following:

$$\sigma_{uy} = 203\ \dot{\varepsilon}\ 0.25\ kg/cm^2$$

where $\dot{\varepsilon}$ is in $min.^{-1}$. We have not extended our observations far enough in the brittle range of behaviour to determine the strain rate dependence of the brittle strength.

Discussion by C. Cochard on paper by L.W. Gold.

Can you tell me if the cross-sectional dimension of the crystals has an influence on the number and the repartition of the cracks?

Reply to question by C. Cochard.

Most of our studies have been carried out on ice with grain size of 1 to 3 mm. As the cracks are associated with either one or two grains, it would be expected that as the grain size increased, the number of cracks per unit area would decrease and their size increase. This appeared to be the case for specimens that we prepared from natural columnar-grained ice of larger grain size.

ICE SYMPOSIUM 1970
REYKJAVIK

BRITTLE FRACTURE OF SNOW ICE

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SYNOPSIS

The dislocation pile-up model of Stroh has been used to study crack nucleation. By energy balance considerations, the author has derived the brittle fracture criteria for both tensile and compressive tests. For an uniaxial tensile load, the stress required for crack propagation is smaller than the nucleation stress; once nucleated the crack will be self-propagating with respect to the applied stress. For an uniaxial compressive load, once nucleated the crack will come to rest at a finite length. By considering the energy dissipated in the plastic region at the head of this stable crack,it has been shown that when unstable quasi-brittle fracture occurs under compressive stress the ratio of brittle fracture stress to maximum ductile stress is 0.6. This analysis both predicts and provides a physical interpretation of the observed features of snow ice brittle behavior.

RESUME

La formation de fissures dans la glace de neige est étudiée à la lumière d'un modèle d'empilement de dislocations proposé par Stroh. Par un bilan énergétique, l'auteur dérive les critères de rupture fragile pour des essais en tension et en compression. Pour une tension uniaxiale, la contrainte nécessaire pour propager une fissure est inférieure à la contrainte de nucléation; aussitôt nucléée la fissure se propagera indéfiniment. Pour une compression uniaxiale, la fissure après avoir été nucléée se propagera seulement sur une certaine distance et deviendra alors stable. En considérant l'énergie dissipée dans la zone plastique à l'extrémité de cette fissure, l'auteur montre que, lorsqu'une fracture quasi-fragile se produit en compression uniaxiale, la contrainte de rupture fragile est égale à 0.6 de la contrainte maximum ductile. Cette analyse prévoit et explique physiquement les observations expérimentales dans le comportement fragile de la glace de neige.

Many workers have observed that the ice behavior changes from ductile to brittle above a critical strain rate.[1,2,3,4] Up to now, there have been very few attempts to study this phenomenon although most engineering problems deal with brittle fracture of ice. The following paper is a theoretical treatment of the two important stages of crack growth, initiation and propagation. The brittle fracture criteria predicted for both tensile and compressive tests are also compared with experimental data for snow ice.

CRACK NUCLEATION

The idea of cleavage initiation by a dislocation mechanism was first introduced by Zener[5]. Dislocation theory, as well as continuum mechanics, tells us that when slip occurs across a grain, but does not extend to the adjacent grains, a concentrated tensile stress is produced at the end of the slip band. This concentrated stress may become sufficiently high to initiate a crack. Full fracture of the specimen does not, of necessity, immediately follow initiation of the crack, since the latter will relax the concentrated stress and the crack will stop unless it becomes self-propagating with respect to the unconcentrated applied stress.

Schematic representation of the dislocation model used for crack initiation is shown in Figures 1 and 2. In this model, first derived by Stroh[6] and taken a step further by Smith and Barnby[7], the crack is thought to be nucleated by the climb of dislocations which are piled up against a grain boundary in an adjacent grain. The nucleation of the crack will occur as soon as $(\tau_\alpha - \tau_o)$, the effective shear stress on the dislocations, reaches a critical value:

$$(\tau_\alpha - \tau_o) = \left|\frac{3\pi\gamma\, G}{8(1-\nu)L}\right|^{\frac{1}{2}} \qquad \text{Eq. 1}$$

or if n is the number of dislocations in the pile-up, when

$$n = \frac{3\pi^2\gamma}{8(\tau_\alpha - \tau_o)b} \qquad \text{Eq. 2}$$

Here

- τ_α: $\sigma \cos\alpha \sin\alpha$: shear stress on the glide plane due to the applied stress
- τ_o: friction stress opposing the movement of the dislocations
- γ : surface energy
- G : shear modulus
- ν : Poisson's ratio
- L : length of the dislocation pile-up
- b : Burgers vector

Since crack nucleation depends only on shear stress, it will occur at the same stress level for both tensile and compressive tests.

CRACK PROPAGATION

Microcracks once nucleated, may undergo limited growth; for example, across a single grain in a polycrystalline material[8], or they may grow catastrophically and cause macroscopic fracture. The growth behavior of a crack is influenced by local stress, the stress due to the externally applied load, and by the microstructure. An important feature of macroscopic crack growth is that the energy required for this process must be supplied by the external loading system. Therefore the condition for the crack to propagate must be derived by an energy balance in a manner similar to that derived by Stroh[6] and modified by Bilby and Hewitt[9] who showed that the energy of the microcrack is given by:

$$W = \frac{n'^2 b^2 G}{4\pi(1-\nu)} \log \frac{4R}{c} + 2\gamma c - \frac{\pi(1-\nu)}{8G} (\sigma^2_{\phi} + \tau^2_{\phi})\, c^2 - \tfrac{1}{2} n'b\, (\sigma_{\phi} \sin\theta - \tau_{\phi} \cos\theta)\, c \qquad \text{Eq. 3}$$

where n' is the number of dislocations which have climbed into the crack and R is the radius of the elastic field due to the crack, other symbols are defined in Figures 1 and 2.

The stationary values of W occurs when

$$\frac{\partial W}{\partial c} = 0 \qquad \text{Eq. 4}$$

Equation 4 has either two positive real roots, or no real roots. If there are two real roots, the smaller gives a minimum value of the energy W, and so a crack of this length will remain in stable equilibrium in the material. If there are no real roots no position of equilibrium of the crack is possible, and so it will grow indefinitely. The critical case occurs when the roots of Equation 4 are equal; this will happen if:

$$4\gamma = n'b\, \{(\sigma_{\phi}^2 + \tau_{\phi}^2)^{\frac{1}{2}} + \sigma_{\phi} \sin\theta - \tau_{\phi} \cos\theta\} \qquad \text{Eq. 5}$$

The condition for the initiation of the crack Equation 2, must also be satisfied. In the Stroh model the crack is nucleated when the two leading dislocations in the pile-up coalesce, and from then on the stress required to cause the remaining dislocations in the pile-up to climb into the crack is less. Therefore, as soon as the crack is nucleated, $n' = n$.

Under an uniaxial tensile stress, Equation 4 has no real roots, and thus once nucleated the microcrack has no position of equilibrium and spreads catastrophically right through the material. Under an uniaxial compressive stress, Equation 4 has real roots, and thus once nucleated the microcrack comes to rest at a finite length.

The highly concentrated stress at the top of this stable crack is then damped by plastic deformation. So in deriving a criterion for elastic-plastic unstable fracture we must consider the energy dissipated in the plastic zone. The energy change of the system accompanying the extension of an elastic-plastic crack by the length ∂c under compressive stress is given, without the crack orientation factor, by an equation derived by Yokobori[10],

$$\delta E = \left| - \frac{4(1-\nu)}{\pi G} \sigma_y{}^2 a \left(\frac{\pi\sigma}{2\sigma_y} - \cos^{-1} \frac{c}{a} \right)^2 \left(\frac{\partial a}{\partial c}\right) + 4\gamma \right| \delta c, \qquad \text{Eq. 6}$$

where $a = c + s$ and s = length of the plastic zone. In the present article we have used the Hult and McClintock[11] result $\frac{c}{a}$ as a function of $\frac{\sigma}{\sigma_y}$ based on the mathematical theory of plasticity where σ_y = maximum yield stress. It is expressed by Bilby et al.[12] in the following simple form:

$$\frac{c}{a} = \frac{\pi}{2} \frac{1}{f\left(\frac{\sigma}{\sigma_y}\right)} \qquad \text{Eq. 7}$$

where

$$f\left(\frac{\sigma}{\sigma_y}\right) = \frac{1 + \left(\frac{\sigma}{\sigma_y}\right)^2}{1 - \left(\frac{\sigma}{\sigma_y}\right)^2} E \left\{ \frac{\pi}{2}, \frac{2\left(\frac{\sigma}{\sigma_y}\right)}{1 + \left(\frac{\sigma}{\sigma_y}\right)^2} \right\} \qquad \text{Eq. 8}$$

and

$$E \left\{ \frac{\pi}{2}, \frac{2\left(\frac{\sigma}{\sigma_y}\right)}{1 + \left(\frac{\sigma}{\sigma_y}\right)^2} \right\}$$

is the complete elliptic integral of the second kind.

If Equation 6 is now expressed in terms of $\left(\frac{\sigma}{\sigma_y}\right)$ we get:

$$\delta E = \left| - \frac{4(1-\nu)\sigma_y{}^2 c}{\pi G} \{F(\sigma/\sigma_y)\}^2 + 4\gamma \right| \delta c, \qquad \text{Eq. 9}$$

where

$$F(\sigma/\sigma_y) = f(\sigma/\sigma_y) \left| \frac{\sigma}{\sigma_y} - \frac{2}{\pi} \cos^{-1} \left\{ \frac{\pi}{2} \frac{1}{f(\sigma/\sigma_y)} \right\} \right| \qquad \text{Eq. 10}$$

The elastic-plastic brittle crack will propagate at a stress level corresponding to the minimum of Equation 9. This minimum will occur when $F\left(\frac{\sigma}{\sigma_y}\right)$ will reach its maximum. $F\left(\frac{\sigma}{\sigma_y}\right)$ takes a maximum value at $\frac{\sigma}{\sigma_y} = 0.6$ as shown in Figure 3.

Comparison with experimental results

The largest piled up groups can be obtained when the slip line extends across a grain; in this case, we must take L equal to half the grain diameter[7,13]. The most favoured slip lines are those at an angle of 45° to the load axis, so

$$\tau_\alpha = \frac{1}{2}\sigma$$

Inserting these values in Equation 1 we obtain:

$$\sigma_N = \left| \frac{3\pi\gamma G}{(1-\nu)d} \right|^{\frac{1}{2}} + 2\tau_o \qquad \text{Eq. 11}$$

The frictional stress, τ_0 opposing the motion of the dislocations on the glide plane is temperature independent[13] and its experimentally determined value was 3 kg/cm^2 for snow ice at the circumstance of crack nucleation. As nucleation is the critical event in tensile tests, the tensile brittle fracture stress is given by Equation 11 where the only temperature dependance comes through G. As it is seen from Figure 4, the agreement between experimental data and values calculated from Equation 11 is very satisfactory.

Figure 5 shows the drop-off in compressive strength with increasing strain rates in the ductile to brittle transition region. Above a critical strain rate, in the brittle region, the ultimate compressive stress is strain rate independent and the theoretical 0.6 ratio is very well verified. Tests were performed on cylindrical samples (1" x 3") and the physical constants of the snow ice used were:

density $\rho = 0.89$
Elastic modulus: $E = 52423 - 673 \times T(^{o}C)$ kg/cm^2
Shear modulus: $G = E \div 2.6$
Poisson's ratio: $\nu = 0.3$
grain diameter: $d = 0.1$ cm
surface energy[14]: $\gamma = 109$ erg/cm^2

CONCLUSION

Through a theoretical analysis based on energy balance considerations, the brittle fracture criteria for both tensile and compressive tests are proposed for snow ice. These criteria are in very good agreement with experimental data. Thus, by the use of dislocations theories, it has proved possible to rationalize the features of brittle behavior of snow ice.

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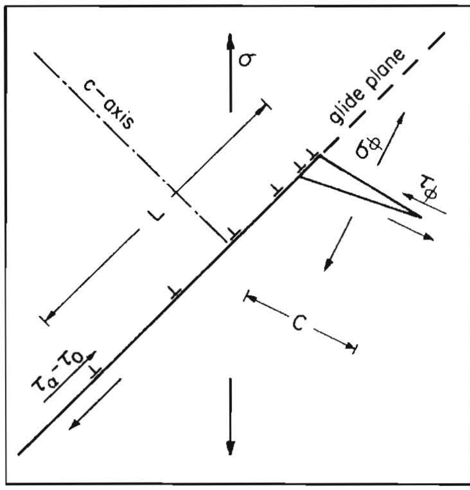

Fig. 1 — Schematic representation of the model used for crack initiation under a tensile stress.

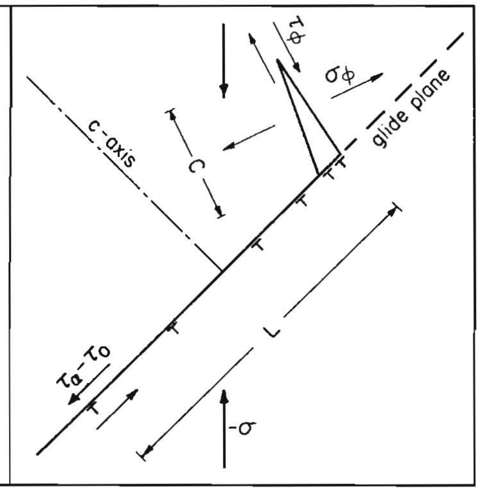

Fig. 2 — Schematic representation of the model used for crack initiation under a compressive stress.

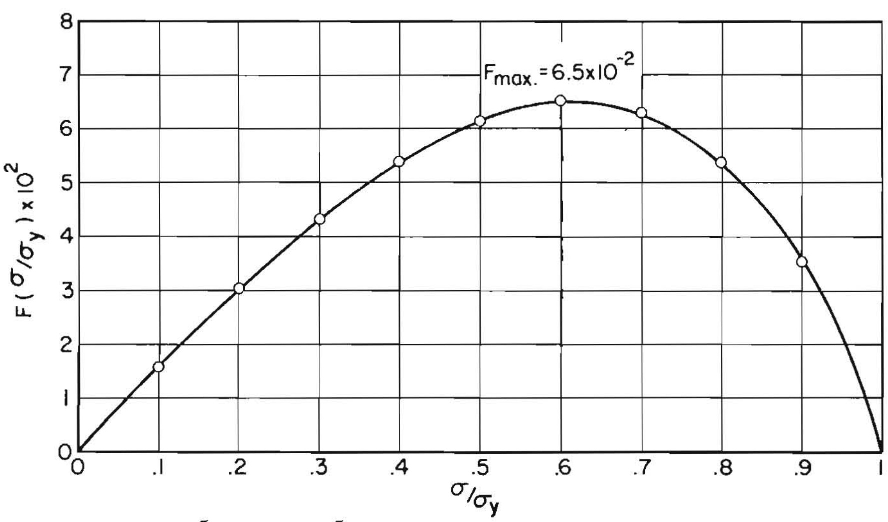

Fig. 3 — $F(\frac{\sigma}{\sigma_y})$ versus $\frac{\sigma}{\sigma_y}$ as calculated by equation 10

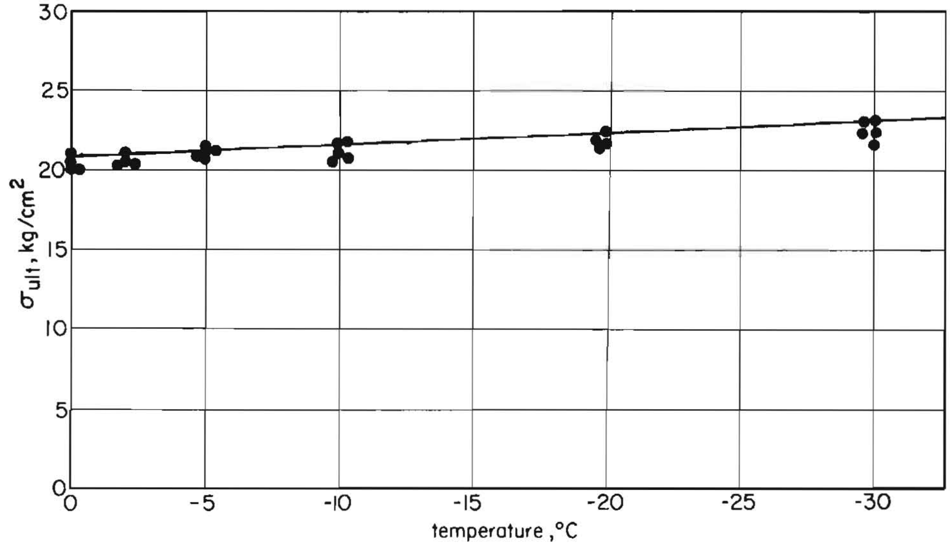

Fig. 4 — Tensile brittle stress versus temperature as compared to equation 11.

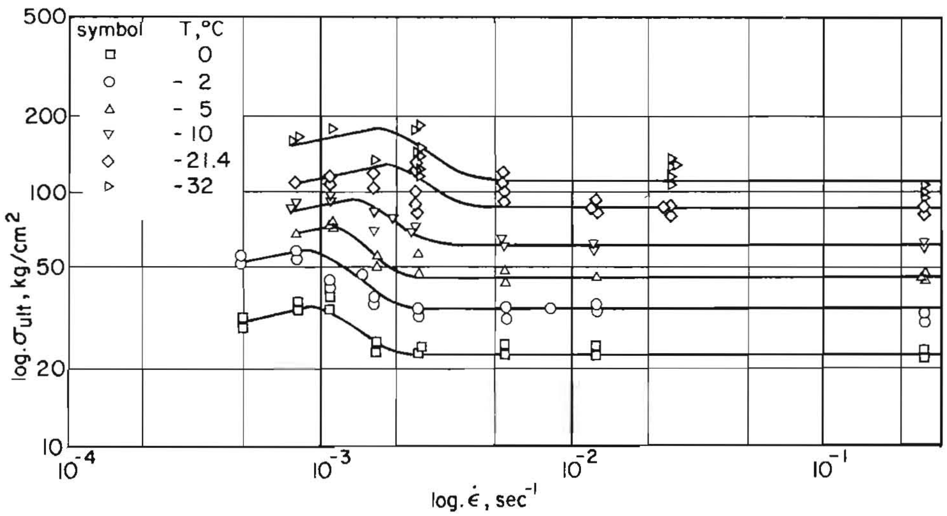

Fig. 5 — Compressive strength versus strain rates at various temperatures.

DISCUSSION by: L. GOLD

on paper by: CARTER 5-2

The theory that Mr. Carter has used to describe the failure of ice has been used with considerable success for several materials. I would, however, suggest that caution be used in its application to ice. The theory describes the conditions associated with a propagating crack. Failure in ice often involves the linking up of previously formed cracks, or severe deformation and ultimate failure of material between cracks. In laboratory tests, the final failure in compression is usually associated with the propagation of a failure zone, rather than a single crack, originating in the vicinity of the loading plate (i.e. in a region of stress concentration). These characteristics of failure have to be taken into consideration in the theoretical analysis of laboratory studies of the strength of ice.

CLOSURE by: D. CARTER

The type of failure described by Dr. Gold corresponds exactly to ductile fracture of ice. This failure occurs for strain rates below the ductile-to-brittle transition when fracture takes place by the plastic tearing of the bridges between dislocation cracks. Above the ductile-to-brittle transition, failure occurs by axial cleavage fracturing. So the criterion for quasi-brittle fracture of ice must be associated with the propagation of an elastic-plastic crack.

ICE SYMPOSIUM 1970
REYKJAVIK

ADFREEZING STRENGTH OF ICE

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SYNOPSIS

In order to fully utilize facilities on ice, it is necessary to determine the strength of structural members in ice for use as bollards, anchors, and piling. The adfreezing strength of freshwater ice, sea ice, and freshwater-sand slurry ice were determined using steel pipe, wood cylinders, steel box beams, and WF steel H-beams. In all cases the adfreezing strength increased with decreasing temperature; it was considerably lessened, however, after being subjected to long term loads. Results of these tests also showed that pile type and shape, as well as type of ice and temperature, determined adfreezing strength.

INTRODUCTION

In order to fully utilize facilities on ice such as the natural fast-ice wharf at McMurdo Station, Antarctica,[1] it is necessary to determine the strength of structural members in ice for use as bollards, anchors, and piling. Some information is available on the tangential adfreezing strength of structural members in permanently frozen ground, but little is available on their strength in ice. This paper presents the results of rapid and long-term pullout tests on different types of piles frozen into low-salinity ice and backfilled with freshwater, freshwater-sand slurry and seawater.

TEST PROCEDURE

Piles were set in 1.6$\pm$ 0.2 ppt salinity ice which was maintained at the test temperature in a walk-in cold chamber. Holes were drilled with an 11-cm-diameter dry auger; after placement of the pile, the hole was backfilled with prechilled freshwater, 34-ppt seawater, or a 1:3 freshwater-sand slurry.

The piles, which were 45-cm-long, were fabricated from 7.3-cm-diameter painted steel pipe, 7.3-cm-diameter straight dry-wood cylinders, 6.0- by 7.5-cm steel box beams, and 7.5-cm (5.8 by 6.0-cm) WF steel H-beams. A steel plate with an eye was welded to one end of each pipe, box, and H-beam. The pipe and box beams were sealed with a steel plate welded to the bottom. A steel rod with an eye on one end was inserted through a coaxial hole in each wooden cylinder and the bottom of the rod was welded to a plate with a diameter slightly less than that of the cylinder.

The short-term laboratory pile-pulling apparatus consisted of a 100-cm-long, 15-cm (6-inch) WF beam with a steel padeye in the center for connection to the piles. One end of the beam rested on a 22,680-kg (50,000-lb) capacity load cell and the other was lifted by the ram of a 9070-kg (20,000-lb) capacity hydraulic jack.

The long-term load apparatus consisted of a beam connected at one end to the padeye of the piles and loaded at the other end with weights. This was balanced on a sharp-edged steel sawhorse. Placement of the beam on the sawhorse and the amount of weight determined the pullout force on the pile. Long-term loads were run for two weeks.

RESULTS

Short-Term Tests

The tangential adfreezing strength versus temperature for each of the piles with freshwater backfill is shown in Figure 1, and with freshwater-sand slurry backfill in Figure 2. All of the piles show an increase in pullout strength with decreasing temperature. Since the box and H-piles were the same material, the adfreezing strength should be the same; the difference in tangential adfreezing strength then, is due to friction, to some influence due to shape, or to some other factor or combination of factors.

The wood piles developed the highest strength with a freshwater backfill, probably because the ice crystals penetrated partially into the wood; with a slurry backfill, which has less ice, the tangential adfreezing strength of the wood was less than the wood or the H-pile with a freshwater backfill. With both types of backfill the pipe and box piles had the lowest tangential adfreezing strength. With a slurry backfill the tangential adfreezing strength of the pipe, wood and H-piles were all quite close at warm temperatures but the difference increased with decreasing temperatures.

The H-piles were also tested with a seawater backfill and after being subjected to long-term loads; these results are compared with the results of tests with freshwater and freshwater-sand slurry in Figure 3. The tangential adfreezing strength of the piles with seawater backfill was about one-third that of the freshwater, even less than the piles subjected to long-term loading. Long-term loading decreased the bearing capacity to about two-thirds the tangential adfreezing strength of the freshwater backfill, even though the piles may not have pulled out during the long-term tests.

The use of a tapered wooden pile butt end down, or an H-pile with a cross bar between the flanges (Figure 1) resulted in an increased in pullout strength such that the ice broke rather than failing the pile-ice contact. The desirability of increasing the pullout strength of a pile in this way depends on the direction of the load and whether ground failure or pile failure is more critical

Long-Term Tests

The creep and failure of the piles at -1.7°C are shown in Figure 4, and at -12°C in Figure 5. At -1.7°C, the smallest load at which failure occurred was 0.54 kg/cm^2 after 25 hours, or 5% of the tangential adfreezing strength (Figure 1). No creep occurred at a load of 0.05 kg/cm^2. With 10-degree decrease in temperature to -12°C, the creep rate was very low until a load greater than 1.6 kg/cm^2 was applied. The smallest load at which failure occurred at -12°C was 2.3 kg/cm^2 after 151 hours or 14% of the tangential adfreezing strength. At that temperature, no creep occurred at loads less than 0.58 kg/cm^2.

The loads on two piles at -12°C were increased after 173 hours from 0.73 kg/cm^2 to 2.5 kg/cm^2 and from 1.3 kg/cm^2 to 2.9 kg/cm^2. The latter failed withint 20 hours after the increased load was applied. The piles with the load of 2.5 kg/cm^2 continued to creep 0.3 cm but did not fail during the next 186 hours, after which the test was terminated. This was greater than the minimum load applied at one time that failed a pile (2.3 kg/cm^2) and was equal to one which failed in less than 46 hours. This indicates that a greater load can be sustained without failure when applied in increments.

Comparison with Piles in Permafrost

All of the NCEL piles showed an increase in tangential adfreezing strength with decreasing temperature. This temperature dependence has also been noted by Linell[2] with piles in permanently frozen ground.

Although others have determined tangential adfreezing strength in specific situations, insufficient data are generally provided to make comparisons; using Russian data on wooden piles[3] and data from this study, however, Figure 6 was developed. The NCEL tests show that maximum strength occurs with a pure ice backfill, while the Russian data indicate that the maximum adfreezing strength generally occurs when a slurry is saturated with ice. The importance of water in the backfill to act as an adhesive agent was also illustrated by Crory[4] shown here as Figure 7.

The difference in data may be due to some influencing factor that was not taken into consideration in the comparison. Results of the NCEL tests show that the pile type, shape and treatment as well as backfill determines adfreezing strength, as exemplified by the difference in tangential adfreezing strength between the steel box and H-piles. The influence of other factors on the tangential adfreezing strength of wooden piles was pointed out in tests by Linell and others[2] in which creosoted piles developed 20% less strength than untreated ones. Stallabrass and Price[5] observed that surface contamination and water impurities reduced adhesion of ice to metal.

After the NCEL long-term loading, the tangential adfreezing strength was reduced about one-third. Similar results have been obtained by Crory[6] with piles in permafrost. Tests of two 10BP57 piles driven into permafrost with an average temperature about -0.60C failed after long-term loading at 1.3 and 1.5 kg/cm^2, which compares closely with the tangential adfreezing strength of the NCEL WF H-piles after long-term loading (Figure 3). Pipe piles pulled out after long-term loading to failure[4] (measured as gross settlement greater than 3.8 cm) had a bearing strength less than half the prefailure strength. Linell[2] found that piles subject to loads for two weeks or more failed at a lower short-term stress than piles subject to a load for one day or less.

At the warmer temperature of -1.7C, the rate of creep of the NCEL piles under long-term load increased with time (Figure 4); this was also observed with larger H-piles[7] in later NCEL tests. Linell[2] observed that piles in warm permafrost subject to long-term loads developed increasing creep rates many days after load application and even after appearing to show decreasing creep with time. Eventually those piles that showed reversals in creep rate failed.

The similarity in results of long-term loading and rapid pullout tests of piles in both ice and permanently frozen ground indicate that, in general, what is applicable in permafrost also applies for piles in ice, but not necessarily in snow. Kovacs[8] reports that closed-end piles driven into snow support most of the load through end-bearing rather than skin friction and adhesion. They do, however, demonstrate the same creep with time and temperature dependence.

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Discussion By: T. M. Dick

On Paper By: N. S. Stehle 5-3

In practical problems such as described by Mrs. Stehle it would seem important to consider the mechanics of the anchor which depends largely on surface tangential stress for its strength.

Clearly the diameter of the drilled hole with respect to the size of the pile is possibly an important factor. A large volume of water would expand more and induce a greater normal pressure as the ice expands. However, because of the face surface the pressure may be relieved by plastic flow. It would be of interest to discover if the effective strength for pullout is significantly improved by increasing the diameter of the hole.

Reply to Discussion By: T. M. Dick

By: N. S. Stehle

The influence of the expansion pressure of ice may indeed be significant; in these tests we found piles of the same material - pipe and H-piles - had significantly different tangential adfreezing strengths. This was attributed primarily to the H-shape which would be affected by the expansion pressure of ice on all surfaces, as opposed to the pipe which was exposed to the ice on the outside surface only.

Any advantage obtained from an increase in pullout strength due to expansion pressure from a greater amount of ice would probably be offset by the increased amount of potentially weaker material. In the laboratory tests we found that ice failure would occur rather than pile-ice contact failure during low temperature tests with 6 inches or more backfill. In those ice failure cases where the hole size was nearly the size of the pile, failure occurred in the tank ice; as the diameter of the hole was increased relative to the pile diameter, failure was more likely to occur in the backfill ice.

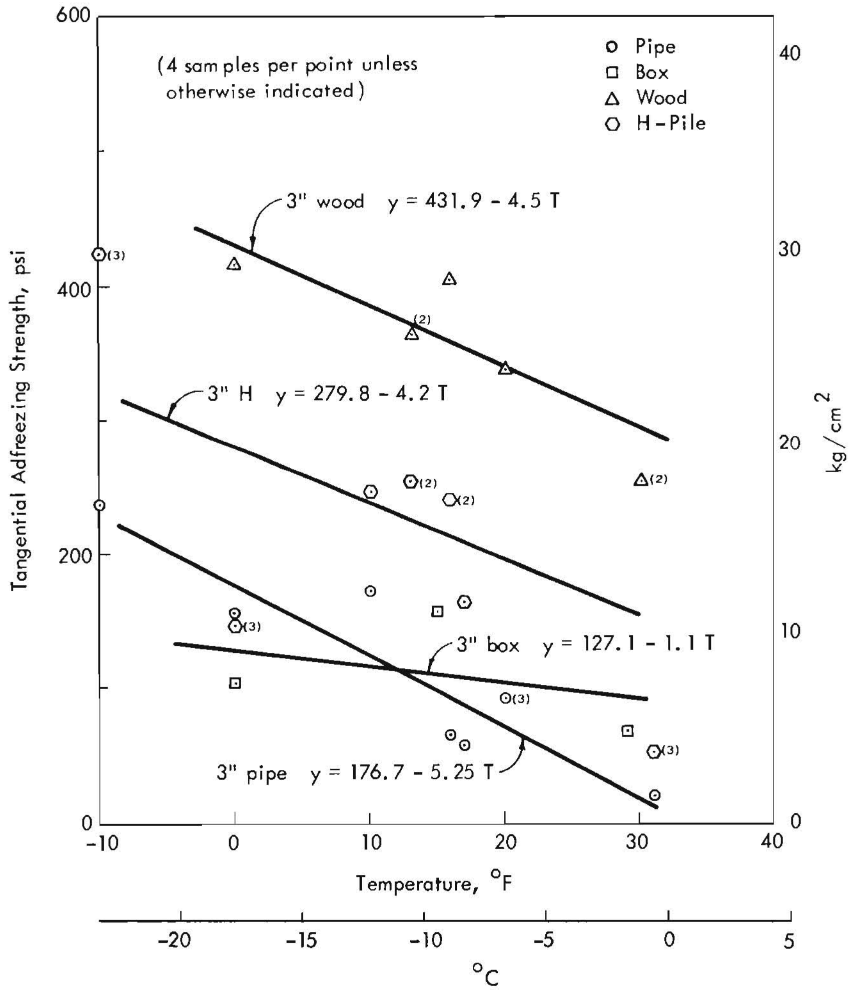


Figure 1. Tangential adfreezing strength of 7.5 cm piles with freshwater backfill.

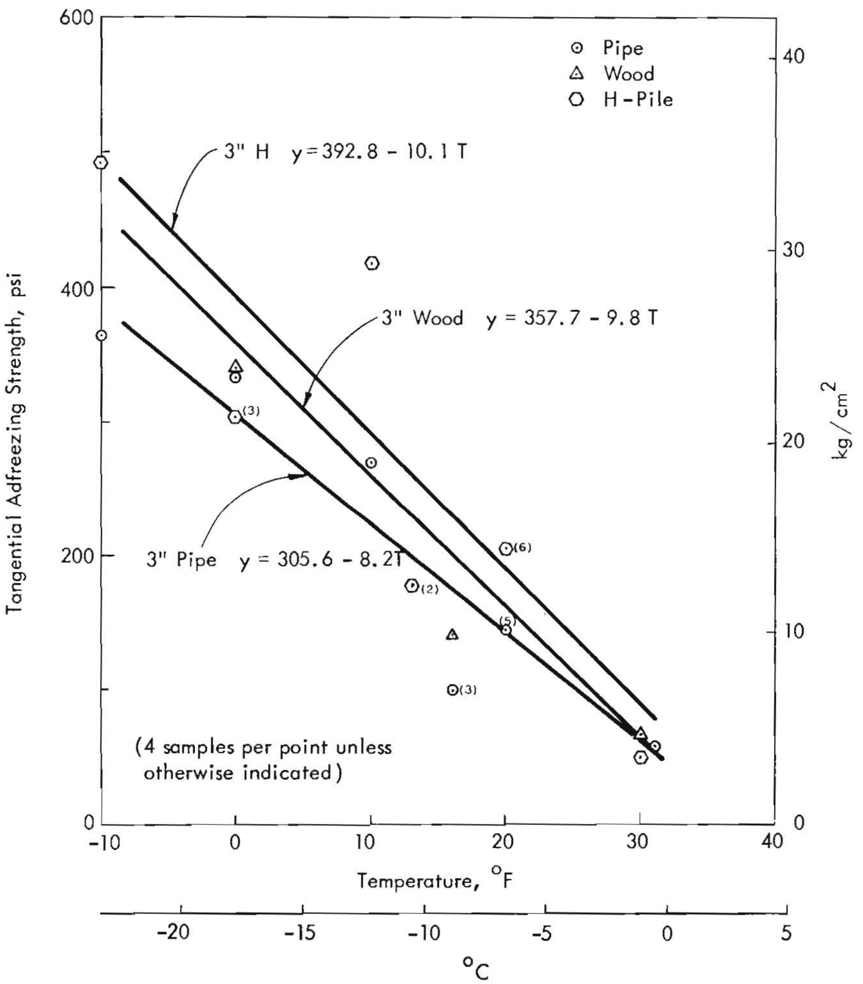


Figure 2. Tangential adfreezing strength of 7.5 cm piles with slurry backfill.

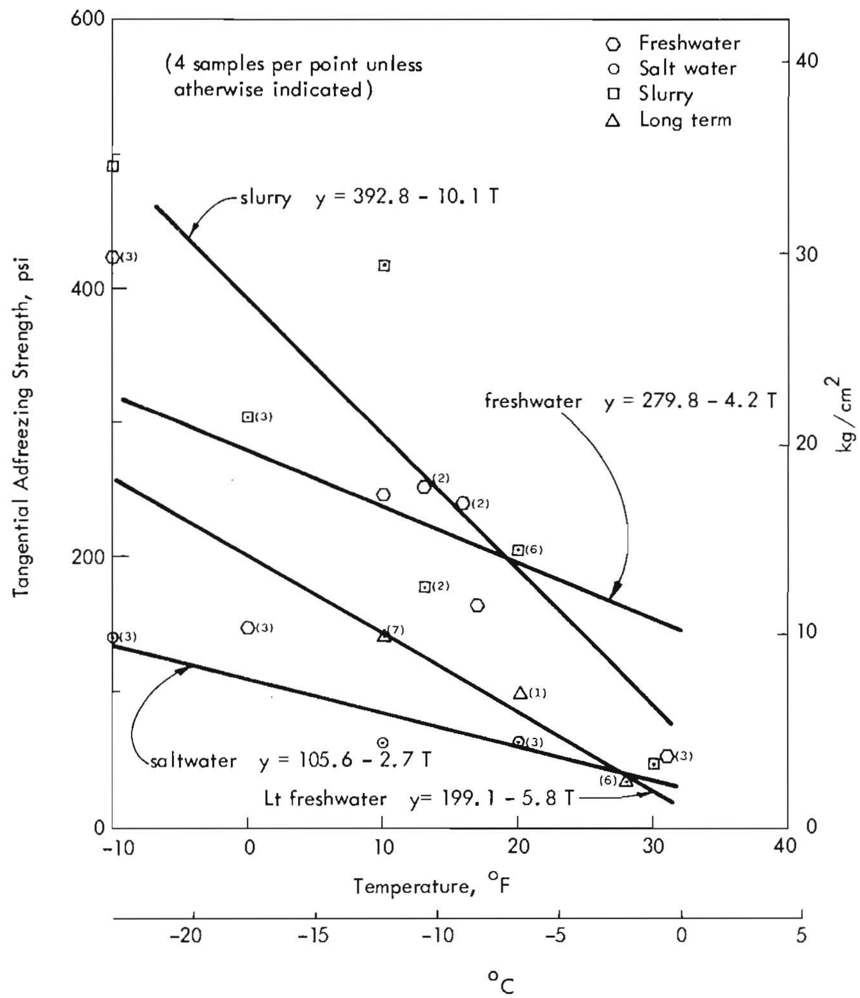


Figure 3. Tangential adfreezing strength of 7.5 cm H-piles with different backfills.

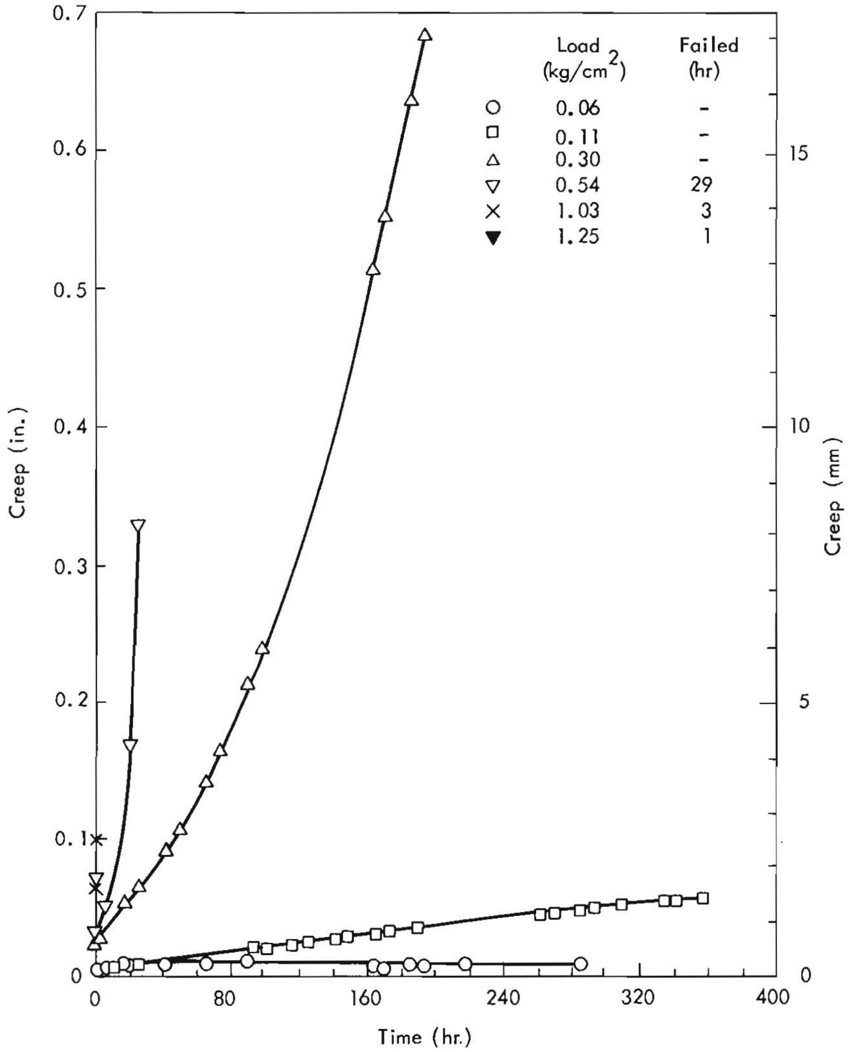


Figure 4. Long-term creep of 7.5 cm H-piles at -1.7° C.

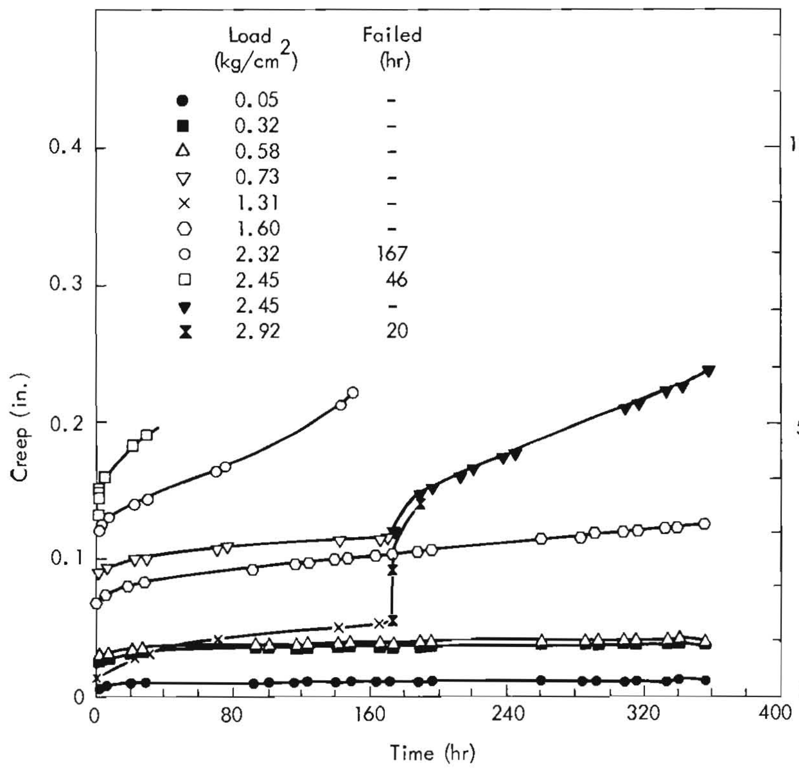


Figure 5. Long-term creep of 7.5 cm H-piles at -12° C.

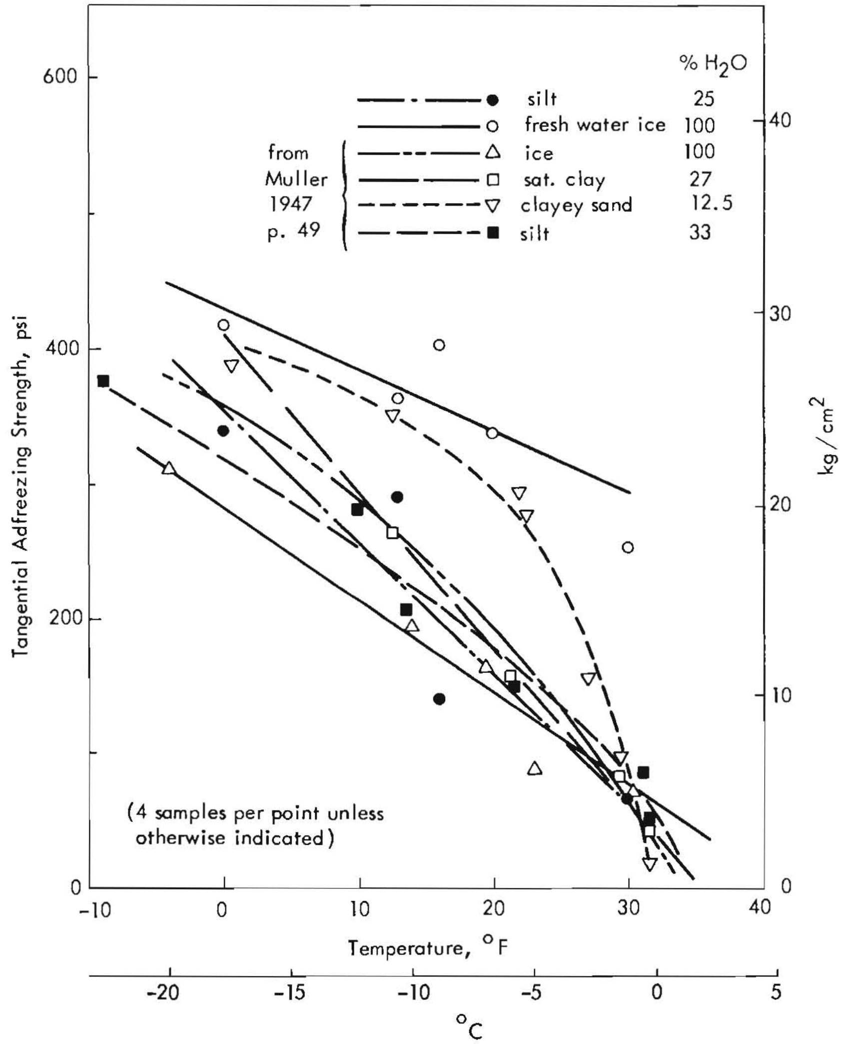
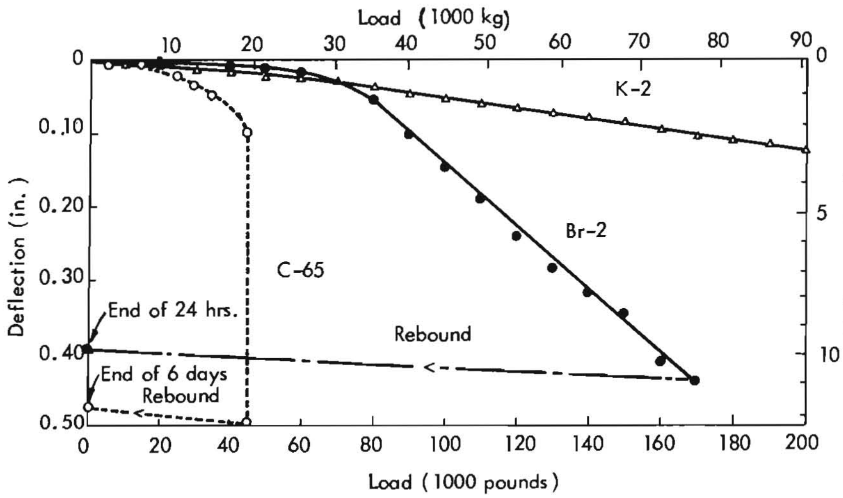


Figure 6. Tangential adfreezing strength of wooden piles.



C-65 (Fairbanks), dry sand backfilled 6WF25; length in permafrost 10.0 ft. Average permafrost temperature 30.4°F; loaded at 5000 pounds/4 days, 45 thousand pound load held total of 22 days. Unloaded in one minute.

BR-2 (Bethel), driven 8BP36 section in fine silty sand; length in permafrost 16.5 ft. Average permafrost temperature 30.9°F; rate of loading 10,000 pounds/day. Unloaded in five minutes.

K-2 (Kotzebue), sand-slurried 8-inch pipe; length in permafrost 11.0 ft. Average permafrost temperature 27.8°F; load initially at 10,000 pounds/day. Unloaded and loaded in 10 minute cycles.

Figure 7. Load settlement tests in frozen sand.

ICE SYMPOSIUM 1970
REYKJAVIK

EXPERIMENTAL RESEARCH FINDINGS ON DECREASE OF RIVER ICE STRENGTH IN SPRING

RESULTATS EXPERIMENTAUX SUR L'ABAISSEMENT DE LA RESISTANCE DE LA GLACE DE RIVIERE AU PRINTEMPS

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The B.E.Vedeneev All-Union Research Institute of Hydraulic Engineering (VNIIG), Siberian Branch, Krasnoyarsk, U.S.S.R.

Synopsis

Considered is the effect on the ice cover strength of variations in hydrometeorological characteristics in spring. Solar radiation is shown to play a predominant role in the reduction of ice strength. A relationship is presented for practical control of the decrease in ice strength obtained during bending tests on cantilever specimens.

Résumé

Le rapport traite le problème de l'influence de la variation des caractéristiques hydrométéorologiques au printemps sur la résistance de la couverture de glace. Le rôle déterminant de la radiation solaire dans la diminution de la résistance des glaces est demontré. Pour le contrôle pratique de l'abaissement de la résistance des glaces on propose une relation déduite lors des essais sur la flexion des éprouvettes en console.

Variations in hydrometeorological characteristics in spring induce a decrease in ice strength. Field observations conducted on the Ob and Yenisei rivers /R.1,2/ confirm the conclusion on the predominant effect of solar radiation on reducing ice strength in spring.

Figure 1 shows the results of comparison of relative bending strength values obtained during tests on $h \times h \times 3h$ cantilever ice specimens (h –ice cover thickness) with relative values of specific integral short-wave radiation heat absorption.

The value of S_o denotes specific crystallization heat of ice and R_o– bending strength of cantilever ice specimens at 0^{o}C unaffected by solar radiation.

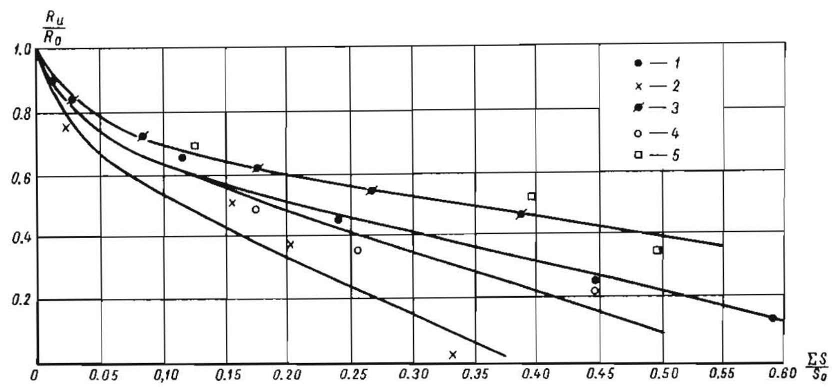

Fig.1. Dependence of relative values of bending strength of ice $\left(\frac{R_u}{R_o}\right)$ in spring on the values of specific integral short-wave heat absorption ($\frac{\Sigma S}{S_o}$). 1–three-layer ice (new-snow ice h = 15 — 5 cm with coal dust interlayer, finely crystalline ice h = 20 cm, coarsely crystalline ice h = 60 — 50 cm, with air bubbles over the thickness of the 2-nd and 3-rd layers and at their boundary); 2 — two-layer ice (finely crystalline ice h = 30 — 15cm, coarsely crystalline ice h = 60 — 55 cm, with air bubbles over the thickness and at the boundary of the layers); 3 — two-layer ice (new-snow ice h = 35 — 20 cm, coarsely crystalline ice h = 40 — 35 cm, with air bubbles over the thickness of both layers); 4 — three-layer ice (new-snow ice h = 15 — 5 cm, brownish crystal autumn ice h = 20 — 35 cm, coarsely crystalline ice h = 10 — 25 cm, with water in the thickness of the ice cover); 5 — three-layer ice (new-snow ice h = 15-10 cm, frazil ice h = 25 — 30 cm, coarsely crystalline ice h = 10 — 15 cm).

At $\frac{\Sigma S}{S_o} < 0.1$ the rate of decrease in ice cover strength attains a maximum and is nearly independent of the ice structure. At $\frac{\Sigma S}{S_o} > 0.1$ the effect of the ice cover structure on the rate of reduction in the strength of ice becomes predominant.

Differences in the amount and nature of solar radiation absorption by the ice cover are attributable to various degrees of the transparency of ice, which in its turn depends on the ice structure. This governs the development and specific features of volumetric melting of ice in spring, which causes a loss in strength. Snow precipitation on the ice cover may completely stop lowering of ice strength.

Solar radiation absorbed by ice may be a detriment to its strength only at certain values of the non-radiation components of the heat balance. Further experiments are to refine this statement as well as to reveal the physics of breaking up of ice due to solar radiation effect.

For practical control of the decrease in ice strength the following relationship may be used

$$R_u = R_0 - \sum_{i=0}^{i=n} K_i \Delta\tau_i$$

whose pattern was proposed by N.N.Petrunichev /R.4/. The value of K_i designates a reduction in ice strength during 24 hours in the period $\Delta\tau_i$, with K_i assumed constant. The R_0 and K_i -values should be estimated experimentally for each region. On the basis of field observation data on the Yenisei river indicated in Fig.2 the value of R_0 is assumed equal to 55 t/m^2; K_i-values are presented in the Table.

Table

Ice cover structure	Average daily temperature t_{av} in period of record $\Delta\tau$	K_i kg/cm^2 per 24 hrs
One-layer crystal ice h = 0.5 — 0.8 m without snow	Fair with some cloud $t_{av} > -5^{\circ}$C	0.45 — 0.55
Same	Solid low cloud cover $t_{av} > -5^{\circ}$C	0.06
Same but with 5 — 10 cm thick snow cover on ice	Fair with some cloud $t_{av} > -5^{\circ}$C	0.03
Frazil ice without snow	Same	0.18 — 0.20
Two-layer ice h = 0.5 — 0.8 m (new-snow ice h = 15 — 25 cm, crystal ice h = 40 — 60 cm) without snow	Same	0.25
Same but with 5 cm thick snow cover on ice	Same	0.35

The data pertaining to ice cover structure, prediction of precipitation trend and daily average air temperatures allow to regard the above relation at definite values of R_o and κ_i as prognostic, which circumstance is essential in effecting control over ice crossing maintenance in spring, in evaluating possible dynamic ice pressure against structures and in assessing the passage of ice through hydraulic structures and bridges both under construction and operation.

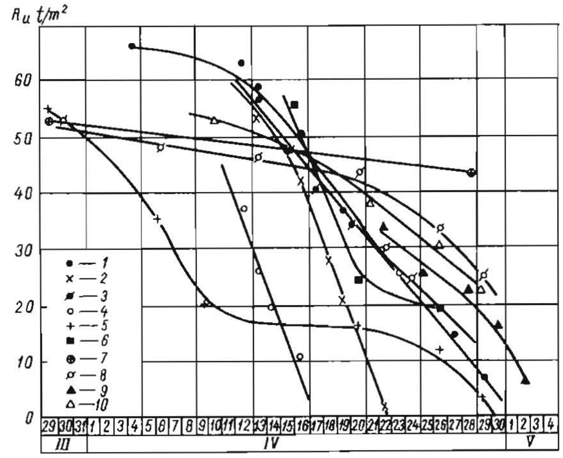

Fig.2. Variation in ice strength according to bending tests on cantilever specimens obtained during a 6-year period on the Yenisei river in spring.
1-4-ice whose structure is shown in Fig.1;
5-6 - one-layer crystal ice h =45-65cm;
7 - one-layer crystal ice h = 45-65 cm in a reach unaffected by direct solar radiation;
8 - two-layer ice (new-snow ice h = 25-15 cm, crystal ice h = 60-40 cm);
9, 10 - three-layer ice (new-snow ice h = 15-10 cm, frazil ice h = 30-50 cm, coarsely crystalline ice h = 10-20 cm.)

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ICE SYMPOSIUM 1970
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IMPACT PENETRATION OF ARCTIC SEA ICE

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INTRODUCTION

Materials such as ice when subjected to dynamic loads experience extreme strains and concomitant changes in both density and phase. To investigate pertinent problems, it is necessary to develop large-deformation theory techniques. Under these conditions, solutions must be obtained by extensive use of numerical calculation schemes, and recourse to computer codes became inevitable. These codes provide information concerning strains, displacements, and stresses in both the ice slab and structure,and for suitable fracture criteria; yield-failure surfaces, break up geometries, and critical impact velocities. In constructing a code, expressions representing the conservation of momentum (i.e., equation of motion), mass (i.e., equation of continuity),and energy are written in terms of finite deformation velocities. In addition, constitutive relations describing material behavior are required. The present large-deformation code (CANDIA CODE) can treat both dissimilar structures and targets, and the case of multilayered media. Moreover, one can include anisotropic and inhomogeneous physical behavior, compactibility under hydrostatic stress, and elastic-plastic behavior in shear. The CANDIA CODE[1,2,3] can be described as a two-dimensional (2 space variables, 1 time variable), axisymmetric, large-deformation theory, elastic-plastic, finite-difference, artificial viscosity type code.

Basic Theory.--For the projectile-slab perforation problem at normal incidence, the differential equations of motion in terms of stresses under the assumption of cylindrical symmetry are

$$\frac{\partial\sigma_r}{\partial r} + \frac{\sigma_r - \sigma_\theta}{r} + \frac{\partial\tau_{rz}}{\partial z} = \rho\dot{u} \qquad \frac{\partial\sigma_z}{\partial z} + \frac{\partial\tau_{rz}}{\partial r} + \frac{\tau_{rz}}{r} = \rho\dot{w} \tag{1}$$

where components of particle velocity in the r and z directions are given by u and w, and dots denote partial differentiation with respect to time along the particle path. Material density, ρ, can be a function of r, z, and t. The equation expressing conservation of mass is

$$-\frac{1}{\rho}\dot{\rho} = \frac{\partial u}{\partial r} + \frac{\partial w}{\partial z} + \frac{u}{r} \tag{2}$$

An appropriate stress-strain relationship can be represented in terms of a dilatational part and a deviational part,

$$p = -\frac{1}{3}\left(\sigma_r + \sigma_\theta + \sigma_z\right)$$

$$S_r = p + \sigma_r \qquad S_\theta = p + \sigma_\theta \qquad S_z = p + \sigma_z \qquad S_{rz} = \tau_{rz} \tag{3}$$

where p is the hydrostatic stress and S_{ij} the stress deviator.

Nonzero components of the strain rate tensor are $\dot{\epsilon}_r, \dot{\epsilon}_\theta, \dot{\epsilon}_z$, and $\dot{\epsilon}_{rz}$, and the relationships between strain rates and particle velocity components are

$$\dot{\epsilon}_r = \frac{\partial u}{\partial r} \qquad \dot{\epsilon}_\theta = \frac{u}{r} \qquad \dot{\epsilon}_z = \frac{\partial w}{\partial r} \qquad \dot{\epsilon}_{rz} = \frac{\partial u}{\partial z} + \frac{\partial w}{\partial r} \tag{4}$$

In an analogous manner, the strain rate tensor can be cast in dilatational and deviational parts as follows

$$\dot{e} = \frac{1}{3}\left(\frac{\partial u}{\partial r} + \frac{u}{r} + \frac{\partial w}{\partial r}\right)$$

$$\dot{\epsilon}'_r = -\dot{e} + \dot{\epsilon}_r \qquad \dot{\epsilon}'_\theta = -\dot{e} + \dot{\epsilon}_\theta \qquad \dot{\epsilon}'_z = -\dot{e} + \dot{\epsilon}_z \qquad \dot{\epsilon}'_{rz} = \dot{\epsilon}_{rz} \tag{5}$$

For an elastic material, the linear stress-strain or constitutive relationships can be expressed as

$$\dot{S}_r = 2G(\dot{\epsilon}_r - \dot{e}) \qquad \dot{S}_\theta = 2G(\dot{\epsilon}_\theta - \dot{e}) \qquad \dot{S}_z = 2G(\dot{\epsilon}_z - \dot{e}) \qquad \dot{S}_{rz} = G\dot{\epsilon}_{rz} \tag{6}$$

for the deviator parts of the stress-strain tensor, and

$$p = 3Ke \tag{7}$$

for the dilatational part. If the material is elastic, ideal-plastic, the stresses are assumed to be restricted by the Von Mises yield condition. This relationship, together with appropriate flow rules, defines the elastic-plastic constitutive relations. In many cases the response under rapid loading is nonlinear. In particular, for certain materials which are described as being compactible or subject to locking deformation,[4-8] the stress-strain relationship in dilatation can be represented analytically by a power series.

Method of Numerical Integration.--The complete set of equations is integrated numerically using a method due to Von Neumann and Richtmyer[9,10]. First, for a fixed instant in time, the equations of motion are resolved to yield particle velocities. Then, these quantities are translated into strain rates through the kinematic relations whereupon introduction of appropriate constitutive equations yields the stress rates. Subsequent integration of the stress rates results in the stress quantities themselves. At this point, stresses are compared with the yield criterion to ascertain whether plastic deformation occurs and, if so, suitable corrections are made to adjust the stresses so that they will lie on the yield surface. During the calculations, special consideration is directed to boundary meshes and whether free surfaces or interfaces are being considered.

Sample Problem.--Results obtained from calculations for ice impact by a blunt cylindrical projectile at normal incidence are shown in Figs. 1, 2, and 3. Figure 1 illustrates variations in shear stress in both penetrator and slab 60.11 μsec, after impact. Similar results for normal stress in the axial direction are given in Figure 2. Development of fracture surfaces in the target material* is portrayed in Figure 3. All results are plotted for radial planes through the central axis of impact and curves are symmetric about this axis; thus, only left-hand results are mapped.

In Figure 1 the darkened stress wave or isocontour delineates the extent of regions that have experienced shear stresses equal to or greater than the shear yield stress of sea ice. As expected, shear stresses are greatest in magnitude at the continuous corner which marks the outer edge of the penetrator. The normal stress distributions shown in Figure 2 are at earlier times ($t \approx 20$ μsec) all compressive and isocontours are uniform and uncomplicated with peak stresses occurring close to the penetrator-target slab interface and the central axis of impact. Later, a region of tensile stress is developed from the free surface of the slab at a point removed in the radial direction from the contact area. The contours illustrate too that stress wave reflection has occurred and a second region of tensile stress is created close to the lower surface. Because sea ice is more readily susceptible to damage under tensile stress than compressive stress, the possibility of engendering failure near the lower surface is distinct. In this case, fracture is said to occur as a result of spalling or scabbing.

Figure 3 indicates progression of the fracture surface that develops in the slab as a result of impact. Initially, the shear stress near the penetrator corner and adjacent slab surface is greater than the assumed shear failure stress of sea ice;

Sea Ice: Density = 0.918 g/cc, Elastic Modulus = 0.273×10^6 psi, Poisson's Ratio = 0.29, Shear Strength = 70 psi Tensile Strength = 125 psi thus, a cylindrical-shaped crack is initiated and begins to propagate into the target material. Continued propagation of the crack occurs in the vertical direction within the region of high shear stress concentrated at the tip of the moving crack. At time $t = 62.85\ \mu sec$, the shear fracture surface has reached a depth equal to 0.8 times the sea ice slab thickness where maximum extent of the shear yield stress contour occurs and it is to be expected that an alternate mechanism of failure can take place. In fact, Figure 2 shows that a region of tensile stress has begun to develop in the same area and time period that characterize the furthest reach of shear failure stresses. Here, values of maximum principal stress exceed the tensile yield strength of sea ice; thus, tensile or cleavage fracture can occur in planes orthogonal to the directions of maximum principal stress. It is found that the cleavage surface proceeds along a path headed 26^{o} in azimuth from the axis of impact. When this surface reaches the sea ice slab bottom, incipient perforation is apparent, and passage of the projectile through the target material is preceded by ejection of a cylindrical-conical plug corresponding to the fracture surface geometry.

It is noted that the same cylindrical-conical shear plug configuration has been observed experimentally in instances of sea ice perforation by projectiles.[11,12] Moreover, for the impact velocity employed in the calculations (i.e., $V_1 = 7.7 ft/sec$) both theory and experiment[2] indicate that perforation would result. Furthermore, the theoretical relative proportion of cylindrical shear failure surface length to slab thickness corresponds closely with results observed in experiments on both laboratory and Arctic sea ice.[2]

At present, the CANDIA CODE is being employed to investigate problems in which a plane ice sheet interacts with a noncurved hydraulic structure such as a dam. In this case, two-dimensional plane stress and plane strain representations of the problem are being considered.

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*It has been assumed simply that fracture takes place when the maximum principal stress value exceeds the tensile fracture stress of the sea ice.

DISCUSSION by Cochard, Canada

on paper by Ross 5-5

Q. Can you, by using your analysis determine also the stress repartition for low rate of loading?

A. Yes, however as noted the present code treats only dynamic problems in which inertia forces are taken into account. To analyze static or quasistatic problems in which inertia is neglected but time varying loads can occur, we use a separate version of our code which solves the finite difference equations characterizing the problem by relaxation methods in lieu of the artificial viscosity method. At present, we have both plane stress and plane strain versions of a quasistatic code in addition to the axisymmetric dynamic code.

DISCUSSION by Gold, Canada

on paper by Ross 5-5

Q. Are there some conditions for which the shear type failure will extend through the full thickness of the ice cover?

A. Yes, when the sea ice thickness is large compared to projectile diameter and impact velocity is high, shear failure extends down through 90-95% of the slab thickness. On the other hand, when the sea ice is relatively soft and ductile; that is, warm temperature and high salinity or brine content, complete shear perforation takes place.

ILLUSTRATIONS

1. Isocontours of Shear Stress, τ_{rz}, Produced in Laboratory Sea Ice Slab and Aluminum Projectile as a Result of Impact at Normal Incidence (Dark Isocontour Outlines Shear Yield Region)
2. Isocontours of Normal Stress in the Axial Direction, σ_{zz}, Produced in Laboratory Sea Ice Slab as a Result of Impact at Normal Incidence (Shaded Portions Represent Areas of Tensile Stress)
3. Propagation of Shear and Tensile Fracture Surfaces in Laboratory Sea Ice at Succeeding Instants of Time

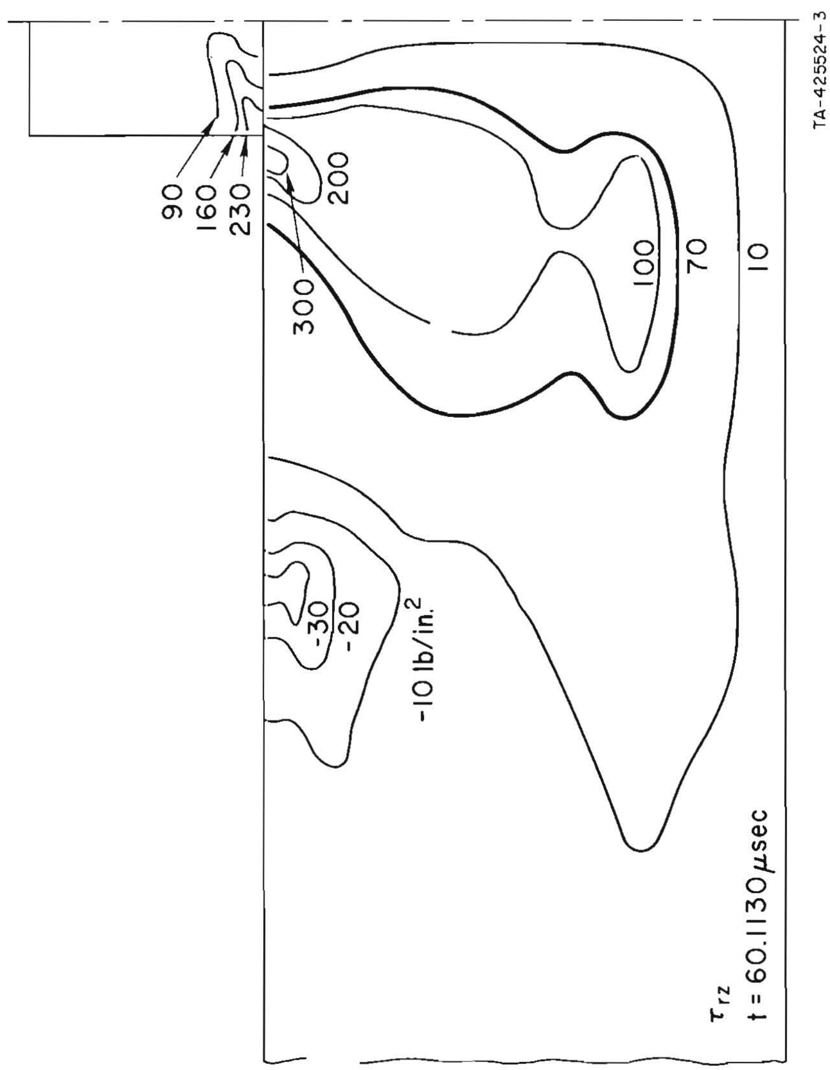
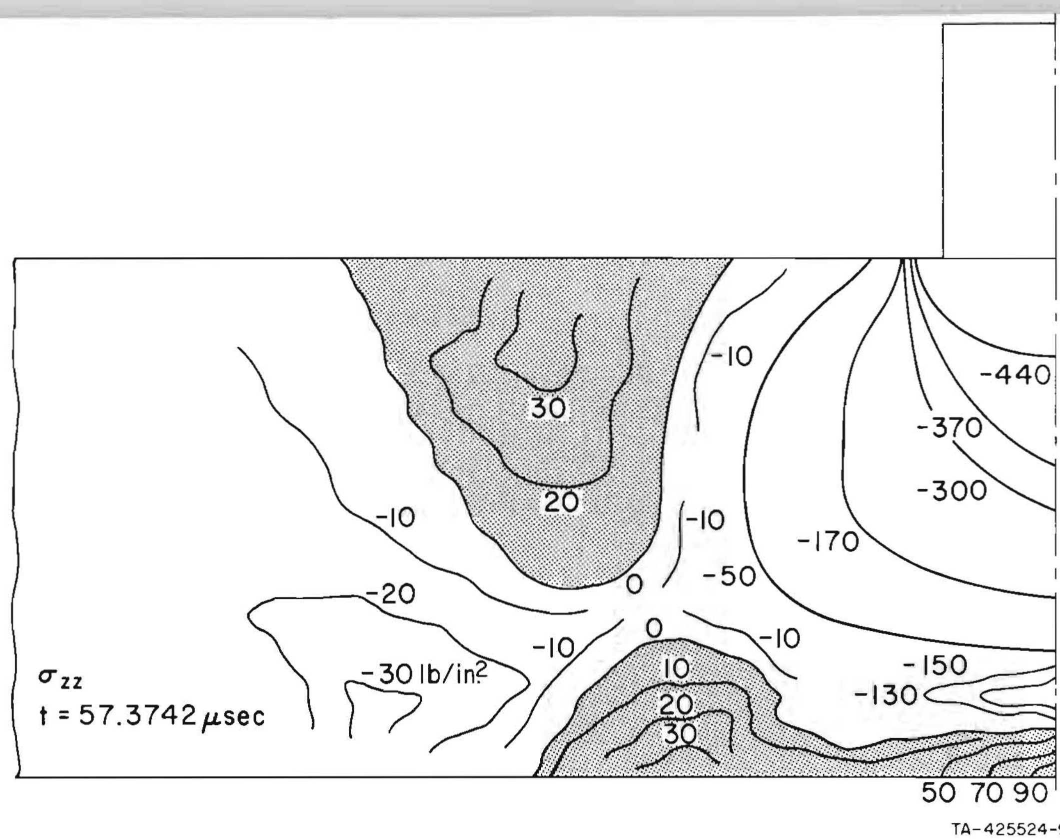

90
160
230
200
300
100
70
10
-30
-20
-10 lb/in.^{2}
τ_{rz}
t = 60.1130 μsec
TA-425524-3

FIGURE 1



-10
-440
30
-370
-300
20
-10
-10
-170
-50
-20
0
0
-10
-10
-150
σ_{zz}
-30 lb/in.²
10
-130
t = 57.3742 μsec
20
30
50 70 90
TA-425524-9

FIGURE 2

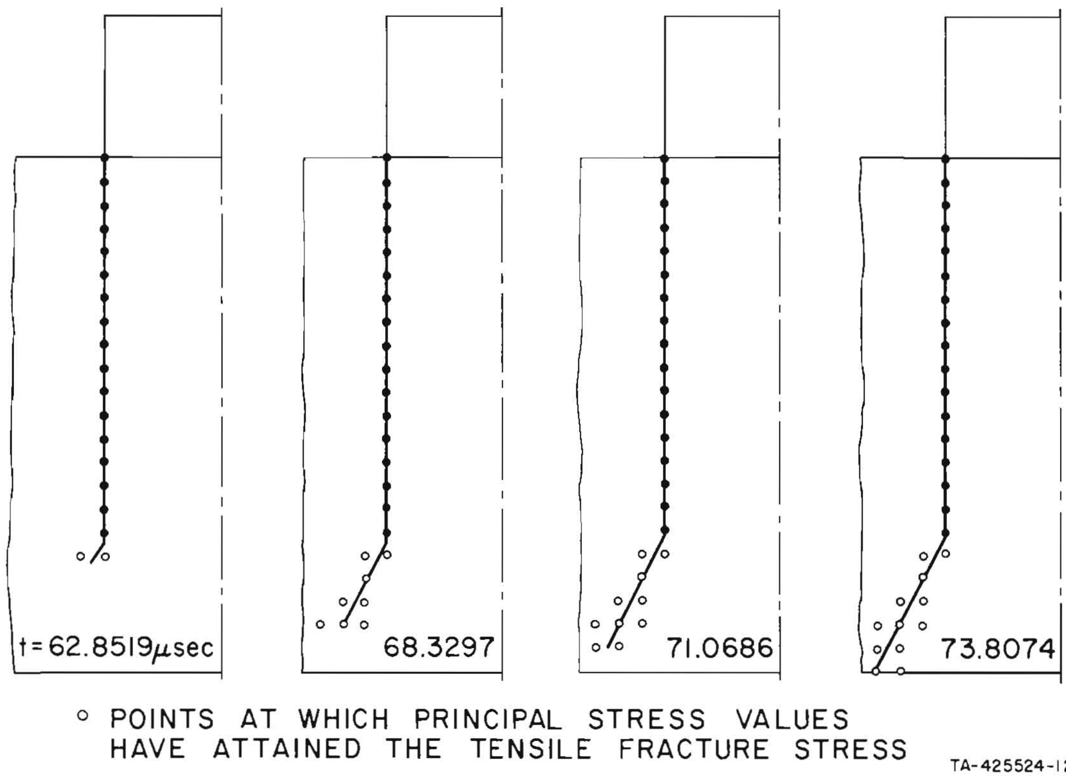

t=62.8519μsec
68.3297
71.0686
73.8074
○ POINTS AT WHICH PRINCIPAL STRESS VALUES HAVE ATTAINED THE TENSILE FRACTURE STRESS
TA-425524-12

FIGURE 3

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STATE OF RESEARCH ON ICE THERMAL THRUST

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SYNOPSIS

The thrust exerted by the thermal expansion of ice covers on hydraulic structures are still unknown. Up to now, researches are not conclusives and the suggested values cannot be scientifically accepted because they should take into account the crystallographic structures and the optical orientations of the different types of ice formed in nature. This paper presents the state of art of this question followed by a description of the actual research under way at Université Laval.

RESUME

Les poussées exercées par l'expansion thermique des champs de glace demeurent encore des inconnues en mécanique des glaces. Les recherches réalisées à date ne sont point concluantes et les valeurs suggérées peuvent difficillement se justifier. En effet, aucune étude ne fait état, d'une manière systématique, des structures cristallographiques et des orientations optiques des divers types de glace rencontrés en nature. Cette communication présente d'abord un état de la question suivi d'une description des recherches en cours à l'Université Laval.

STATE OF ART

In North America, it is generally accepted by designers to take values between 15-30 tons per linear meter (10-20 kips per linear foot) for this thrust. This range of values has principally its origin in the judgment and experience gained by engineers. On Figure 1, we have drawn thrust curves given by Rose[1], U.R.S.S. norms CN-76-59[2] and Drouin[3] (calculated from Monfore's[4] experimental values) for realistic conditions[5,6] of initial ice surface temperature (-40°C., -40°F.) and rate of temperature rise at the top of an ice sheet (2.8°C./h., 5.0°F./h.). As the degree of restriction of an ice cover is an unknown parameter; these curves are for the case of uniaxial restriction only. If we compare values for two thicknesses (45, 90 cm; 17.6, 35.2 po.) of ice we get the following magnitudes:

AUTHOR	THRUST Without lateral restraint and solar energy			
	Tons per linear meter		Kips per linear foot	
	h = 45 cm.	h = 90 cm.	h = 17.6 in.	h = 35.2 in.
ROSE	4.8	8.8	3.2	5.9
FROM MONFORE'S VALUES	22.6	23.6	15.2	15.8
CN-76-59, U.R.S.S.	13.0	26.0	8.7	17.4
DESIGNERS*	15 - 30		10 - 20	

* with solar energy and with or without lateral restraint.

For the reasons already explained by different authors[7,8], Rose's[1] values based on the Brown and Clarke[9] experimental results are without basic signification. The U.R.S.S. norms CN-76-59[2] are calculated from Royen's[10] ice rheological equation which has been found imprecise[7,11,12]. Furthermore, the simple fact that the thrust is a linear function of the ice thickness show that these values are not calculated from the temperature distribution in an ice cover. The values calculated from Monfore's experimental results give thrust of the same order of magnitude that the mean value used by designers. This do not prove that the problem is solved. Monfore did the same experimental test with many natural ice samples and the maximal stress obtained was varying between factors of 1 to 4.25 (5.6 - 24 kg./cm^2, 80 - 341 psi.). Taking small samples from natural ice sheets, there is good possibilities to get different ice sample structures, including monocrystals, with different optical orientations. The results published by Monfore[4] were obtained in an analysis without dimensions using, as the reference points, statistical mean values for many similar tests.

The dimensions of a concrete gravity dam are more or less influenced by the static ice thrust. Taking into account that the ice cracks and the snow on the ice sheet reduces drastically the thermal thrust, we assume, with the present knowledge, that 22 tons per linear meter (15 kips per linear foot) is a reasonable value for a thick ice sheet moderately restricted. For this thrust value, the increase of weight in function of the structure height is shown on Figure 2. For dams higher than 21 meters (70 feet), the increase in weight is less than 10%. Taking into account the overall factor of safety, the effect of the ice thrust becomes negligible. For smaller dams the increase in weight is significant and damages can be important if the static ice thrust is under-estimated. However, if the thrust value is over-estimated, this contributes to increase the cost of such dams.

PRESENT RESEARCH

A systematic research in order to obtain an acceptable solution should consider the different natural types of ice as[13]:

1- Columnar ice with preferred horizontal optical axis.

2- Columnar ice with preferred vertical optical axis.

3- Columnar ice with random optical axis.

4- Snow ice.

5- Congealed frazil ice.

The primary ice layer in a reservoir is generally formed of one of the types of columnar ice. During the winter, the water level fluctuations contributes to form snow ice layers on the primary ice layer. The research of the maximal ice thrust becomes, for a particular site, a function of the ice sheet formation, the local atmospheric conditions[5] and of the degree of ice sheet restraint.

Uniaxial Restriction

The experimental method adopted consist in submitting cylindrical ice samples to different constant strain rates and this for several constant temperatures. Examples of creep curves (primary creep) of snow ice obtained are shown on Figure 3. For these examples, the corresponding constant rate of temperature rise is 4.50°C./h. From these creep curves it is possible to determine the stress-time curves, for different initial temperatures, relative to thermal expansion of ice samples restraint in one direction. This is obtained by computing the time required to reach, from a given initial temperature, the creep curve temperatures. The curves being obtained are shown on Figure 3 (dashed lines). This technique has the great advantage to do experimental tests at constant temperatures.

Biaxial Restriction

The experimental set-up used to simulate lateral restraint of an ice sheet consist essentially in a rigid invar ring (3 mm. thick, 30 cm. diameter, 5 cm height) in which a circular thin plate of ice is introduced. The ice disk is first cut with a band saw at a diameter $\approx$ 2 - 3 mm smaller than the ring, cooled to the desire initial temperature and placed in the ring. The interstice is then filled with water untill the ice disk is well welded to the metal ring.

Invar has been selected because this alloy has a very low thermal expansion coefficient and interesting mechanical properties. The ring deformations are given by strain gages sticked along the exterior perimeter. The relative ice disk strain rate is obtained by making the difference between the relative deformation of free expanded ice and the relative deformation of the exterior ring perimeter. The radial stress exerted by the ice disk is calculated by the thin-walled cylinder formula.

Columnar ice with preferred horizontal c-axis

Figure 4 shows typical results obtained with columnar ice with preferred horizontal c-axis. The grain diameters are of the order of 0.75 cm. The ice disk temperature is raised in such a way to obtain a constant strain rate of the ice disk. At the beginning, the stress increases linearly and then it increases more slowly up to the maximal stress. When the ice temperature reach a value in the vicinity of -2°C., the stress begins to decrease at a large rate. At 0°C., the stress falls drastically. The parameters for this test are the following:

Initial ice temperature	-26.3°C
Ice strain rate	$3.48 \times 10^{-8} sec^{-1}$
Corresponding ice temperature rate	≈ 2.5°C./h.
Maximal stress	14.8 kg/cm^2

Biaxial test results for this type of ice are difficult to associate with those of creep tests done on small samples. Hypothesis relative to the deformation mechanism of columnar ice should be formulated and verified. In the case of biaxial restraint the forces acted essentially perpendicular to the basal plane and the problem is reduced to a test similar to one on a large monocrystal with many sub-boundaries, since the c-axis are more or less perpendicular to the growth direction. On the other hand, the uniaxial deformation mechanism is quite different because glides occurs along the boundaries.

Snow ice

Biaxial tests with snow ice give stress and strain curves quite different than those obtained with columnar ice. Results of a typical test are shown on Figure 5. The stress-time curve is nearly linear up to the maximal stress value. When the maximal stress is reached the ice strain rate increase at a higher value up to 0°C. With the same test conditions as with columnar ice, the change of the ice strain rate at the time when the maximal stress is reached has been observed for all of our tests with snow ice. Independently of all factors, the decrease of the stress after having reached his maximal value is 0.6 kg/cm^2 per °C rise. The parameters for this test are the following:

Initial air temperature	-28°C
Ice strain rates	
-28°C to -9.7°C	≈ $3.60 \times 10^{-8} sec^{-1}$
-9.7°C to 0°C	≈ $6.20 \times 10^{-8} sec^{-1}$
Corresponding ice temperature rates	
-28°C to 9.7°C	≈ 2.6°C/h.

$-9.7^{\circ}C$ to $0^{\circ}C$	$\approx 4.4^{\circ}C./h.$
Maximal stress ($-9.7^{\circ}C$)	$12.7\ kg/cm^2$

This later maximal stress compared with the corresponding one obtained from uniaxial tests is higher by a factor ≈ 2. Other biaxial tests are required to precise the majoration factor to convert uniaxial maximal stress to biaxial maximal stress.

CONCLUSION

The field of research of the thermal ice thrusts has been inactive for a long time. The experimental approach briefly explained in this paper, is a new step toward the solution of this problem which deal only with the early stage of primary ice creep. Recent progress in the determination of the rheological properties of types of ice with specific experimental tests will permit to obtain an acceptable solution of the thermal thrust exerted by ice sheets on hydraulic structures.

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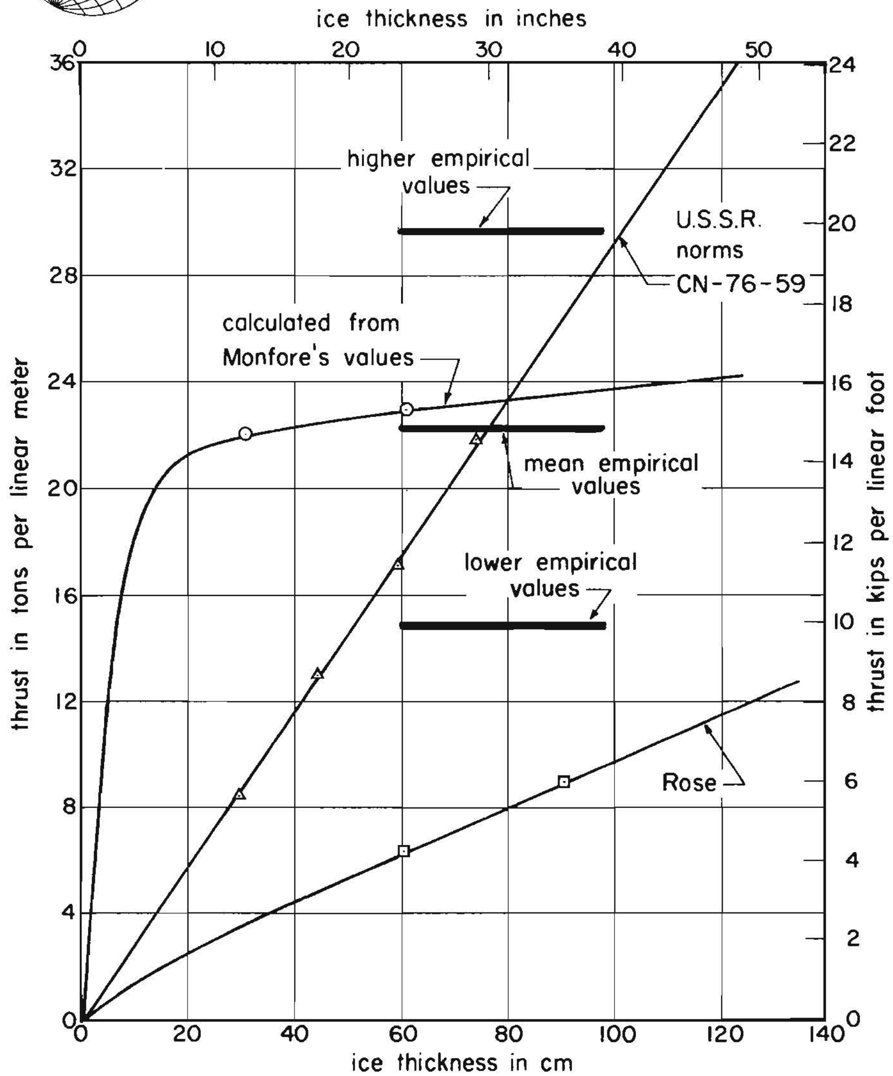

Fig. I. THEORITICAL AND PRACTICAL VALUES OF ICE THRUSTS ASSUMING ON THE TOP OF THE ICE COVER AN INITIAL TEMPERATURE OF -40°C (-40°F), A RATE OF TEMPERATURE RISE OF 2.8°C/M. (5.0°F/H). AND NO LATERAL RESTRAINT FOR THE THEORITICAL VALUES.

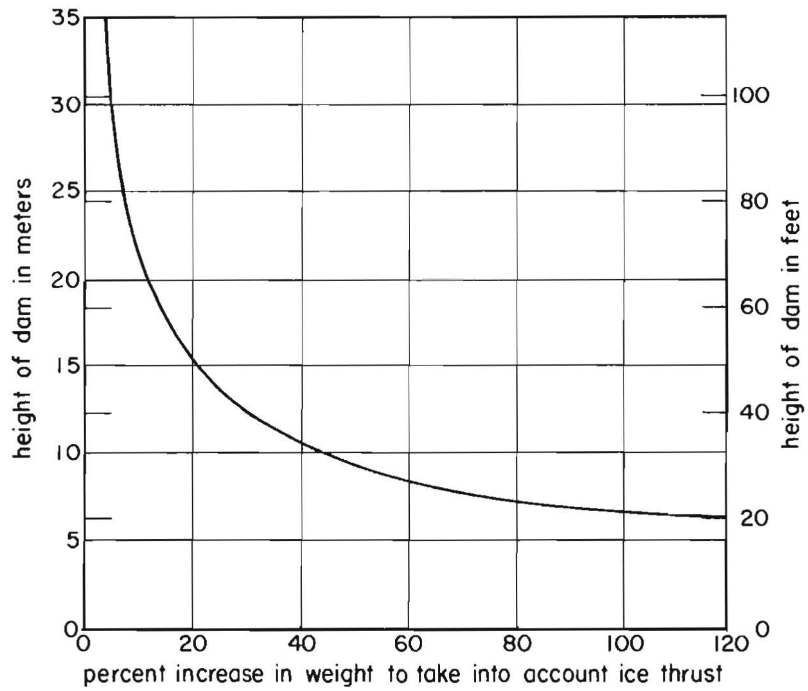

Fig. 2. EFFECT OF ICE THRUST ON GRAVITY CONCRETE DAMS:
1 - ASSUMING ROCK FOUNDATION.
2 - ASSUMING ICE THRUST EQUAL TO 22.4 tons/meter (15 kips/foot)

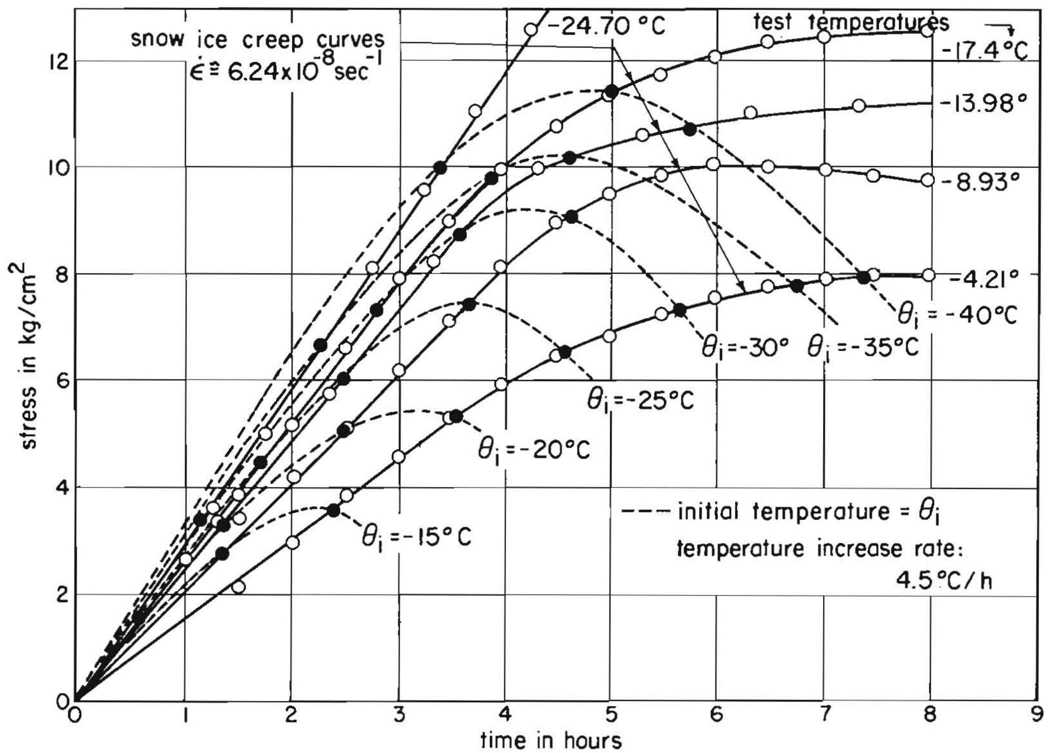

Fig. 3. STRESS–TIME CURVES RELATIVE TO THERMAL EXPANSION OF SNOW ICE RESTRAINT IN ONE DIRECTION DETERMINED FROM THE SNOW ICE CREEP CURVES.

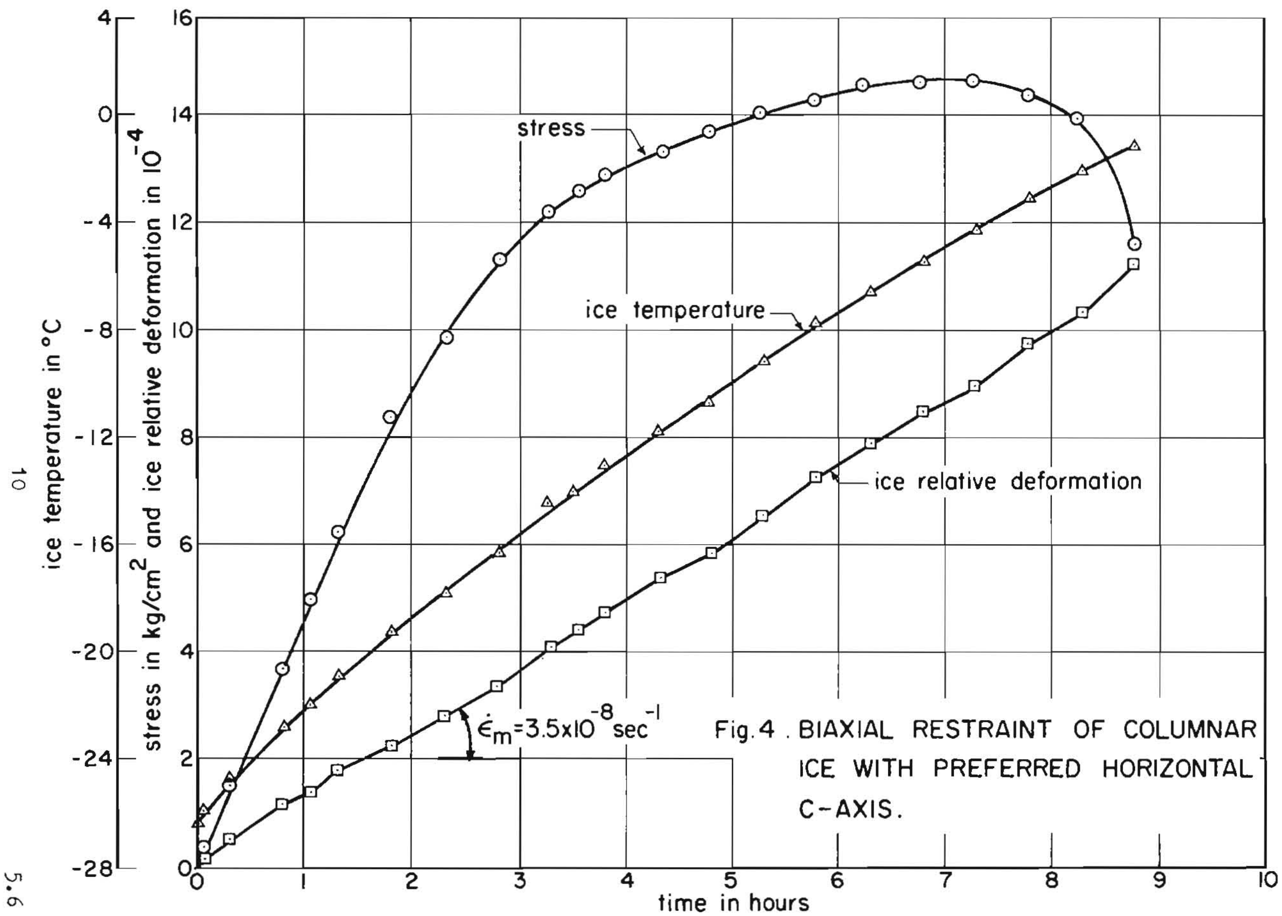

Fig. 4 . BIAXIAL RESTRAINT OF COLUMNAR ICE WITH PREFERRED HORIZONTAL C-AXIS.

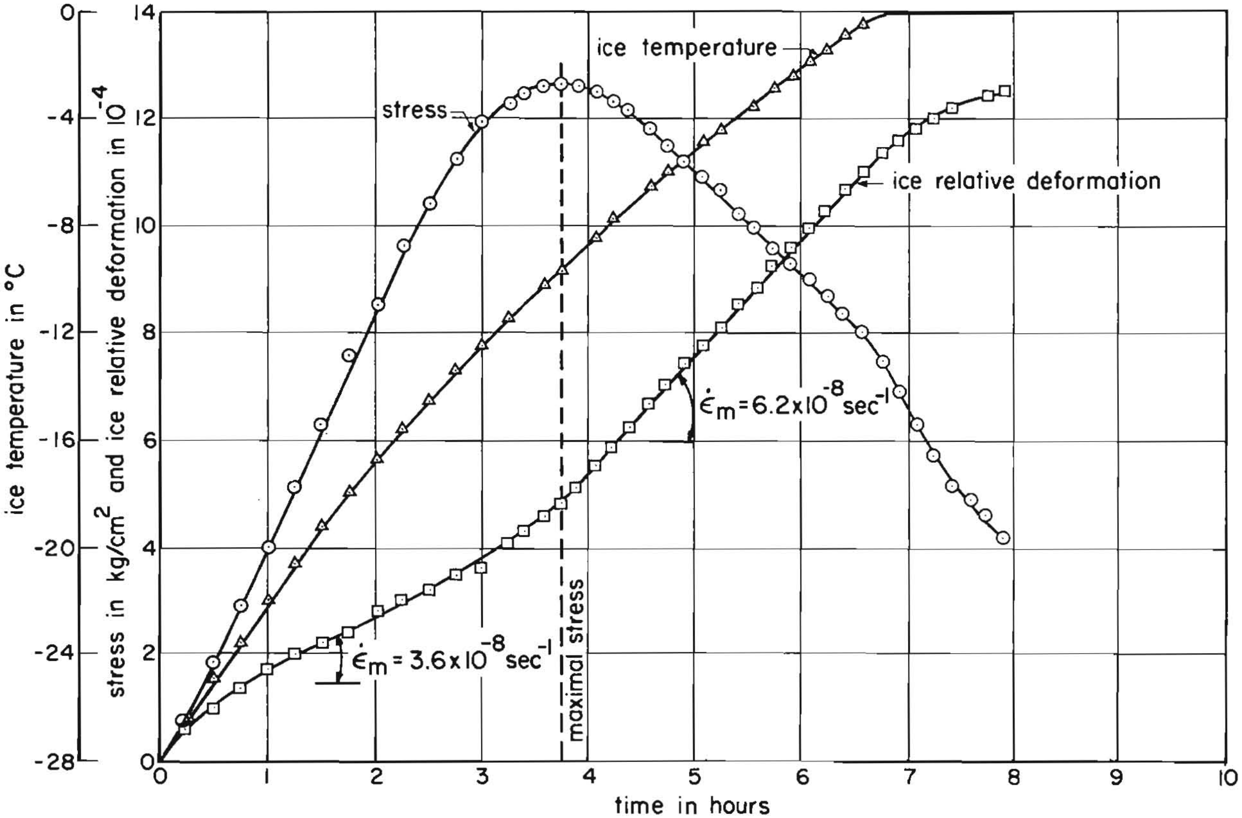

Fig. 5. BIAXIAL RESTRAINT OF SNOW ICE

ICE SYMPOSIUM 1970
REYKJAVIK

THE POTENTIAL OF THERMAL PILES IN ARCTIC MARINE STRUCTURES

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SYNOPSIS

The heat transfer characteristics of three types of thermal piles, the Long Pile, the Balch Pile and thermal convection loops, have been investigated. Empirical performance relationships have been developed for several piles and the effect of change of size has also been investigated. The results suggest that thermal piles could be used to build large steel-reinforced ice structures for use in the Arctic Ocean. The thermal piles would build the ice in place during the winter and would also act as the tensile reinforcement.

INTRODUCTION

The design of Arctic marine structures is the greatest challenge facing Arctic engineers today. The very large Prudhoe Bay oil field undoubtedly extends off the Alaskan Arctic coast and will require permanent marine structures for development. Off shore loading facilities to service icebreaking tankers and other types of Arctic marine structures will also be needed.

Several designs for such structures may be feasible. These include artificial islands in shallow water and massive steel and concrete structures in deeper water. Ice structures have been proposed. One type of Arctic marine structure to be considered is the steel-reinforced ice structure with the ice frozen in place by the use of thermal piles.

THERMAL PILES

Thermal piles are a general type of heat transfer device powered by environmental energy which pick up heat at their base and discharge it at their top when the top is colder than the base. Two types, the Long Pile[1] and the Balch Pile,[2] are named after their Alaskan inventors and were originally conceived for inducing or enhancing permafrost. It is an obvious step to consider using them for building offshore ice structures. The Institute of Arctic Environmental Engineering has been testing the overall heat transfer capacity of Long and Balch piles. While the program has not been comprehensive, it has been extensive enough to shed light on both behavior and heat transfer capacity.

The Long Pile, shown in Figure 1, is generally a vertical pipe partially filled with a refrigerant such as propane or carbon dioxide. When the top is cooler than the base, gaseous refrigerant will condense on the upper walls reducing the pressure and causing the liquid refrigerant to boil. The boiling process absorbs heat from the environment surrounding the base, the condensation process releases the heat to the atmosphere and the Long Pile transfers heat as long as the base is warmer then the top.

The Balch Pile, shown in Figure 2, is a vertical pipe filled with a fluid and equipped with an internal plumbing system for directing the fluid flow. If the bottom of the pile is warmer than the top, the warmer fluid will rise and the colder fluid settle. Heat exchange at the two ends drives the fluid circulation as long as the top is colder than the bottom and heat is continuously picked up at the base and exhausted at the top.

The thermal convection loops shown in Figure 3 are a variant of the Balch Pile and were developed by IAEE for high heat-transfer capacity. They are filled with a fluid and operate in the same manner as the Balch Pile. Convection loops can be fabricated of standard components and designed in a wide range of geometric configurations. The figure shows designs of increasing complexity and effectiveness. All three types of thermal pile are effective heat transfer devices.

Testing of the Long Pile was conducted by immersing the lower end of the pile in a bath and exposing the upper air-cooled end to low temperatures and winds. The bath was insulated and its specific heat known. Performance of the pile was obtained by monitoring the bath temperature which was allowed to fall. Figure 4 shows a typical pattern of air and bath temperatures with time during a test. When the rate of heat transfer was plotted against temperature potential, a linear relationship, such as that shown in Figure 5, was obtained. This showed that the heat transfer rate for a constant air velocity was a linear function of the temperature potential between the bath and the air. When tests were run with different air velocities, a family of relationships were obtained with the slope varying with the velocity of the wind. Heat transfer rate of the Long Pile can be expressed in the empirical form

$$q = (A + B\,V^{n})(T_b - T_a) \quad \text{Btu/hr} \qquad (1)$$

where q is the rate of heat transfer, V is wind velocity, T_b is bath temperature, T_a is air temperature, A and B are fitted coefficients and n is a fitted exponent. The right side of the equation consists of a wind velocity function and the temperature potential. The Long Pile which was tested has the empirical relationship

$$q = (6.79 + 12.136\,V^{.42})(T_b - T_a) \quad \text{Btu/hr} \qquad (2)$$

Heat transfer rates are shown graphically in Figure 6. The Balch Pile and convection loop have an empirical heat transfer rate of the form

$$q = -C + (A + B\,V^{n})(T_b - T_a) \quad \text{Btu/hr} \qquad (3)$$

This is identical to equation (1) except for C which appears to be the energy input required to drive the fluid flow. Heat transfer begins only when equation (3) becomes positive. We find for small to medium size devices operating within a reasonable temperature range that C can be considered a constant. A small Balch Pile four inches in diameter and fifteen feet long has a heat transfer rate of

$$q + -170 + (14.1 + 3.5\,V^{.7})(T_b - T_a) \quad \text{Btu/hr} \qquad (4)$$

The performance curves for this pile are shown in Figure 7.

EMPIRICAL SIZE RELATIONSHIPS

The size of the piles affects the heat transfer rate. For the Long Pile the heat transfer rate is probably about proportional to the cross sectional area and relatively independent of length. However, the heat transfer rates of Balch Piles and convection loops are sensitive to both cross sectional area and length. Limited experimental data suggests they have a relationship of the following form:

$$\frac{q_2}{q_1} = \frac{A_2}{A_1}\left(\frac{L_2}{L_1}\right)^{.5} \qquad (5)$$

where A is the cross sectional area of the pile and L is length. Length affects performance since the driving force is the differential weight of two columns of fluid, one warm and the other cool.

USE OF THERMAL PILES TO BUILD ICE STRUCTURES

Equations (4) and (5) provide a means of estimating the performance of large Balch Piles. Using the monthly mean temperatures and wind velocities at Barter Island, the four-inch Balch Pile would carry off 5.5 million Btu during the winter season of October-May if continuously immersed in sea water at a temperature of 28.6°F. If the diameter were doubled to eight inches and the length doubled to 30 feet, the pile would transfer 30 million Btu/season. If the diameter were again doubled to 16 inches and the length to 60 feet, it would transfer 170 million Btu/season. This is a capacity for freezing 21,000 cubic feet of ice. Large convection loops and Long Piles also can carry very large quantities of heat. The above estimates are the upper limit of performance and would not be realized if the piles were immersed in the Arctic Ocean since the water would already be at its freezing point. Extracting heat from the water would freeze ice on the piles which would serve as an insulator and reduce the overall heat transfer rate. There are limits to the quantity of ice which could be built by a particular pile. This limit for large piles has not been evaluated.

A massive offshore structure could be built in Arctic waters using thermal piles to freeze ice in place and using the piles and bracing system as tensile reinforcement. This type of steel-reinforced ice structure can be very large and strong and yet of relatively nominal cost and should be considered in the design of Arctic marine structures. Ice is the least expensive structural material in the Arctic and can be extremely competent. In fact, the Arctic marine design problem is that of resisting the sea ice. Since ice is strong in compression but weak in tension, ice structures require tensile reinforcement. Perutz[3] and Coble and Kingery[4] among others, report that small quantities of tensile reinforcement greatly improve its structural properties.

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DISCUSSION

Mr. G. Tsang. The proposed thermal piles certainly will work in a brine. However, as ice is formed around the pile, a coat of insulation is wrapped around the pile and will reduce the efficiency of the piles. For this reason, I doubt whether the thermal pile may be practically used for creating artificial ice islands since apparently the area refrigerated by the pile decreases greatly once ice is formed.

Answer: I have shown the general pattern of behavior of different types of thermal piles when the lower end of the pile is immersed in a bath and thus has available relatively large quantities of heat. In this case the heat transfer is limited only by the characteristics of the device and its efficiency in discharging heat to the atmosphere.

The growth of ice on the lower end of the pile would create a thermal barrier which would reduce the heat transfer rate and the rate of ice growth. However, this does not mean that thermal piles cannot be used to build the structures; it means that the amount of ice that could be built by a thermal pile has a practical limit. This limit has not been evaluated but under Arctic conditions the ice formed by the thermal pile can have a cross-sectional area two to three orders of magnitude greater than that of the material comprising the pile.

The design and construction of an ice structure in the Arctic Ocean using thermal piles would not be simple or inexpensive. The critical problem would be protecting the structure during the early winter before it could resist the pack itself. Once that critical period were passed, the structure would become increasingly competant over the years. The ice-thermopile structure can probably be built in relatively deep water in the Arctic Ocean at a fraction of the cost of conventional steel or reinforced concrete structures.

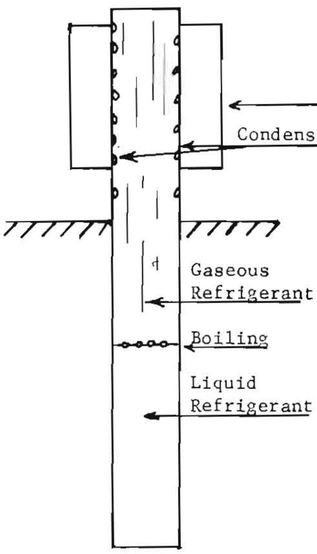

Fig. 1. Long Pile

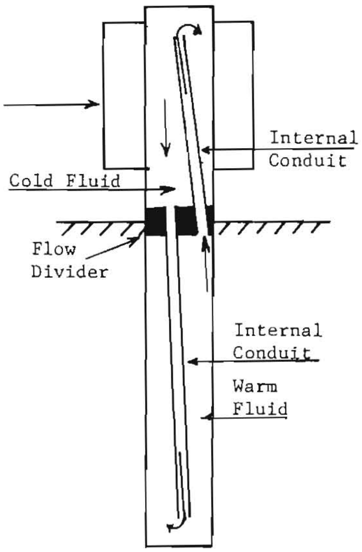

Fig. 2. Balch Pile

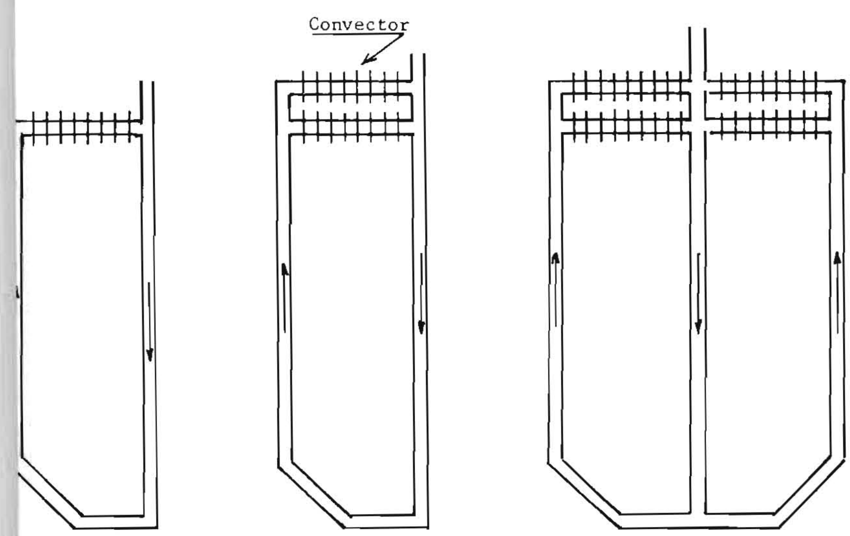

Fig. 3. Thermal Convection Loops

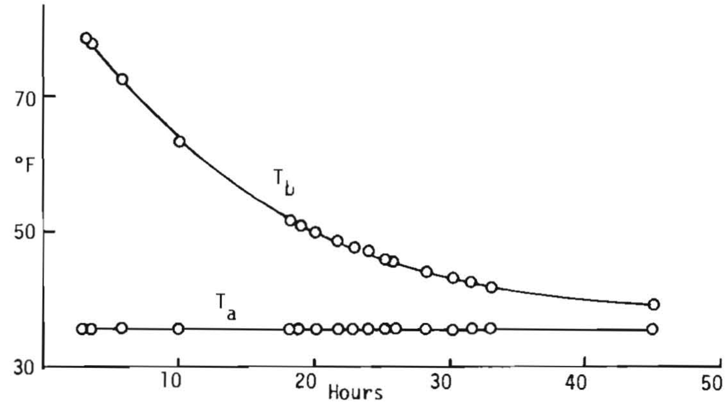

Fig. 4. Air and bath temperatures during a test run.

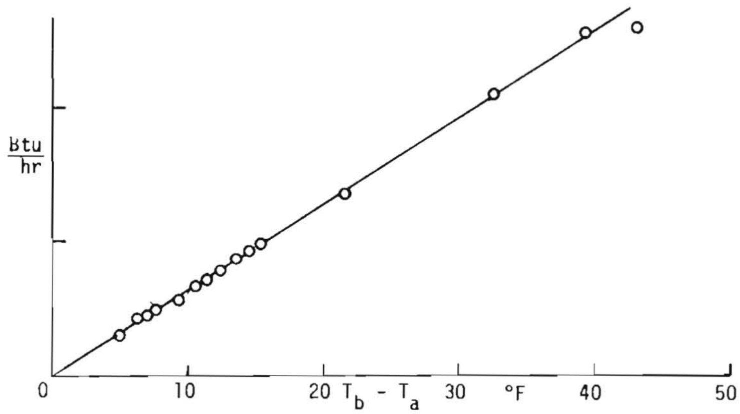

Fig. 5. Heat transfer rate versus temperature potential.

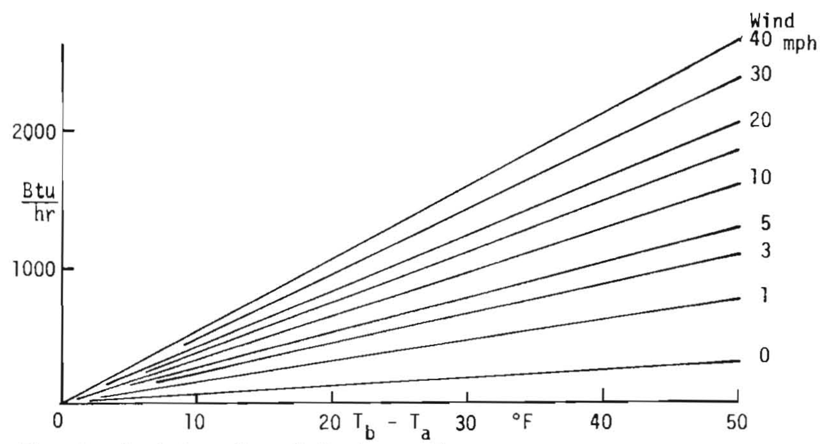

Fig. 6. Heat transfer of the Long pile.

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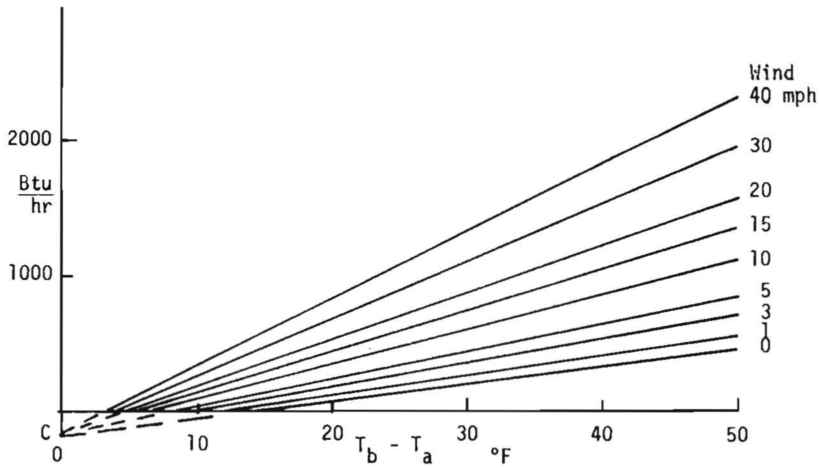

Fig. 7. Heat transfer of 4" Balch pile.

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OFF-SHORE MOORING STRUCTURE FOR THE ARCTIC

Bernard Michel, Dr.Eng. Université Laval Québec
Professor of Ice Mechanics Département de Génie Civil Canada

SYNOPSIS

This paper presents a new type of design for a harbor that will shelter ships in the Arctic Ocean. The ships will be protected against moving ice whatever will be the direction of the ice drift. The mooring structure is cylindrical in shape and is to be set off-shore in deep waters to receive one, two or more of the largest possible tankers. One main feature of this new structure is the fact that it clears ice on its side whatever be the direction of the drift, so that no ice accumulation could form around it.

RESUME

Cette communication fait état d'un nouveau type de port de mer qui pourra opérer dans l'Océan Arctique. Les bateaux seront protégés contre l'impact de la glace de dérive quelle que soit la direction de leur mouvement. La structure d'amarrage est cylindrique et située au large des côtes en eau profonde de façon a recevoir un, deux ou plusieurs pétroliers géants. Une caractéristique importante de cette structure est le fait qu'elle permet de disposer de la glace sur ses côtés, quelle que soit la direction de la dérive. Il est ainsi impossible d'accumuler des glaces le long de la structure.

INTRODUCTION

With the increase in the exploitation of mineral resources, particularly the tapping of oil reserves in the Arctic, there is a need for an appropriate marine terminal to move these resources out of ice covered waters.

Apart from providing simple mooring of ships, such installations are also expected to afford the usual services rendered by a harbor such as shelter against storms, whatever be their direction; loading up of ships, usually tankers with oil directly from a pipe or from a reservoir; refuelling and repairing of ships; providing sundry supplies etc...

Furthermore, in the Arctic, such a terminal should be specially designed to solve properly the problem of ice accumulation in front of, and eventually, all around the wharf, that would quickly render the wharf totally useless at least during the nine months of the year that the winter lasts in these regions.

It is the purpose of this paper to present a new type of ship mooring installation that solves more specifically the ice problems of the Arctic.

BASIC PRINCIPLE

The harbor is essentially a large cylinder set in an area of moving ice in the sea. It will deflect the ice and form a lee area free of moving ice. When there is no ice it will also reduce the storm waves amplitude to an acceptable small value in the lee.

The structure will operate off-shore in deep water, away from landfast ice. Because of possible variation in direction of ice movement and waves the ship will berth in the lee area in any position around the mooring head as shown on Figure 1. The bow of the ship will be secured to the head so that the ship will have its axis extending radially from the wharf.

THE WHARF

The wharf operates in a manner shown on Figure 2. This is a view in plan showing in A the top of the harbor in which all facilities are installed.

In stormy weather there are large sea waves attacking the harbor in the direction shown by arrow F. The amplitudes of the waves is reduced by diffraction to an acceptable value in the sheltered area E. Thus a ship can find a shelter in calm water in this area.

The high sea harbor operates in a similar manner under the action of a moving ice pack as found off-shore in the Arctic. If the ice pack moves in a direction given by arrow F, there is a sheltered area free of moving ice opposite the exposed side of the harbor. Morever because of its cylindrical form there is no possibility of an ice pressure ridge being formed and grounded along the harbor face, that would eventually prevent berthing of the ships.

Figure 3 gives a cross-sectional view of such a wharf. It is made of concrete and comprises a pillar base topped by a frusto-conical capital over which a circular mooring head is built. As illustrated the pillar base is completely submerged amd may be formed by prefabricated caissons which, when assembled, form an annular ring sunk at the bottom of the sea. The capital is built on top and has an inclined outer surface coming out of the water whose ice breaking form is much more efficient not only to break but also to clear the ice underneath on both sides. The upper mooring head may be hollow and formed with a floor and ceiling in order that all operations be carried on in a warm interior. This inner space may be partitioned to provide storing and housing areas. The space beneath the floor is filled with earth or rockfill so it makes a monolithic structure resisting the ice pressures. If the mooring installation serves for oil tankers a reservoir may be provided. The roof may also be used for aircraft landing.

THE SHIP MOORING SYSTEM

The ship has to be held against the mooring head for loading or unloading. It also has to resist wind action that may produce considerable lateral forces when it is blowing perpendicular to its axis. Finally if the direction of ice or wave movement changes, the ship has to be relocated around the wharf in the changing lee area.

To provide for the largest resisting moment with the smallest mooring cables, first mooring means are located at the bottom of the sea in the form of dolphins distributed on a circle circumscribing the mooring head at a distance such as to allow mooring of the stern of the ship on both sides. The dolphins, of the cleat type, are unaffected by moving ice conditions. They provide for holding the ship for any position of the lee area around the wharf. The mooring cables are wound around the cleats by different means, but usually with the help of a tugboat.

To obtain also a high resisting moment at the bow as well as a high resisting lateral force, a special foredeck is required on the ship in the form of an arcuate front as large as possible whose ends are tied to dolphins set on the periphery of the mooring head as shown on Figure 2. The arcuate front has a curvature substantially the same as that of the distance between the bow and the center of gravity of the ship in order to induce stability against small lateral movements. A high resisting fender is required for larger ships. For Arctic conditions it is better installed on the special foredeck of the ship itself instead of around the mooring head.

CONCLUSIONS

This new type of ship mooring installation can operate twelve months a year under the most severe conditions of ice pack movement or storm waves in Arctic waters. It has to be situated off zones of landfast ice in deeper waters and off the course of large ice islands or icebergs. Large ice masses might however be artificially deviated from their course, if necessary.

The harbor could be designed to berth, in deep water, one or more ships of any sizes, even the biggest. With its own oil reservoir it may fill very quickly the largest oil tankers.

No other work of protection is required, like any classical breakwater, and no dredging need be done before construction. The whole structure could be built by sections in existing ship yards and then brought and sunk in place. The overall size is very much reduced because of the radial position of the ship relative to the central structures and thus it could well be one of the most economic solution to the design of a harbor in Arctic waters.

REFERENCES

"Off-shore ship mooring installation" U.S. Patent 807, 236, pending; Canada Patent 086, 066, pending.

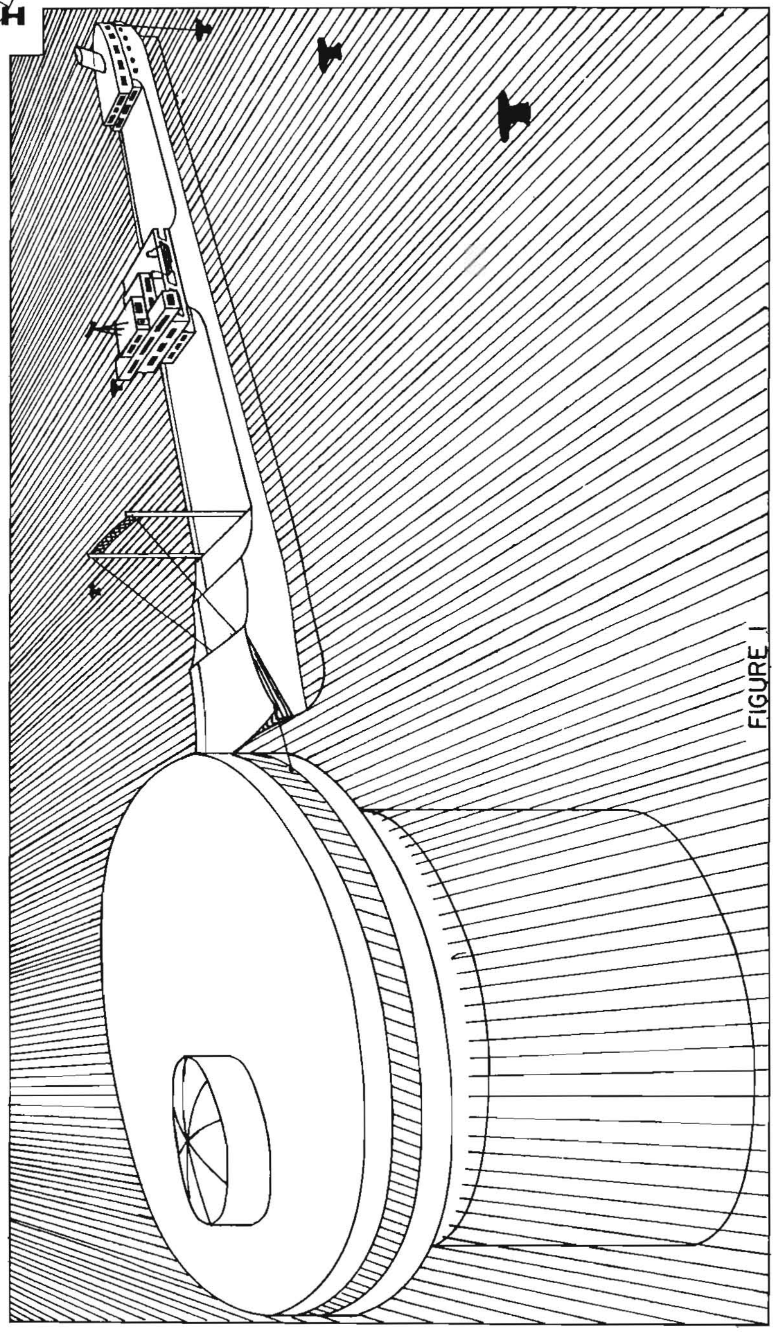


FIGURE 1

VIEW FROM THE TOP — ARCTIC MOORING STRUCTURE

PRINCIPLE OF OPERATION

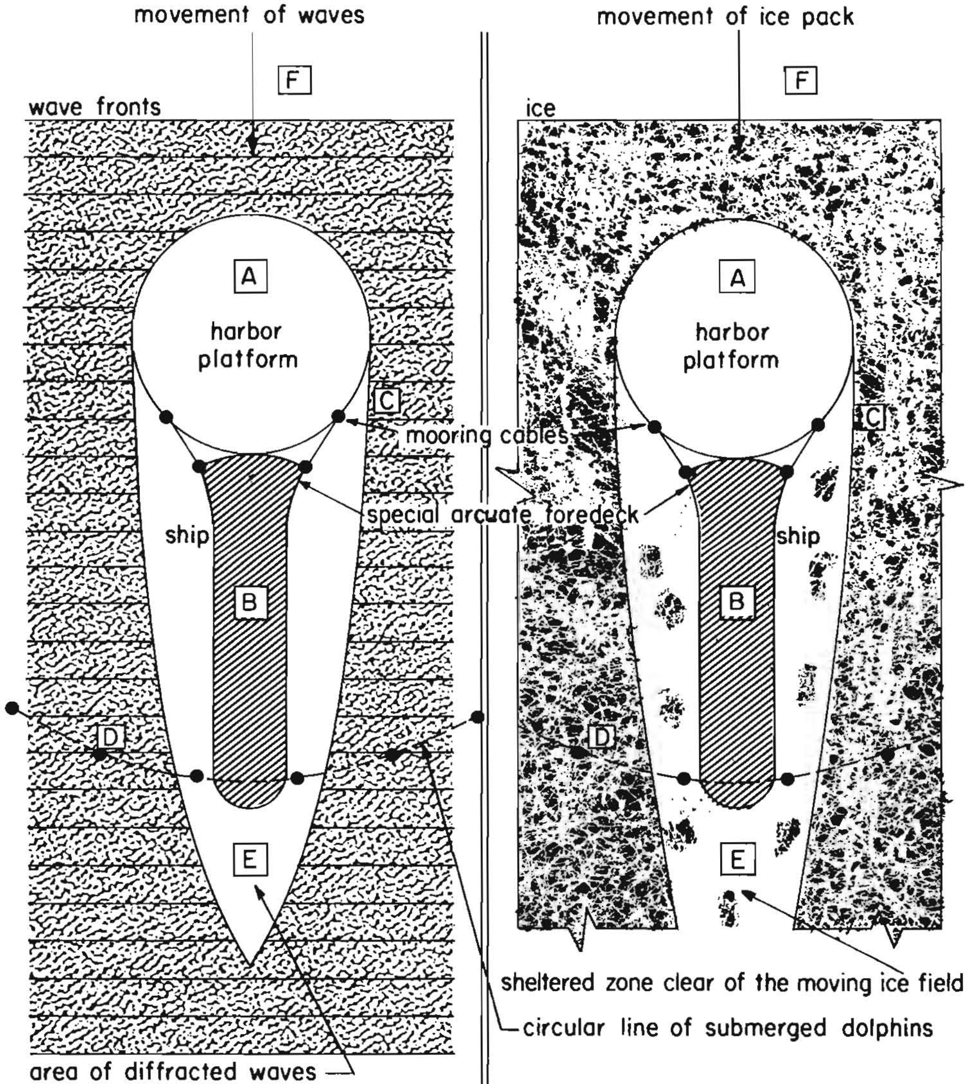


FIGURE 2

SECTION THROUGH ARCTIC MOORING STRUCTURE

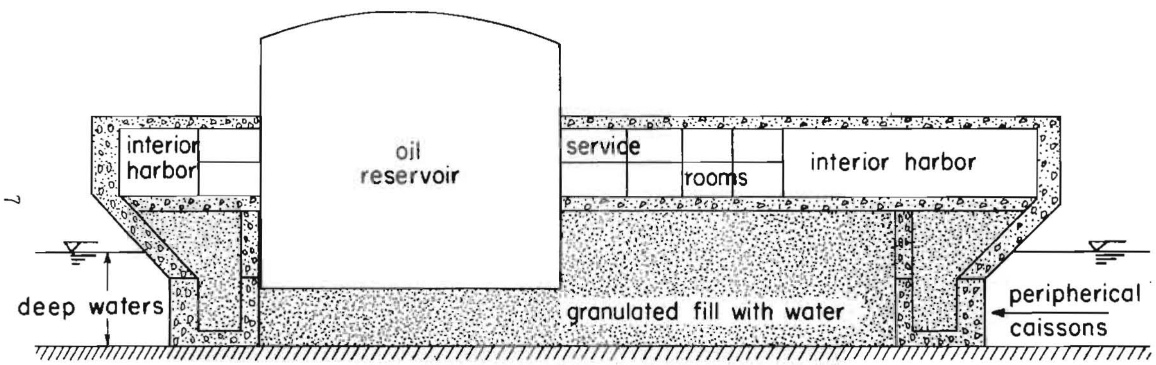


FIGURE 3

DISCUSSION

S. Lazier: Queen's University, Canada.

With regard to the possible "Lee" which may develop behind Dr. Michel's mooring structure, I wish to show two aerial photographs taken in 1970 which illustrate this phenomenon. At the time of break-up in the harbour at Kingston, Canada, an ice sheet of some 20 miles long was pushed against Snake Island by a wind of about 20 m/sec. The dimension of Snake Island are about $\frac{1}{4}$ km wide normal to the ice push, $4\frac{1}{2}$ km in length. The photographs show clearly the "Lee" which developed behind Snake Island. On the windward side the ice piled up to a height of about 8 meters.

DISCUSSION

A. Kovacs: CRREL, U.S.A.

Dr. Evens questioned whether the lee side of the sea ice harbour platform proposed by Dr. Michel would remain open or free of ice as the pack moved passed the platform as represented by Figure 2. Dr. Evens was of the opinion that a free ice area would not occur. My observation of ice accumulations in the lee if grounded ice islands in the Beaufort Sea is that the ice is highly hummocked and as a result is thicker than the surrounding "plate" ice. Indeed I would only expect an ice free area to exist when the ice canopy is highly rarefied.

DISCUSSION

R. Y. Edwards, Jr: Arctec Inc., U.S.A.

Dr. Michel's paper on an off-shore structure is extremely informative. I have a few questions. I must stress that they are exactly that questions to which I solicit answers from experts in the behaviour of sea ice.

On icebreakers upon which I have served and which I have tested, I have observed very frequently the ice shadow or track behind the ship close very rapidly. Is it possible for a stress field to develop perpendicular to the direction of motion of the ice sheet. (The tremendous inertia of ice floes can keep them moving in a particular direction despite a shift in wind to another direction). This field would cause closure of the "ice shadow" behind the structure.

My second question concerns his suggestion for a multiple mooring system. I am sure that a simple point moor at the bow will allow the ship freedom to fetch up against the side of the "ice shadow" if wind is at an angle to the ice shadow, with no particular harm to ship or structure.

DISCUSSION

T. M. Dick: British Hovercraft Corp., England.

Dr. Michel has proposed an interesting solution for the protection of moored vessels from ice movement.

The protection from wave action is not so certain nor likely to be as extensive as shown in Figure 2. If the dimensions of the wharf and ship as illustrated are representative, then waves of length of half the ship length or greater will tend to treat the wharf as being hydrodynamically invisible and will diffract around into the lee of the structure making a rather confused wave system.

The sheltering effect is likely to be minimal unless the wharf is made much greater in size than the illustrated dimensions. Consequently the ship would probably pitch and roll considerably during storms in the absence of ice. Vertical movement, if sufficiently large, could interfere with the loading or unloading operations and this could be an important factor in the short Arctic navigation season.

Owing to the diffraction it is clear that the size of the wharf will depend on the fetch generating the seas. Both ice forces and wave action affecting the structure will depend upon its location. In the design the wharf will need to satisfy both the ice dynamics and the open season wave dynamics.

DISCUSSION

G. Tsang: University of Guelph, Canada.

As the wind in the Arctic is strong it may blow in the cross direction to the sea current (i.e. 90°). With the ship being moored at two ends in the direction of ocean current, the wind may blow broad-side onto the ship and produce a bending moment longitudinally to the ship. Since the wind in the Arctic can be as high as 100 m.p.h., the wind pressure is proportional to the square of the wind, and the bending moment is proportional to the square to the length of the span, the bending moment produced can be very high. This required the strength of the ship be greatly increased. (At present, a ship's strength is designed mainly for the bending moment of the ribs). Then, an economical problem of modifying existing ships and strengthening future ships is posed here.

Also, the orientation of a ship is governed by the combined effect of wind and water drags. So the ship floats up during unloading, (or goes down during loading). The wind drag and water drag change continuously and this cause a continuous change of the resulting force. In order to keep the orientation of the ship, two moorings at the end of the ship are required instead of one.

CLOSURE

B. Michel

I would like to thank Professor Lazier for his most convincing slides showing the lee behind Snake Island in Lake Ontario. On the other hand Mr. Kovacs says that an ice free area does not exist except when there is very little ice and shows this with a picture of a lee behind a grounded island in the Beaufort Sea. Mr. Edwards says that the lee may close very rapidly behind such a structure because of pressure existing in a field which is perpendicular to the motion of the ice sheet. So I think the question is fully opened to discussion.

In my mind there is absolutely no doubt that a lee exist in all cases for the berthing of a ship. And, of course, the second part of this sentence is important because a ship can berth in a lee full of broken up, small unconsolidated ice pieces, slowly moving away. The picture of Mr. Kovacs makes me think of actual icebreaker operation. The lee exist behind the artificial island but it is full of pieces in the same manner as a track made by an icebreaker when it is full of broken up pieces. Of course merchant vessels can follow in these tracks. The amount of small ice pieces in the lee of the structure would depend very much on its forms and model studies have to be carried to find out for each case. It is obvious that a form which would act as a thin surface island connected by one or a few points to be ground would leave most of the ice pass underneath and appear in broken pieces in the track. We believe that the icebreaking form we are suggesting for the mooring structure would clear most of its ice on the side and little would appear in the track. Mr. Edwards is quite right and a lee may close by pressure behind a ship but not that close to be applicable to the mooring structure we are considering. Experience on icebreakers in Arctic conditions seems to indicate that it may close as near as half a mile from the icebreaker. This is indeed many many times the width of the ship. We need only about two times the width of the mooring structure to berth a ship behind it, in this case.

The lee formed against wave action is not an important factor in this design. Mr. Dick is afraid that it will not exist and that this will hamper loading and unloading operations. In fact in the Arctic Ocean the open water fetch is always small at practically all locations and wave heights and lengths are not great. Wave lengths are much smaller than the dimension of the structure required for other purpose, so that there is important diffraction and wave reduction in the lee. But, even if there would not be any diffraction, modern large size tankers do easily load and unload now in heavy open seas elsewhere in the world as long as their axis is perpendicular to the movement of the storm waves. This is a principle of operation in this structure.

There seems to be some misunderstanding as to the arrangement of mooring lines for this structure. In case when there is no lateral wind relative to the direction of ice movement it is clear that only mooring of the bow will be required as suggested by Mr. Edwards. But in case of a strong lateral wind, mooring at both sides of the stern on the bottom dolphins would be necessary for reasons discussed by Professor Tsang and for other reasons. Big tankers berthing along open sea wharfs are already designed against bending to resist lateral wind action.

ICE SYMPOSIUM 1970
REYKJAVIK

ICE PRESSURE ON BRIDGE PIERS IN ALBERTA, CANADA

Research Council of Alberta Contribution No. 465

C. R. Neill   Research Council of Alberta   Edmonton, Canada

SYNOPSIS

Devices designed to measure total ice force during river break-up have been built into two bridge piers. Measurements over a four-year period have yielded unit pressures from moving ice-sheets in the range of 100 to 160 lb/in^2 (7 to 11.3 kg/cm^2), for ordinary to moderately severe conditions.

Analyses have been carried out of the probable ultimate strengths of several old piers that have successfully withstood ice for many years. Estimated unit pressures for structural failure are in the range of 150 to 250 lb/in^2 (10.5 to 17.5 kg/cm^2).

It is concluded that the ice pressure of 400 lb/in^2 (28 kg/cm^2) specified in Canadian codes may be unnecessarily conservative for bridges in inland rivers where the most severe conditions are associated with spring break-up.

Recommendations are offered for the design of force-measuring devices.

INTRODUCTION

Investigation of forces developed by river ice striking bridge piers during spring break-up have been in progress in Alberta for several years. The approach taken has been essentially empirical, involving field measurement of actual ice forces at special sites, plus estimation of the maximum forces likely to have been withstood in the past by certain older bridges.

Some details of the investigations have been previously reported by Sanden and Neill (1). Observations of associated river break-up processes have been reported by Nuttall (2).

CURRENT DESIGN PRACTICE IN CANADA

Bridge design codes used in Canada imply that longitudinal ice forces on piers are to be calculated by taking an average ice pressure of 400 lb/in^2 (28 kg/cm^2), and multiplying by the width of the pier and the expected maximum thickness of ice. This yields very large overturning moments in many cases. The codes do not specify ice thicknesses nor orientation of ice forces in relation to the pier axis. It is possible to apply the full force longitudinally but ignore transverse components, producing elongated piers with relatively low transverse strengths. Designers have been known to use ice thicknesses less than actually expected, on the argument that the specified pressure is unlikely to occur simultaneously over the full thickness. If transverse components are neglected there are obvious advantages in reducing pier widths, because longitudinal forces are thereby reduced in proportion.

It is important to note that the specified unit pressure need not be regarded as a real ice crushing strength, but rather as an empirical value which has been found to yield safe designs. Therefore the fact that crushing strengths considerably in excess of 400 lb/in^2 (28 kg/cm^2) are obtainable from small ice specimens is not necessarily relevant in considering whether the specified pressure is excessive. Some scientists have suggested that the specified pressure ought to be larger for small piers than for large ones. The strong incentive to slender piers offered by the present codes may therefore be unjustified.

FIELD MEASUREMENTS OF ICE FORCES

Special piers equipped to measure ice forces have been installed on two northward-flowing rivers in central Alberta, the Athabasca and the Pembina. Both rivers are normally covered from November to April with ice from 1.5 to 3 ft (0.5 to 1 m.) thick, and are liable to heavy ice-runs. The Athabasca River at Hondo is approximately 850 ft (300 m.) wide. The Pembina River at Pembridge is approximately 250 ft (75 m.) wide.

Fig. 1 shows details of the measuring system used at the Hondo bridge, where measurements of ice forces were made in 1967, 1968, 1969 and 1970. Fig. 2 shows details of the system at Pembridge, where measurements were made in 1969 and 1970. Relatively severe break-up and ice-run conditions occurred at both sites in 1969.

Systems used at both sites are designed to record the total force sustained by the pier, also its time-wise fluctuations. Originally the Hondo site had a strain-gauge load cell to measure top reactions, whereas the Pembridge site had flexural strain-gauges on the load-measuring pipe, but both are now arranged for measurement of top reactions. Provision is made for synchronized filming of ice action.

Table 1 summarizes maximum ice forces and corresponding average unit pressures as inferred from measurements over the 4-year period. Fig. 3 shows a typical length of strain-gauge output, illustrating characteristic oscillations of the ice force under varying modes of breaking and crushing while a more or less continuous ice sheet was being cut through by the pier. Velocities of incision have been generally in the range of 1 to 5 ft/sec (0.3 to 1.5 m/sec). So far it has not been possible to measure ice strength and quality immediately before break-up. Table 3 (p.5) shows some data on associated meteorological and river flow conditions.

TABLE 1 - ICE FORCES, THICKNESSES AND PRESSURES INFERRED FROM FIELD MEASUREMENTS

Site	Pier width		Year	Average ice thickness		Maximum force		Equivalent unit pressure	
	ft.	m.		ft.	m.	lb.	kg.	lb/in^2	kg/cm^2
Hondo	7.5	2.3	1967	1.25	0.38	140,000	63,500	104	7.3
			1968	Slow break-up : negligible forces					
			1969	2.5	0.76	370,000	168,000	137	9.6
			1970	1.5	0.46	200,000	90,700	124	8.7
Pembridge	2.8	0.85	1969	2.0	0.61	130,000	59,000	159	11.2
			1970	Slow break-up : negligible forces					

STRENGTH ANALYSIS OF OLD PIERS

An analysis of the ultimate strength of piers in five bridges that had withstood ice forces for many years yielded estimates as shown in Table 2 for the upper limit of average ice pressure probably sustained. These figures are based on ice 3 feet (0.9 m.) thick, a reasonable maximum figure for the sites in question. The estimated upper limits range from 120 to 250 lb/in^2 (8.4 to 17.6 kg/cm^2), all considerably below the value specified by the codes.

TABLE 2 - RESULTS OF ULTIMATE STRENGTH ANALYSIS OF FIVE OLD PIERS STILL STANDING

River name	Location name	Date of pier construction	Estimated ice pressure that would cause failure (ice 3 ft. thick) lb/in^2	kg/cm^2
Red Deer	Content	1906	120	8.4
Red Deer	Bindloss	1922	240	16.9
Bow	near Calgary	1921	150	10.5
Ghost	at the mouth	1929	250	17.6
Little Smoky	near Triangle	1950	190	13.4

CONCLUSIONS

The field measurements and analyses reported herein tend to suggest that for spring break-up conditions in Western Canada the specified code pressure of 400 lb/in^2 (28 kg/cm^2) may be unnecessarily conservative. This conclusion is not necessarily valid for rivers where hard cold ice may be broken up by winter floods or icebreakers.

The following suggestions are offered to those planning comparable field measurements on piers or similar structures:

- Aim to determine total ice force on structure.
- Make device mechanically simple, stiff, and rugged.
- Design mechanical parts for at least 300 lb/in^2 ice pressure, and allow for higher pressures on small areas.
- Measure reactions rather than bending or torsional stresses.
- Incorporate spare sensors, circuits, and transducers.
- Investigate dynamic behaviour of system under load frequencies in range of 1/2 to 10 cycles per second.
- Incorporate foolproof mechanical device for indicating maximum force, in case electrical sensing system fails.
- Provide for continuous analogue recording in order to register force vibrations.
- Provide for automatic start-up of force recording in case of unexpected ice break-up.

REFERENCES

(1) Sanden, E.J. and Neill, C.R. "Determination of actual forces on bridge piers due to moving ice". Proceedings of 1968 Annual Convention, Canadian Good Roads Association, Ottawa, p. 405-420.

(2) Nuttall, J.B. "Observations on break-up of river ice in north central Alberta". Canadian Geotechnical Journal, vol. 7, no. 4, November 1970.

ACKNOWLEDGMENTS

The results reported were obtained in the course of a research project initiated by E.J. Sanden, Chief Bridge Engineer, through a co-operative research program involving the provincial Department of Highways and Transport, the Research Council of Alberta, and the University of Alberta. Many people participated in the design, instrumentation, operation and interpretation of the systems. Special thanks are due to H. Schultz and Maria Schouten for field observations.

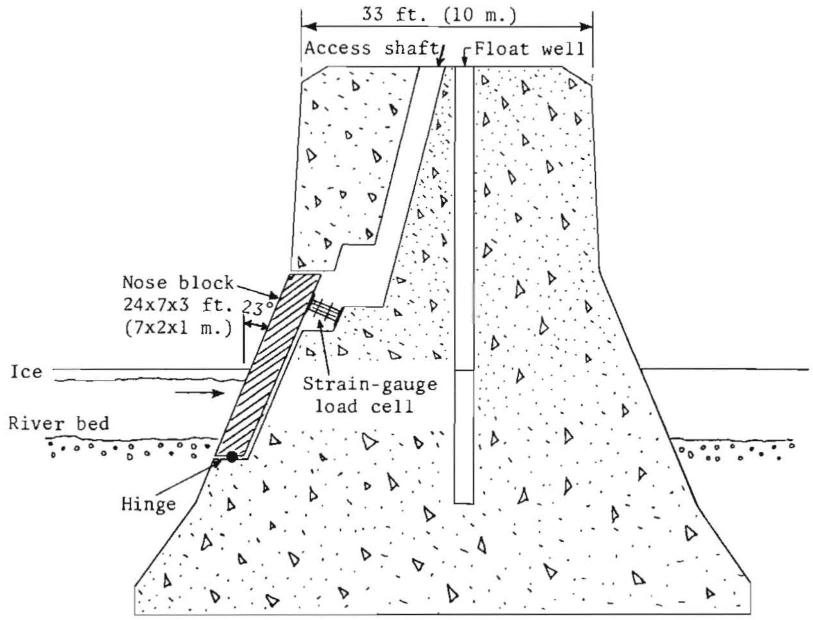
The paper as pre-printed has been revised to suit the constructive criticisms of reviewers and discussers.

TABLE 3 - METEOROLOGICAL AND RIVER FLOW CONDITIONS ASSOCIATED WITH ICE PRESSURE MEASUREMENTS OF TABLE 1

Site	Year	Date of break-up	Meteorological conditions over preceding 7 days				River flow conditions	
			Average daily maximum temp. °C	Average daily minimum temp. °C	Average daily hrs. of bright sun	Total precipitation mm	Maximum daily discharge m^3/s	Max. rise in stage during break-up period m
Hondo	1967	29 April	+10 (1)	-7 (1)	12 (3)	- (1)	730 (5)	1.5
	1968	21 April	+10 (1)	-6 (1)	9 (3)	16 (1)	190 (5)	-
	1969	14 April	+17 (1)	-5 (1)	9 (2)	1 (1)	700 (5)	1.5
	1970	16 April	+10 (1)	-5 (1)	9 (2)	- (1)	590 (5)	1.2
Pembridge	1969	11 April	+10 (4)	-5 (4)	7 (3)	- (4)	240 (6)	0.9
	1970	12 April	+ 8 (4)	-3 (4)	8 (3)	2 (4)	~30	-

(1) Data from Smith meteorological station.
(2) Data from Wagner meteorological station.
(3) Data from Edmonton meteorological station.
(4) Data from Sangudo meteorological station.
(5) Data from Athabasca hydrometric station.
(6) Data from Jarvie hydrometric station.

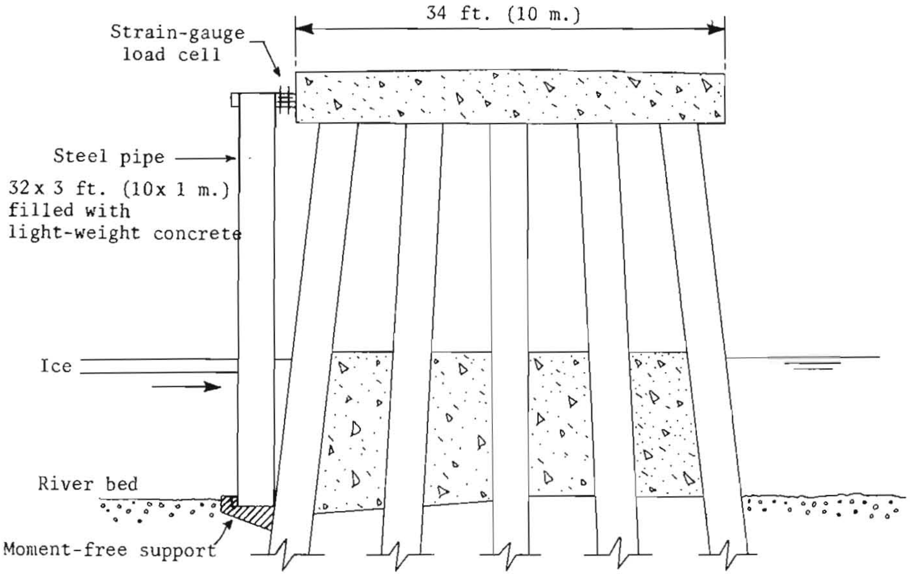
View of bridge from right bank : stripes on pier nose are 3 ft. wide



Longitudinal section of pier showing mechanical arrangement

Fig. 1 - Special pier for measuring ice forces, Athabasca River at Hondo.

View of bridge from right bank : stripes on pipe are 0.5 ft. wide



Side view of pier showing mechanical arrangement

Fig. 2 - Addition of pipe to pier for measuring ice forces, Pembina River at Pembridge.

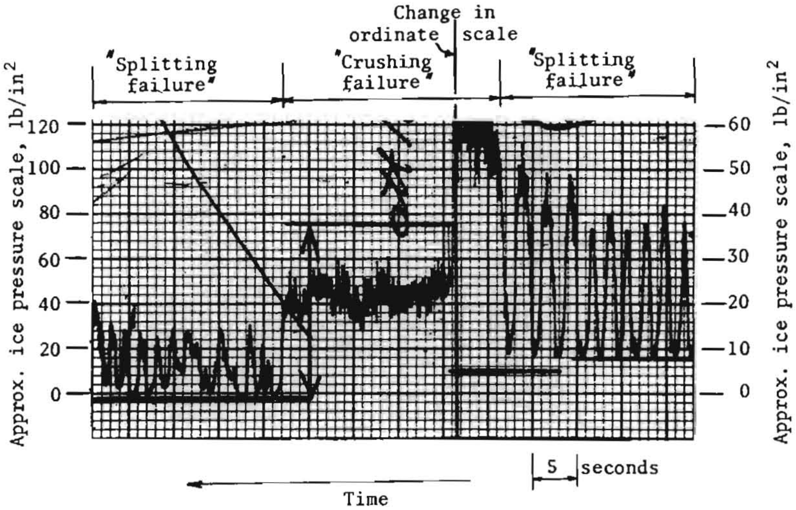


Fig. 3 - Sample of strain-gauge recorder chart showing nature of force oscillations on pier due to moving ice. (From Hondo pier, 1967)

"Crushing failure" : direct crushing of ice over full area of contact around pier nose.

"Splitting failure" : longitudinal splitting of ice-sheet ahead of nose and/or bending failure after partial lifting.

Note: pressure scales are approximate only, since ice thickness was not accurately known at this point.

ICE SYMPOSIUM 1970
REYKJAVIK

DYNAMIC ICE PRESSURE AND DEFORMATION OF STRUCTURES

LA PRESSION DYNAMIQUE DE GLACE ET DE DEFORMATIONS DES OUVRAGES HYDRAULIQUES

K.N. Korzhavin, Prof.,D.Sc.(Eng.) The Institute of Railroad Transportation Engineers Novosibirsk U.S.S.R.

Synopsis

Considered is a calculation scheme for estimating the forces of interaction occuring under dynamic ice loads. Design formulae are presented for determining the magnitude of dynamic ice pressure on vertical piers with due regard for the coefficient of elastic response.

Résumé

Dans le rapport on analyse le schéma de calcul pour définir les forces d'interaction surgissant sous les charges dynamiques. Les formules de calcul permettant de déterminer les valeurs de la pression dynamique de glace sur les appuis verticaux, compte tenu du coefficient de déformabilité, sont données.

Designing and maintenance of bridges and hydraulic structures in the greater part of the territory of the USSR must be carried out with due regard for the interaction of the forces developing under dynamic ice loads. This fact is of special importance for large Siberian rivers, where the breakup is sometimes accompanied by the formation of ice jams, sharp changes in water level and by the movement of large floes of considerable thickness and strength.

In the Soviet Union these problems were previously considered by some specialists (Kuznetsov, Zileev and Korzhavin[1]). This communication offers a fuller solution of the problem.

The following method of calculation is recommended.

A floe of a known area Ω, thickness h, and mass M, moving at a velocity v possesses a kinetic energy reserve which is spent on:

a) work in breaking the edge of the floe in the zone of contact;

b) elastic deformation of piers and floes;

c) the rotation of the floe about the contact point when the impact is eccentric.

It is easy to prove that the elastic deformation of the floe cannot have any appreciable influence on the value of the ice pressure and that practically a central impact of the floe on the pier is most always the case.

In this instance all the kinetic energy of the floe will be spent only on crushing the edge of the floe (Ti) and on the elastic deformation of the bridge piers (Ts) according to the equation

$$\frac{Mv^2}{2} = Ti + Ts \qquad (1)$$

Thus, the more flexible the bridge pier, the less will be the force transmitted to it (P). Assuming that the coefficient of elastic response (a) is the specific deformation of the pier due to a force of 1 ton (applied at the level of ice pressure), the total deformation may be expressed as

$$\lambda = Pa$$

and the work of the deformation as $Ts = \frac{P\lambda}{2} = \frac{P^2a}{2}$

In this connection equation (1) will be

$$\frac{Mv^2}{2} = \int_0^{X_0} Pdx + \frac{P^2a}{2} \qquad (2)$$

where X_o is the maximum depth of intrusion of the pier in the floe at the moment when it stops. Using equation (2), it is possible to find the mathematical relationship for determining the value of dynamic ice pressure on the pier (P), taking into consideration the coefficient of elastic compliance (a), the limit of compressive strength of ice shapes (R_{β}), and dimensions of piers, and also the dimensions and velocity of the floe.

The above analysis justifies the recommendation of the following method of calculation:

1. <u>For vertical bridge pier with triangular starling</u>

(2α - is angle of shape and m is the pier shape coefficient[2])

$$P = 0{,}30\ vh \sqrt{\frac{\Omega}{ah + \dfrac{1}{5R_{\beta}\, m\, tg\alpha}}} \tag{3}$$

2. <u>For vertical bridge pier with half-round starling</u>

As can be seen from the analysis of the problem[1], it is possible to use equation (2) assuming $\alpha = 70^o$, m = 0.90, then

$$P = 0{,}30\ vh \sqrt{\frac{\Omega}{ah + \dfrac{1}{12{,}3R_{\beta}}}} \tag{4}$$

3. <u>For vertical bridge pier of rectangular shape</u>

$$P = 0{,}30\ vh \sqrt{\frac{\Omega}{ah + \dfrac{1}{5{,}5R_{\beta}}}} \tag{5}$$

Equation (5) was obtained considering the approach of a round ice floe towards a rectangular pier, as it is improbable that a straight ice edge should simultaneously contact the pier along its entire width. Besides, there will be no local crushing effect in this case.

It is self-evident that the pressure calculated from equations (3), (4) and (5) cannot exceed the one determined on the basis on the crushing of the floe along the entire width on the pier.

It may be shown that for elastic response coefficients, $a < 0.001\text{-}0.005$, the piers may be considered as absolutely rigid.

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ICE SYMPOSIUM 1970
REYKJAVIK

THE PRESSURE OF FLOATING ICE-FIELDS ON PILES

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A. Synopsis

The investigation of the pressure of floating ice fields against piles has been started from the assumption that the maximum pressure of ice is limited by its compressive strength. Therefore this strength was first of all ascertained by laboratory tests on cubes of several ice species. The results contain the influence of temperature, velocity of deformation and direction of pressure on the cubic strength. In order to employ these laboratory results for the calculation of structures, a relationship between the strength in laboratory tests and in nature was derived by measuring the pressure of floating ice-fields on a pile of a bridge, which crosses the tidal estuary of the EIDER River.
The investigation leads to an equation, which allows the calculation of ice pressure against piles.

A. Résumé

Pour l'investigation de la pression des champs de glace flottante contre des pilots on a présumé que le maximum de la pression par la glace est limité par sa résistance à la compression. C'est pourquoi cette résistance représente une valeur fondamentale qu'on a verifiée par des essais à pression de cube pour plusieures espèces de glace, afin de déterminer sistématiquement les différentes influences comme la température, la vitesse de deformation, la direction de la pression et le contenu de gaz. Pour employer ces résultats de laboratoire pour le calcul des constructions on a mesuré la pression par la glace sur un pilot d'un pont afin d'obtenir une relation entre les résultats de laboratoire et ceux de la nature. Pour le moment les résultats conduisent à une équation qui permet le calcul de la pression de glace contre des pilots.

B. Introduction

In cold regions the pressure of ice is decisive for the calculation of hydraulic structures. This pressure, however, is still unknown. It is therefore not surprising that in severe winters hydraulic structures will be destroyed by ice run.

In rivers the danger of ice pressure decreases with time, because the ice run in spring will be controlled by ice-breakerships and because the ice formation will be reduced by the heated water of power-stations.

In coastal regions there is no way of keeping the ice forces from structures and just in these locations the question of ice pressure becomes more and more important, for example by the offshore-construction of deep-water harbors, transloading-points, light-houses and bridges.

The authors investigation (2, 1970) of the pressure of floating ice-fields on piles has been based on the assumption that the maximum pressure of ice is limited by its compressive strength. This strength was first of all ascertained in compression tests on cubes in order to determine systematically the different influences, such as temperature, velocity of deformation and direction of pressure. The received cubic strength cannot be immediately employed for designing structures, because in nature the rupture of ice occurs in another way than in our laboratory tests. In nature the contact between ice and structure is, for example, smaller than in the experiments between ice cube and pressure plate. Moreover the shape, the width of the structure and the thickness of ice has an influence upon the strength.

Because the fundamental strength properties nevertheless should be utilized for calculating ice forces, it was necessary to derive a relationship between the strength in laboratory tests and in nature. This was done by measuring the ice-forces on a pile of a bridge.

C. Laboratory tests

Strength properties were investigated by compression tests on ice-cubes from river, lake and harbor (fresh-water-ice) and from the North-Sea, Baltic-Sea and brackish-water (salt-water-ice).

The edge lengths of the cubes were 10 cm. The tests were performed at ice temperatures of 0^{o}, -10^{o} and -20^{o} C in two different directions (perpendicular and parallel to the growth-direction). The velocity of deformation was varied from $S = 3 \cdot 10^{-3} \frac{1}{sec}$ to $S = 3 \cdot 10^{0} \frac{1}{sec}$. Plywood panels were placed between the cube area and the pressure plate, in order to average out the unevenness on the cube surface, so that the test results scattered only up to ± 5 %.

Results

1. By lowering the temperature, the strength of ice increases at a rate of about:

   $\alpha = 4{,}5\ kp/cm^2 \cdot {}^{o}C$ with fresh-water-ice and
   $\alpha = 2{,}5\ kp/cm^2 \cdot {}^{o}C$ with salt-water-ice.

   This strengthening is nearly linear down to -20^{o} C. The lesser strength of salt-water-ice is attributed to the liquid brine cells within the ice.

2. At a deformation velocity of $S = 0{,}003 \frac{1}{sec}$ there is a maximum in strength (Fig. 1, 2). This result is explainable from the deflection-time-curve. The maximum appears at all ice species at the same strain rate and is more evident, the colder the ice is. The deformation velocity of $S = 0{,}003 \frac{1}{sec}$ corresponds to an ice sheet velocity of only a few cm/sec.

3. If the pressure acts parallel to the growth direction, the strength of fresh-water-ice is 20 % higher (Fig. 1) than if the pressure direction is perpendicular to the growth direction. With salt-water-ice these relations are just reverse.

4. Between air-content within the ice and strength exists a nearly linear relation.

D. Measurement in nature

The compressive strength of sea ice, as occurs in nature, was measured in winter 1967/68 and 1968/69 at a pile of a bridge, which crosses the tidal estuary of the EIDER during the construction of a tidal barrier.

Along the entire German coast of the North Sea and also just outside the estuary of the EIDER lie large flat areas (wadden ground), where ice fields can grow very quickly. These ice fields float up only at higher tides and then drift with the tidal current against the bridge, where the ice fields are cut up by the piles. In the hereby occuring state of stress the ice strength has maximum values.

The testing instrument consists of a shield with 50 pressure cells

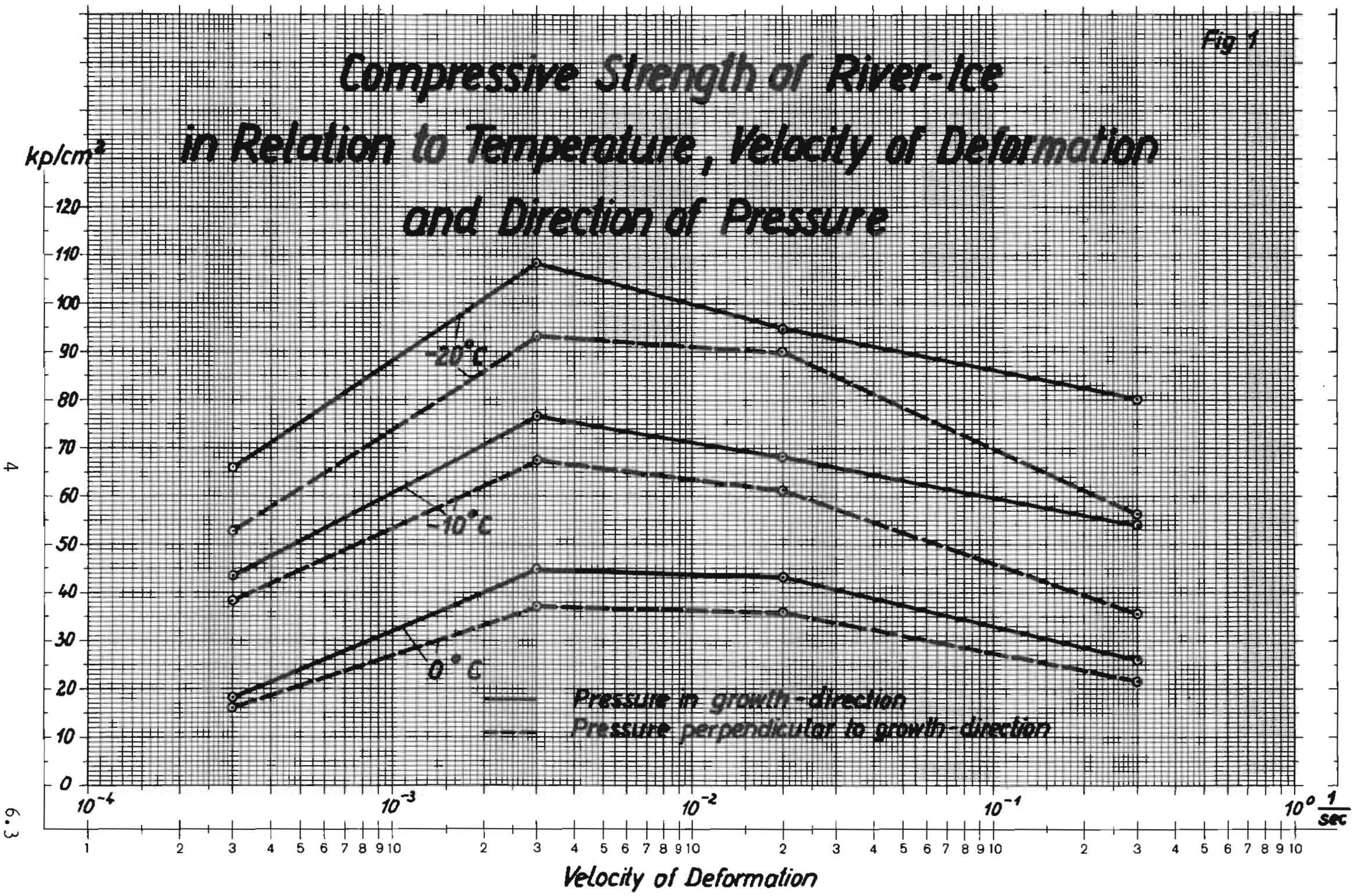
Fig 1
Compressive Strength of River-Ice
in Relation to Temperature, Velocity of Deformation
and Direction of Pressure
kp/cm²
0
10
20
30
40
50
60
70
80
90
100
110
120
-20°C
-10°C
0° C
Pressure in growth-direction
Pressure perpendicular to growth-direction
10⁻⁴
10⁻³
10⁻²
10⁻¹
10⁰
1/sec
1 2 3 4 5 6 7 8 9 10 2 3 4 5 6 7 8 9 10 2 3 4 5 6 7 8 9 10 2 3 4 5 6 7 8 9 10
Velocity of Deformation

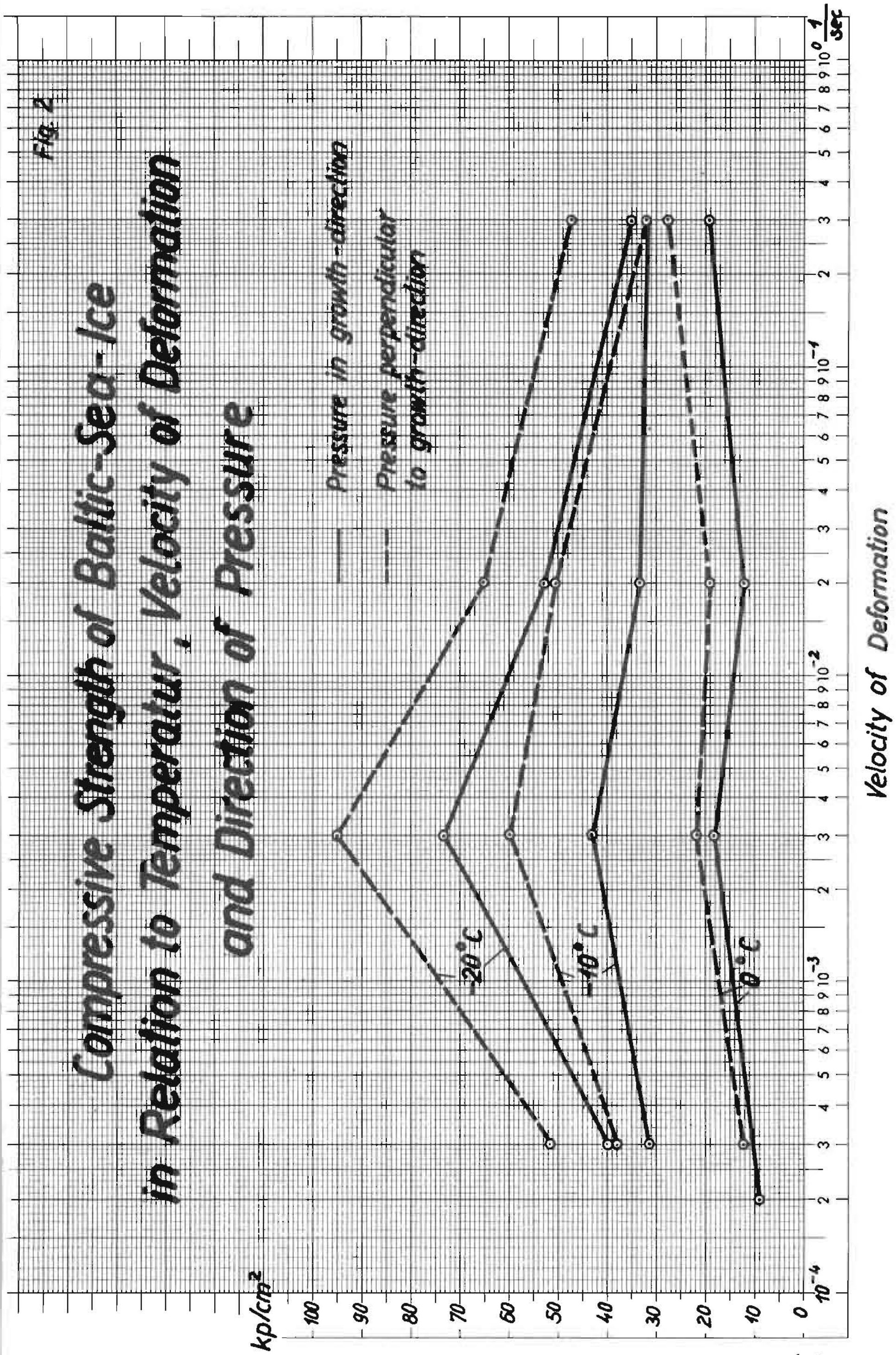

Fig. 2
Compressive Strength of Baltic-Sea-Ice
in Relation to Temperatur, Velocity of Deformation
and Direction of Pressure
Pressure in growth-direction
Pressure perpendicular to growth-direction
-20°C
-10°C
0°C
kp/cm²
100
90
80
70
60
50
40
30
20
10
0
Velocity of Deformation
1/sec

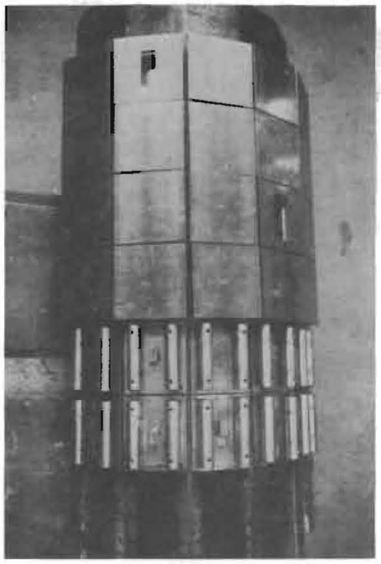

FIG. 3 SHIELD WITH 50 PRESSURE CELLS

FIG. 4 POINT FOR MEASURING ICE PRESSURE ON A PILE OF A BRIDGE

(Fig. 3), 5 in each altitude level halfway encompassing the pile (Ø 60 cm). The area of the pressure cells was 15 cm x 15 cm. In some of these pressure cells were situated smaller ones with areas of 25 cm^2 and 50 cm^2 in order to determine the relationship between strength and area of pressure. The shield was fixed on the sea-side of the pile (Fig. 4).

Insulating the electronic part of the pressure cell against salt water presented a particular problem. It was solved with BOSTIC-NEOSEAL and SILICON-CAOUTCHOUC.

Result

In a paper of probability (Fig. 5) the strength of sea-ice in nature, related to several areas of pressure ($\sigma_f \rightarrow 25\ cm^2$, $\sigma_F \rightarrow 200\ cm^2$, $\sigma_\emptyset \rightarrow 840\ cm^2 \rightarrow$ the whole width of the pile at an ice thickness of 14 cm) was compared with the cubic strength from laboratory tests with the same ice.

1. If the pressure is related to an area of F' = 200 cm^2 the compressive strength in nature ($\sigma_{F'}$) is only half of the cubic strength (σ_{WB})

$$\frac{\sigma_{F'_{99}}}{\sigma_{WB_{50}}} = \kappa_{F'} = 0,5$$

This reduction of the cubic strength is attributed to the incomplete contact between ice and structure. Therefore κ is called contact coefficient, although this value includes the different state of stress in the cube

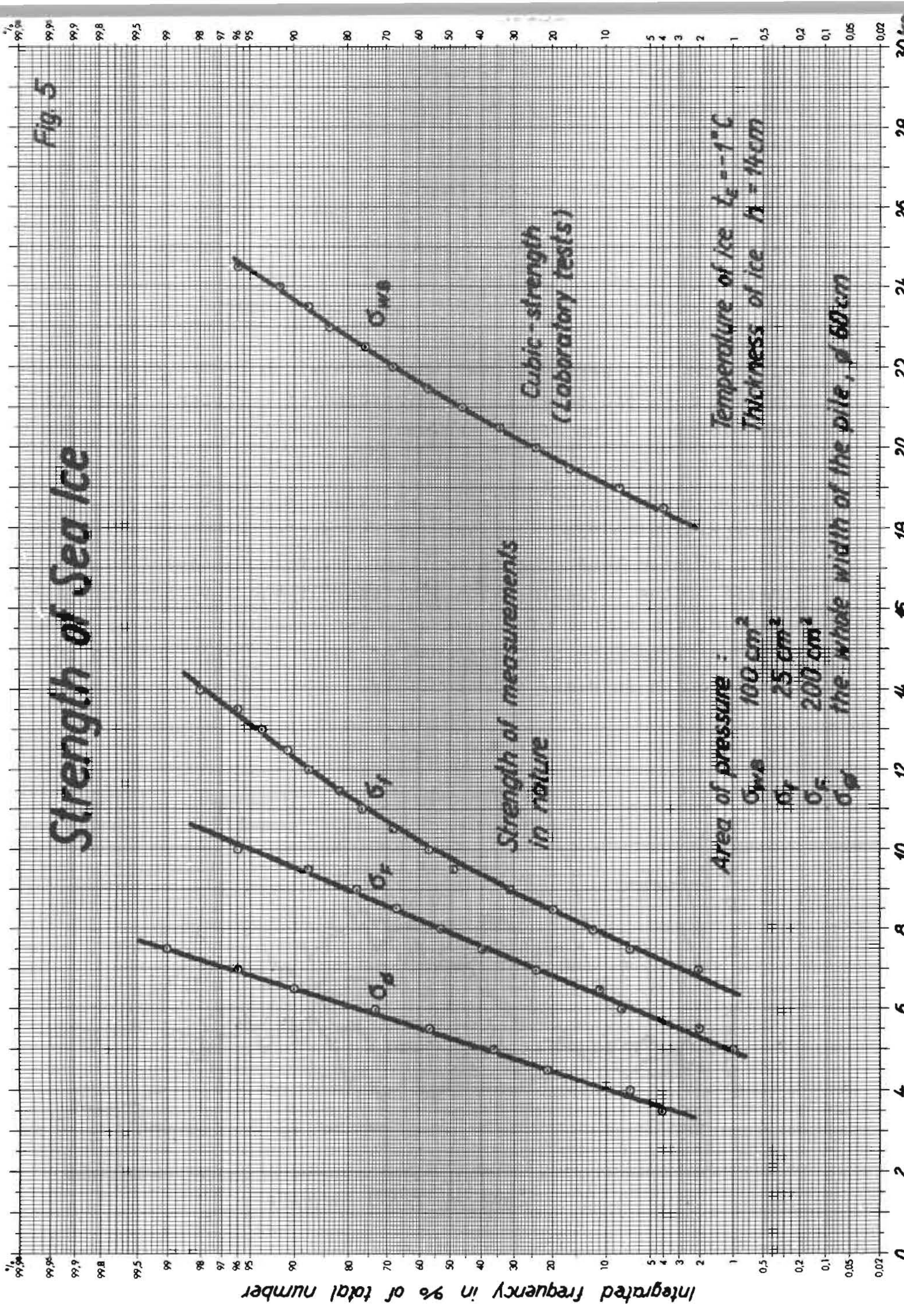
Fig. 5
Strength of Sea Ice
Cubic-strength
(Laboratory tests)
Strength of measurements
in nature
Temperature of ice $t_E = -1°C$
Thickness of ice $h = 14 cm$
Area of pressure :
σ_{WB} 100 cm^2
σ_f 25 cm^2
σ_r 200 cm^2
σ_d the whole width of the pile , ⌀ 60cm
Integrated frequency in % of total number
30 kp

pressure experiment and in nature.

2. If the area of pressure is only $f = 25\ cm^2$, the coefficient of contact increases to $\kappa_f = 0,56$.

3. Because the peaks of pressure (Fig. 6) occur simultaneously only on 1 or 2 of the 5 side by side load cells - the others being largely unpressured - a second reduction factor from the proportion

$$\frac{\sigma_{\emptyset}}{\sigma_F} = 0,66 \quad \text{was ascertained.}$$

$\sigma_{\emptyset}$ is the mean pressure over the whole width of the pile × thickness of ice, projected in the direction of floating. "0.66" takes into account first of all the shape of the structure, but also the increase of the area of pressure from 200 cm^2 to 840 cm^2.

4. From measurements of different thicknesses of ice follows, that the strength of ice increases, if the ratio thickness of ice to width of pile becomes greater (Fig. 7). This is caused by the increase of the threedimensional stress. If the ice sheet grows thicker, the number of planes of shear increases linearly, but also the extension of the planes of shear is lengthend, so that the strength increase exponentially.

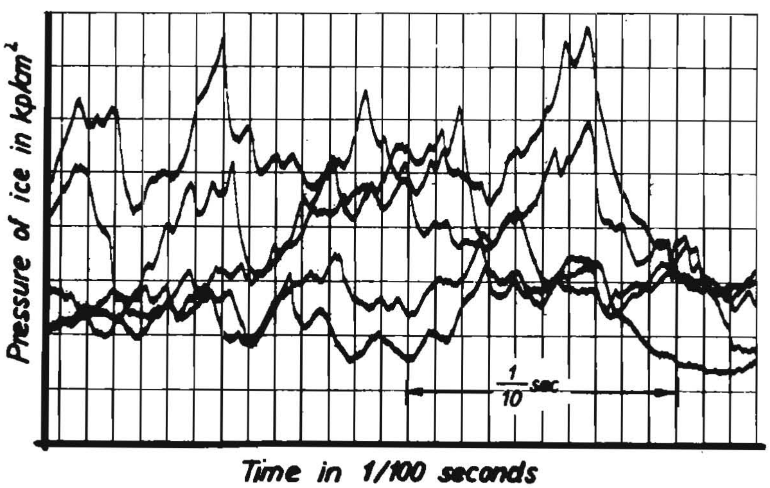

FIG. 6 ICE PRESSURE ON 5 SIDE BY SIDE PRESSURE CELLS

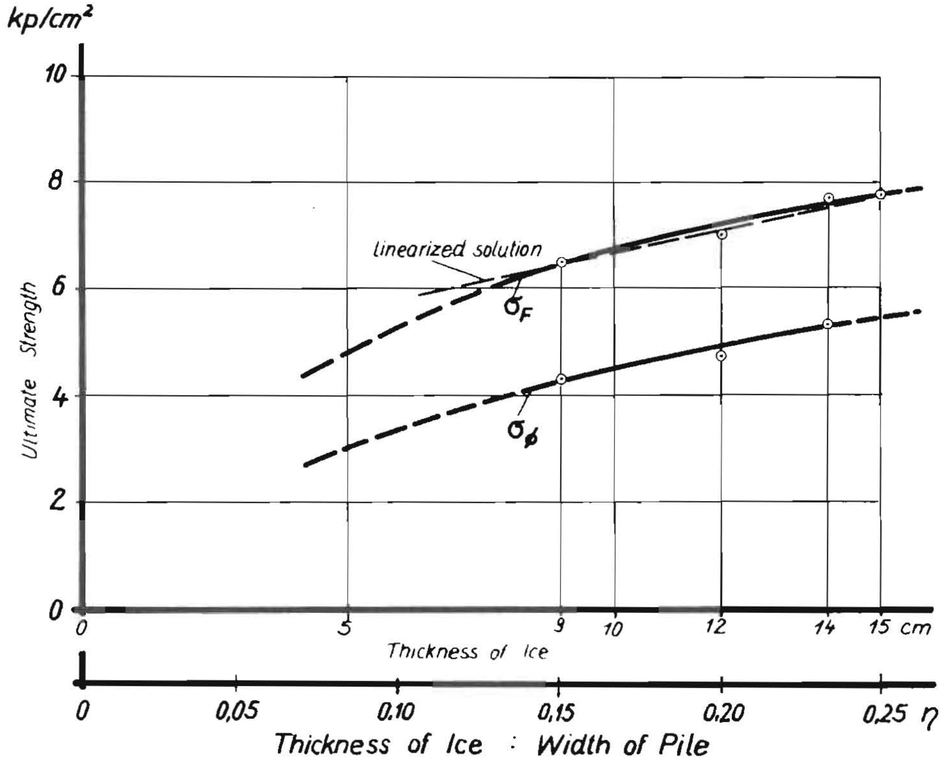

FIG. 7 RELATION BETWEEN ULTIMATE STRENGTH AND THE RATIO THICKNESS OF ICE TO WIDTH OF PILE

5. The pressure of brittle ice with low cubic strength was nearly the same as the pressure of new ice, because of the closer contact between brittle ice and structure. It can be assumed, that the coefficient of contact decreases with lowering temperatures.

From the results of the laboratory tests and the measurements in nature the following equation is derived in order to determine the maximum pressure of floating ice-fields on piles:

$$P = \left[0,5 \cdot 0,66\ (\sigma_{WB}(0^{\circ}\ C) + 0,35 \cdot \alpha\ (t_L - t_w)) + 12,5\ (\eta - 0,15)\right] h \cdot b$$

0,5	= Coefficient of contact
0,66	= Coefficient of form (pile Ø 60 cm)
$\sigma_{WB}(0^{\circ}\ C)$	= Cubic strength of ice at 0° C and a deformation velocity of $S = 0,003\ \frac{1}{sec}$

$0{,}35 \cdot \alpha(t_L - t_w)$ = Influence of temperature

α = Temperature factor

t_L = Air-temperature during the last 24 hours

t_w = Water-temperature

0,35 = Factor to get the mean temperature of the ice-sheet (after KORZHAVIN, 1)

$12{,}5\ (\eta - 0{,}15)$ = Influence of thickness in proportion to the width of pile

η = Thickness of ice: width of structure

h = Thickness of ice

b = Width of pile

This equation should be verified at higher values of η and at low ice temperatures.

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DISCUSSION:

C.R. NEILL: Dr. Assur (guest lecture) explained Dr. Korzhavin's contact coefficient in terms of statistical correlation of force peaks on different parts of a contact area. Dr. Schwarz introduced the same contact coefficient, but also a second reduction factor based on incomplete correlation of peaks.

Could the author and perhaps Dr. Assur comment on this point? If there is a reduction factor for incomplete correlation of peaks, what is the correct interpretation of the contact coefficient?

J. SCHWARZ: There are two reduction factors of the cubic strength; the first is called contact-coefficient (0,5); it is related in our case to an area of pressure of 200 cm^2. It is affected by the incomplete contact between ice and structure and by the state of stress within the ice.

The second reduction factor (coefficient of form = 0,66) is given, if the consideration of the ice forces will be extended from one pressure cell to the whole width of the pile. This coefficient depends also on the contact, because of the increase of the pressure area, but the influence of shape is favourised.

R. EDWARDS: First, I should like to compliment Dr. Schwarz on an excellent experiment and paper. I would like to know however, what his opinion is on the influence of the vibration of his instrumented pile upon the validity of the force measurements, which he has accumulated. The vibration was quite noticeable in the informative film, he showed in his presentation.

J. SCHWARZ: Of course, there was a vibration of the testing pile, but this fact only has an influence upon the velocity of ice deformation. For example, if the pile oscillates forward, the relative floating velocity of the ice sheet, which is proportional to the velocity of deformation, increases and on the contrary, if the pile oscillates backward, the velocity of deformation decreases.

The relationship between the ice strength and the velocity of deformation was investigated in laboratory tests and is taken into account in the equation on page 6.

PEYTON, H.: Dr. Schwarz has accomplished a fine piece of experimental and analytical work. The data should be useful in developing further results in analyzing ice forces on structures. Especially of interest are the correlations between force at a specific location to the unit areas surrounding it, also the correlation of force at a unit area from one time to another. Both of these analytical results should prove useful in predicting ice forces on a variety of structural types.

Both the data and the movie show excellent correlation with COOK Inlet ice conditions as I have observed them.

ICE SYMPOSIUM 1970
REYKJAVIK

THE NUTCRACKER ICE STRENGTH TESTER AND ITS OPERATION IN THE BEAUFORT SEA

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Synopsis

Growing energy consumption and dwindling reserves in the populated regions is spurring the search for oil in the frontier areas of the far North. This paper describes the work sponsored by several Canadian oil companies aimed at providing ice strength data for the design of offshore structures for Arctic waters. The design and operation of a novel ice testing device is described. Some of the results obtained during tests in January and February of 1970 are presented and discussed.

La consommation croissante de l'énergie et l'épuisement des réserves dans les régions peuplées ont poussé la recherche du pétrole jusque dans les régions éloignées du Grand Nord.

Cet article décrit le travail financé par plusieurs sociétés pétrolières canadiennes destiné à obtenir les informations sur la résistance de la glace nécessaires au dessiøn des structures qui devront être utilisées pour le travail au large dans les eaux Arctiques. L'auteur décrit un nouvel appareillage pour la mesure de la résistance de la glace ainsi que son fonctionnement.

Quelques-uns des résultats obtenus en janvier et février 1970 sont présentés et discutés.

Introduction

Increasing population and affluence of the industrial nations has resulted in a big increase in world energy consumption. Consequently the easily accessible fossil fuels which have supplied North America and Europe since the Industrial Revolution are becoming exhausted, and an international search for new sources of energy is underway. The North American oil industry is playing an important part in the search by exploring frontier areas, the most significant of these being the far North.

The Arctic sedimentary basins extend under both land and sea. Offshore development of these basins will be made difficult by the ice environment which exists year round in most areas.

One of the obvious problems facing offshore development in the Arctic is that of providing structures strong enough to withstand the moving ice. The limiting force between a fixed structure and moving ice is a function of the failing stress of the ice in the relevant mode of failure. Hence knowledge of the maximum nominal ice stress or pressure is an important prerequisite to preliminary design studies of offshore Arctic structures. During preliminary consideration of the problem we could not find reference to any previous work which would allow us to predict the maximum nominal failing stress of the ice in the Mackenzie Delta area of the Beaufort Sea (which is the area of our immediate interest). We were aware of the work of Peyton[1] and Assur in relation to the Cooke Inlet platforms but did not have access to the complete results of their studies. In any case the ice conditions of the Cooke Inlet and Mackenzie Delta are not similar and we took note of Gold's remarks[2] on the desirability of field measurements.

Consequently in the summer of 1969 we started to consider various methods of conducting field tests which would give us the required data.

The Test Proposal

Our initial objective was to investigate the failure of a horizontal ice sheet caused by the relative movement of a vertical cylindrical structure. More specifically the aim was to determine the failing loads involved and also record the modes of failure.

Originally we considered installing several instrumented test piles in Kugmallit Bay near Tuktoyaktuk, N.W.T. After freeze-up, the ice forces would be determined by measuring the bending strains in the piles. Preliminary study of this proposal revealed several technical difficulties and also the high costs which would be involved.

It was natural therefore, that we started to think about alternative methods of obtaining the same information (i.e. the failing stress of ice in the pier-mode of loading). Instead of installing fixed piles and measuring the forces imposed by ice moving against them, why not push between two tubular members set in the ice, and measure the load-deflection characteristics up to the point of ice failure? This new line of approach seemed attractive in many ways and several alternative methods of moving tubular members through the ice were considered. Of all the alternatives, the so-called "nutcracker" device, illustrated in Figure 1 and described in the next section appeared to be the simplest to engineer and operate.

The design, manufacture and shipment of four nutcracker testers attracted interest and received a certain amount of publicity. This publicity resulted in participation in the project by five other oil companies and eventually led to the formation of the Arctic Petroleum Operators Association. A.P.O.A. has currently a membership of nineteen companies and has so far been involved in a total of four major Arctic research and data gathering projects.

The "Nutcracker" Device and Associated Equipment

The principle of operation of the device is readily seen from Figure 1. In essence, the two loading legs are hinged at the bottom and can be pushed apart by means of up to three hydraulic rams which link the legs together at the top. The outriggers and floats are necessary to enable the device to be floated in the water just prior to freeze-up. After freeze-up, the floats can be disconnected from the loading legs so as not to interfere with the tests.

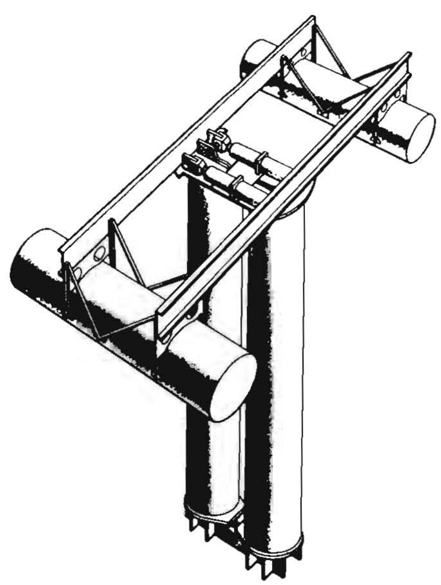

Figure 1. The Nutcracker Ice Tester

The design case for the devices was five feet of ice and a 600 psi nominal ice failing stress.

Each nutcracker device weighs about 9 tons, most of this weight being in the loading legs; each leg is 16 feet long by 30 inches diameter by 1 inch thick. Each hydraulic ram weighs 300 pounds and is capable of a push of 172 tons at the maximum hydraulic pressure of 10,000 psi; ram stroke is 12 inches. Hydraulic fluid at a pressure up to 10,000 psi can be supplied to the rams through flexible hose from a 16 horsepower gasoline powered pump. (Enerpac P.G. 1528 Hi flow.) The rate of loading is controlled by manually operating an adjustable pressure relief valve (Enerpac VC-110).

The load applied to the ice is deduced from the hydraulic pressure which is measured by means of a pressure transducer (Viatran PTB 103) mounted on the inlet to the central ram. The signal from the transducer is fed into an oscillographic recorder so that load is obtained on a time base. The distance between the two loading legs is measured by means of a linear potentiometer (Bourns model 128) and this signal is also recorded.

Four nutcracker units were manufactured at National Tank Limited in Calgary. They left Calgary for Hay River on August 13th and were barged down the Mackenzie to arrive at Tuktoyaktuk by September 15th. The units were then assembled and anchored in Tuktoyaktuk harbour to await freeze-up. The units were disturbed by storms during freeze-up and they were finally locked into the ice closer than we had intended, however this proximity did not affect their operation or the results obtained.

Field Tests

Unfortunately I cannot describe in this paper the detailed results obtained from the tests. This is because the test program was jointly sponsored by several oil companies and the terms of agreement require that the results remain confidential until 1974. However, I hope that what I can disclose, even though it is of a general nature, will be of sufficient interest to justify the presentation of this paper.

All four test devices were successfully operated within a ten day period in January 1970. The equipment and apparatus worked remarkably well despite temperatures in the -20°F to -40°F range.

Typical ice stress and deflection results are shown in Figure 2. The deflection at failure (referred to the centre of the ice sheet) was about one inch. With this amount of deflection the loading leg was almost exactly at right angles to the ice sheet. The ultimate deflection occurred entirely on one leg only.

All four units yielded rather similar plots of stress against time and deflection against time, despite the differences between tests in such things as loading rates, ice temperature and salinity profiles, and loading leg geometry.

In the above context it should be noted that: three of the units had loading legs which were 30 inches in diameter whilst the other one simulated a 60 inch diameter pier; the loading rate up to failure varied from test to test between 200 and 1500 psi/min; the salinities were generally low - the highest value measured being about 3 parts per thousand by weight. Another parameter which affects nominal ice stress and which was not easily determined is effective ice thickness. Not unexpectedly, the ice thickened locally around the units as shown in Figure 3. In view of the fact that ultimate failure was very local around the loading leg the effective ice thickness was assumed to be equal to the local ice thickness; nearly twice the nominal ice thickness.

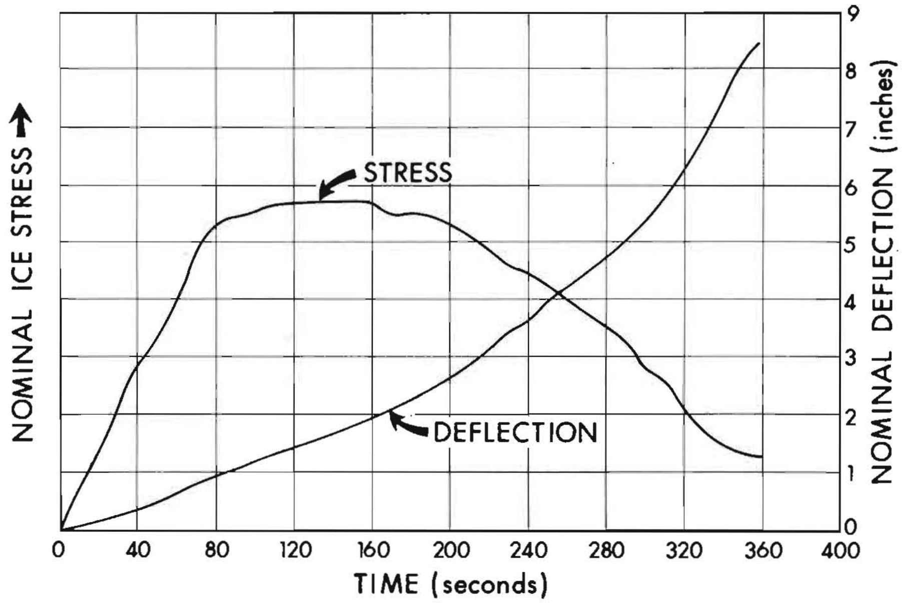

Figure 2. Typical Test Record

Figures 4 to 9 are typical views of the test devices in operation. Observation of the tests and study of the photographic records leads me to the conclusion that after a stress re-distribution by tensile and shear cracking, the ultimate failure was one of local crushing around the loading leg.

The test devices were re-operated in February. Similar results were obtained except that because of thicker ice, we did not have enough load capacity to fail the larger unit in the elastic mode. However, we did induce creep failure by maintaining the maximum available load for over one hour.

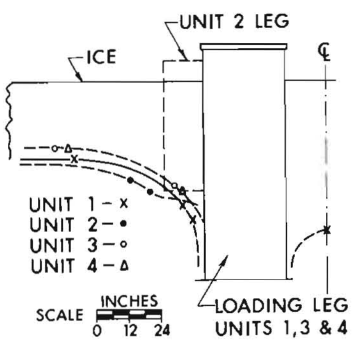

Figure 3. Ice Thickness Contour

Figure 4. Test in progress; ram stroke about 2 inches

Figure 5. Test in progress; ram stroke about 8 inches

Figure 6. Test in progress; ram stroke about 9 inches

Figure 7. Test complete; inspecting the failure

Figure 8. Local crushing in front of loading leg

Figure 9. Another test in progress; note radial cracks

Closure

The tests described have greatly increased our knowledge about the pier-mode type of failure of Mackenzie Delta ice. However in relation to an offshore structure they are essentially model tests and the problem remains of how to extrapolate the results to the design of a large platform.

Acknowledgement

This contribution is presented with the approval of the Canadian oil companies participating in the project, namely: Amoco Canada Petroleum Company Ltd., Chevron Standard Limited, Elf Oil Exploration & Production Canada Ltd., Gulf Oil Canada Limited, Imperial Oil Limited, Texaco Exploration Company.

I would like to express my thanks to all those engineers and technicians who have given advice and assisted in the project. In particular I thank Mr. R.E. Hedley for his loyal field work.

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DISCUSSIONS

G. Frankenstein

I would like to say that this was a tremendous piece of work. We are conducting Model Studies at CRREL which are similar but better (because we can control the conditions). In preliminary tests we observed an increasing load until a circular crack formed, then the load would fall off until the pile reached the failure crack then the load increased to a new maximum value - then a circular crack would form and the process would repeat.

B. Ross

(1) Please comment on effects of rotation of nutcracker leg on the results which were obtained.

(2) What were characteristic temperatures and salinities for sea ice tested?

K.R. Croasdale

At failure the leg movement through the ice was between one and two inches which corresponds to an angular tilt of only about 0.4 to 0.8 degrees. Furthermore the legs were preset inwards 1.5 inches so that in fact at peak load the loading leg was almost perfectly at right angles to the ice sheet.

Temperature and salinity profiles were obtained immediately before each test. Salinities up to 3 parts per thousand were measured; a typical ice surface temperature was -9°F.

S. Hanagud

As a further comment to your reply to Dr. Ross, I would like to mention that even presetting will not completely correct for rotation and there are bending stresses introduced in the ice-slab. These effects can be calculated and appropriate correction can be applied to calculate the ice-failure stress.

K.R. Croasdale

I accept that our loading was not purely horizontal. However, I think it is unlikely that the very small rotation up to failure induces bending stresses in the ice which are of any significance. Even so, we shall attempt to calculate this effect.

ICE SYMPOSIUM 1970
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THE INTERACTION BETWEEN STRUCTURES AND PRESSURE RIDGES

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SYNOPSIS

Offshore structures of a fixed nature may initiate the formation of pressure ridges in sea ice. These pressure ridges in turn may create additional pileups of sea ice, thus isolating the structure to ships from the ice sheet field by a possibly impenetrable barrier.

There are virtually no offshore, man-made structures in Arctic areas of North America. The writer will, of necessity, call upon observations of reactions between some of nature's temporarily fixed ice islands and dynamic sea ice as well as on observations of the reaction between a moving artificial structure, the S.S. MANHATTAN, and pressure ridges.

There has been considerable speculation on the effects of dynamic pressure ridges striking coastal structures. A few rules-of-thumb have been developed, but these have not been widely accepted, as most designers of potential offshore structures are not satisfied to design on the basis of untested, empirical formulae. The writer has been fascinated by the fact that the literature and general discussion among engineers often considers the effects of pressure ridges on structures but seldom considers the conversely possible effects of structures on pressure ridges. It is this latter topic which is the prime subject of this paper.

During the winter of 1968-1969 many ice island fragments washed onto the coast of northern Alaska in the vicinity of Prudhoe Bay. These ice fragments were fresh water fragments which had broken off from the Ward Hunt ice shelf of Ellesmere Island many years before. They varied in size from only a few feet to 10 acres in area, and they became grounded in depths ranging from approximately 20 feet to 90 feet. The larger fragments formed a series of ice islands following roughly the 90-foot depth contour approximately 20 miles from shore, for a distance of 70 miles along the coast. After two winters and a summer, this string of islands is still grounded off the coast.[1] Though the islands have fractures as a result, apparently, of temperature cracking and have been through the melt season of 1969, it appears that they will be in existence for several years, diminishing in size each year. University of Alaska engineers thickened two of these islands by adding approximately 12 feet of ice to the top of one and 3 feet of ice to the top of another, thus providing average overburdens of approximately 300 pounds/ft^2 and 150 pounds/ft^2 respectively.[2,3]

These ice islands are essentially offshore structures and have afforded University of Alaska engineers an excellent opportunity to study some aspects of the interaction between offshore structures and pressure ridges. Through the winter of 1968-1969 the ice islands created a continuous shear pressure ridge just seaward of the line of islands. At one location the height of the ridge was approximately 30 feet, and of course this ridge was grounded along much of its length because of the limited water depth. In very few places was the shear ridge less than 10 feet in height, probably averaging on the order of 15 feet high. The horizontal extent of the shear ridge was from 100 to 300 feet.

During the winter, ice moving outside of the ridge piled up against the ridge and broadened it seaward in the horizontal direction normal to the axis of the ridge. In many places this shear ridge apparently initiated the formation of many more compression and shear ridges further seaward. On the shoreward side of the ice islands, the ice was generally rather smooth, and though it has been fragmented while the ice was thin, as the winter progressed and the ice thickened to 6 or 7 feet there appeared to be relatively little motion of this shoreside ice sheet. The line of barrier ice islands remained stationary after the ice had thickened to from 4 to 5 feet, although there was a great deal of movement of the sea ice just outside of the string of islands.

It is apparent that a shear ridge formed as a result of the presence of offshore structures lends protection or provides an energy-absorbing buffer zone for structures of this type. It is also apparent, however, that such ice build-ups initiated by offshore structures would render the structures quite difficult to approach from the seaside by ships, since the ice in the shear ridge and subsequent ridges outside of the shear ridge would pile to considerable heights and be generally grounded on the bottom, thus leaving no place for the ship approaching the structure to dislocate the ice. Experience of the voyage of the S.S. MANHATTAN indicates that very dynamic, loosely cemented, first-year pressure ridges provide virtually no obstacle to a ship if the ship is able to displace the ice horizontally or vertically.

Through the winter of 1969-1970 the barrier islands had diminished in size and moved somewhat shoreward. A shear ridge developed in a fashion similar to that of the previous winter, but the ridge was generally not as pronounced -- that is, the ice in the ridge did not pile up as high as it had the previous winter. The writer speculates that since the barrier ice islands were located closer to the shore, they did not experience as much dynamic ice on their seaward side as they did the first winter.

From combined experience thus far, University of Alaska research engineers are generally of the opinion that where single-year ice is involved pressure ridges do not provide any particular additional hazard to structures which are at least ten times as broad as the sheet-ice thickness. This is because the ice fragments which form the pressure ridge are loosely cemented and the ridge is easily severed. Such an observation is certainly consistent with the experience of the S.S. MANHATTAN when encountering pressure ridges of recent origin, regardless of their sizes.

It is important to note that offshore tanker loading schemes involving submerged loading facilities could be badly affected by pressure ridges of any kind moving past the tanker as it lies in a loading posture. The connection between the ship and the loading facility could well be severed by ice moving beneath the ship if the joint interaction of ice, ship design, and proximity to bottom is not well solved.

The question of the interaction between offshore structures and old pressure ridges which have had an opportunity to survive one or more summers of melt is, to the best of the writer's knowledge, unstudied and one which requires field study.

Until this phenomenon is carefully studied in the field, structures cannot be safely designed to withstand large, healed pressure ridges. Certainly experience, once again, on the S.S. MANHATTAN indicates that such pressure ridges are extremely formidable objects for a ship, and they would present great force against a fixed structure.

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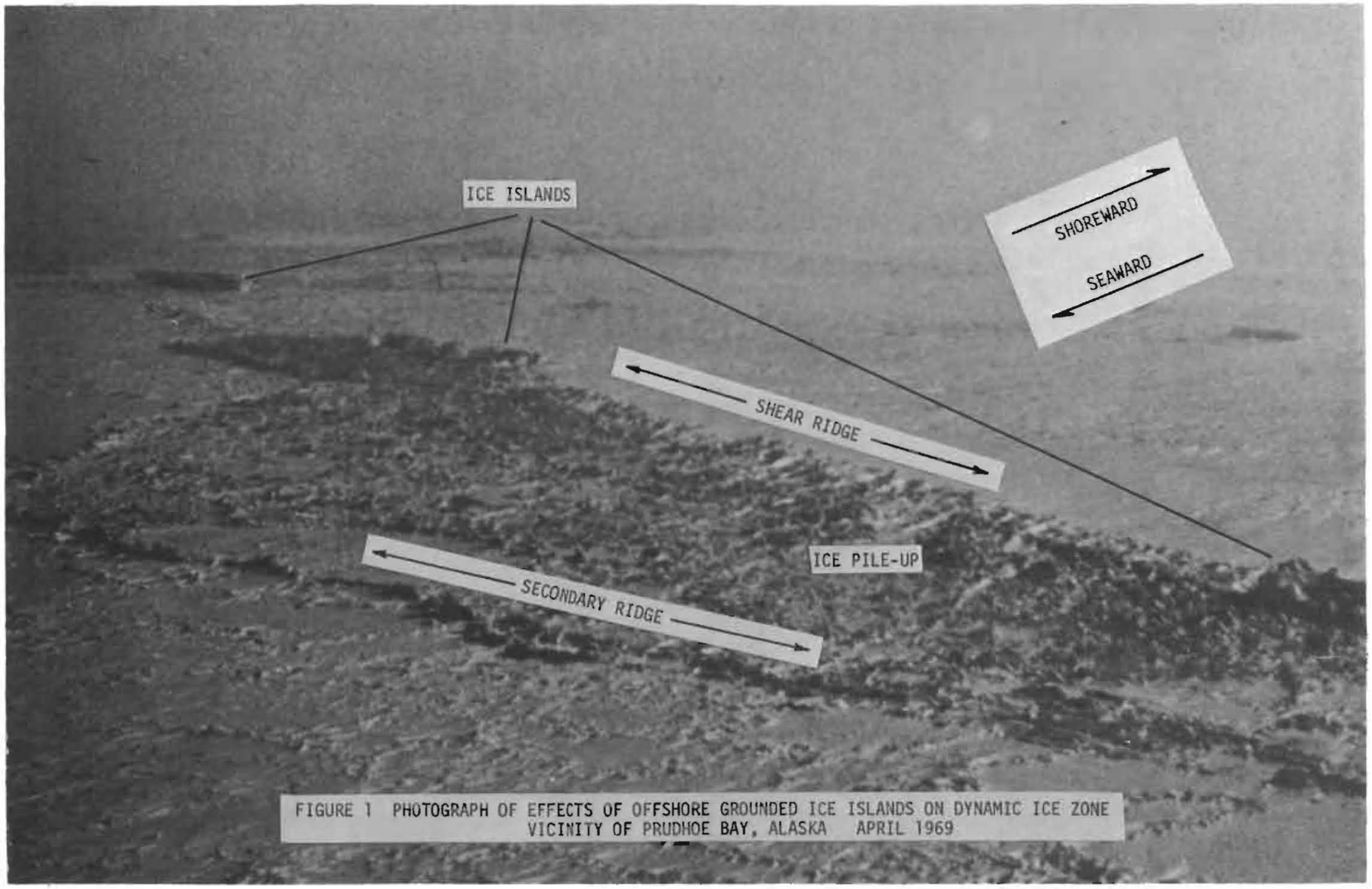

FIGURE 1 PHOTOGRAPH OF EFFECTS OF OFFSHORE GROUNDED ICE ISLANDS ON DYNAMIC ICE ZONE
VICINITY OF PRUDHOE BAY, ALASKA APRIL 1969

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ICE PRESSURES AGAINST ISOLATED STRUCTURES

E. Dinkla - Vice President
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1 -INTRODUCTION

The paper describes the approach taken in determining the ice loadings on some offshore structures located in areas subject to heavy ice movements.

2 - NORTHUMBERLAND STRAIT CROSSING

The site of the proposed crossing is in the Atlantic between New Brunswick on Canada's mainland and Prince Esward Island.

The area is characterized by large ice fields several miles in extent, including pressure ridges which could move at speeds up to a maximum of 8 feet per second following completion of the crossing.

To establish design criteria, investigations were made resulting in a maximum ice thickness of 4 feet, an average compressive strength of 220 psi and a theoretical pressure ridge extending 8 feet above and 40 feet below the ice sheet as shown in Figure 1. It was decided to include a safety factor of 2 in the design loading to allow for the margin of uncertainty, particularly for the effect of the pressure ridges.

Consideration was given to the effects of the ice on both cylindrical and conical piers.

(a) - Cylindrical Pier

As an illustrative example, the total ice force will be determined for a 25-foot diameter cylinder, using the formula $p = I \times m \times k \times 220$ psi

The various factors are adopted from tests conducted by Korzhavin[(1)] and from research by Dr. Assur.

Inserting the factors gives an effective crushing stress of 180 psi. This corresponds to a total force of 2,590 kips.

The effect of pressure ridges and the design safety factor of 2 are not reflected in this load. The ridges act as a local reinforcement of the ice sheet. Field measurements made over a period of two years on 14-foot diameter cylinders located in Cook Inlet, Alaska indicated that the load from a ridge could be from 1.5 to 3.6 times greater than the load from the parent sheet. Predominant values varied between 2.0 and 2.5.

A factor of 2.2 was adopted, giving an ice load including ridges of 5,700 kips. The final design loading, including the safety factor of two is 11,400 kips.

This example illustrates that whereas the crushing strength of a uniform ice sheet can be defined within reasonable limits, this is not the case with pressure ridges Large safety factors will have to be applied until more definitive data become available.

(b) - Cone-shaped Pier

The magnitude of the design ice force on a circular pier led to the adoption of a cone-shaped pier, designed to fail the advancing ice sheet in bending.

As an ice sheet advances, the area of contact between the ice and the cone increases, resulting in vertical and horizontal forces being imposed by the pier on the ice sheet. The vertical force is the active load inducing bending in the sheet. (Figure 2 and 3)

Methods developed by Nevel [2] and Meyerhof [3] were used in the analysis.

As an illustrative example, a 45-degree cone, measuring 74 feet in diameter at water level will be considered. The vertical load to fail the 4-foot ice sheet was found to be 270 kips.

As the ice is moving at a high velocity, an appreciable vertical acceleration of the leading edge increases the vertical load over that under slow loading conditions. This load increase was estimated to be 200 kips.

In assessing the effect of ridges, many different ridge patterns were visualized. Ridges may be located so as to increase the bending strength of the ice sheet or, they may be located near the leading edge of a sheet and be subject to acceleration forces as described before.

The additional load from ridges was estimated to be 580 kips and the total vertical load to fail the ridged ice sheet was 1,050 kips.

Here again, a design safety factor of 2 was included and the design ice loading was 2,100 kips vertical. The corresponding horizontal component would be 2,800 kips.

The example illustrates, as in the case of the cylindrical pier, that the failure strength of a uniform ice sheet can be defined within reasonable limits, but that many assumptions have to be made to assess the strength of a ridged ice sheet.

3 - AN OFFSHORE STRUCTURE IN THE ARCTIC

For a recent design in the Arctic, a conical-shaped structure was adopted for rather different reasons.

The structure would be subject to forces resulting from an ice sheet up to 7 feet in thickness, with pressure ridges which could be several years old and well consolidated. The tidal range in this particular area is small and the ice cover may be stationary for extended periods of time. The ice sheet could, therefore, freeze to the structure and circumvent the development of the bending type of failure.

A structure such as this would have to be designed to resist the force resulting from a crushing type failure. The cone shape in this situation has no advantage over a cylinder with the same diameter at water level.

The real advantage of the cone, in this particular application, is in the breaking up of the advancing ice ridges.

Assuming, for illustrative purposes, a diameter at water level of 150 feet and an effective crushing strength of the ice of 300 psi, the total load from the ice sheet is 45,000 kips.

If for design simplicity the ridged sheet is assumed to be equivalent to a uniform sheet two and four times the original 7-foot thickness, the vertical force required to break these sheets in bending can be calculated and is 4,000 and

16.000 kips respectively. This increases the original load by a fraction only. The load increase from ridges crushing against a cylinder is obviously much greater and shows the advantage of the cone.

CONCLUSION

The loading from ice ridges may, in certain cases, greatly exceed the loading from the basic ice sheet and it is necessary to assess:

(a) - The size and strength properties of a design ridge;

(b) - Its mode of failure and the resulting force imposed on the structure.

Many assumptions may have to be made to assess the load increase. The random pattern and the heterogenous nature of the ridges make such an assessment difficult.

Ice knowledge in this field would be greatly advanced by a number of large scale tests conducted in areas with different ridge characteristics.

One such program is being planned by the Department of Transport of the Government of Canada. Ice measuring panels to determine the overall ice load are to be installed on some of the new conical-shaped lighthouses to be constructed in the Gulf of St. Lawrence.

The ice measurements taken and data collected by the U.S.S. Manhattan for a great variety of ice conditions during its trip through the North West Passage should also greatly advance ice knowledge in this area.

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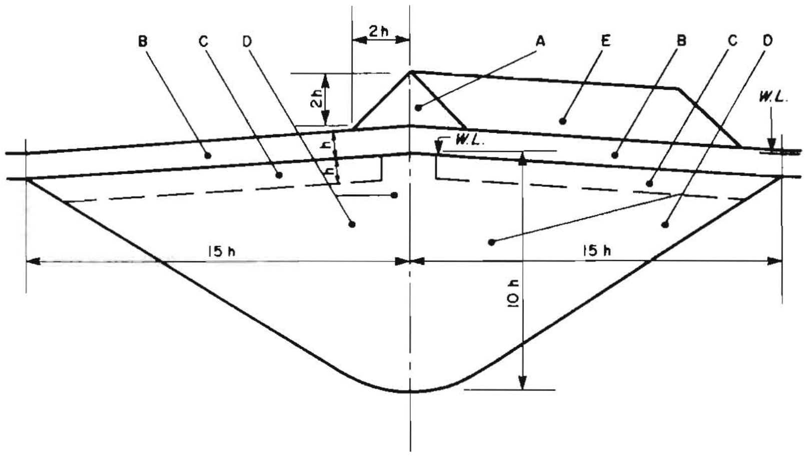
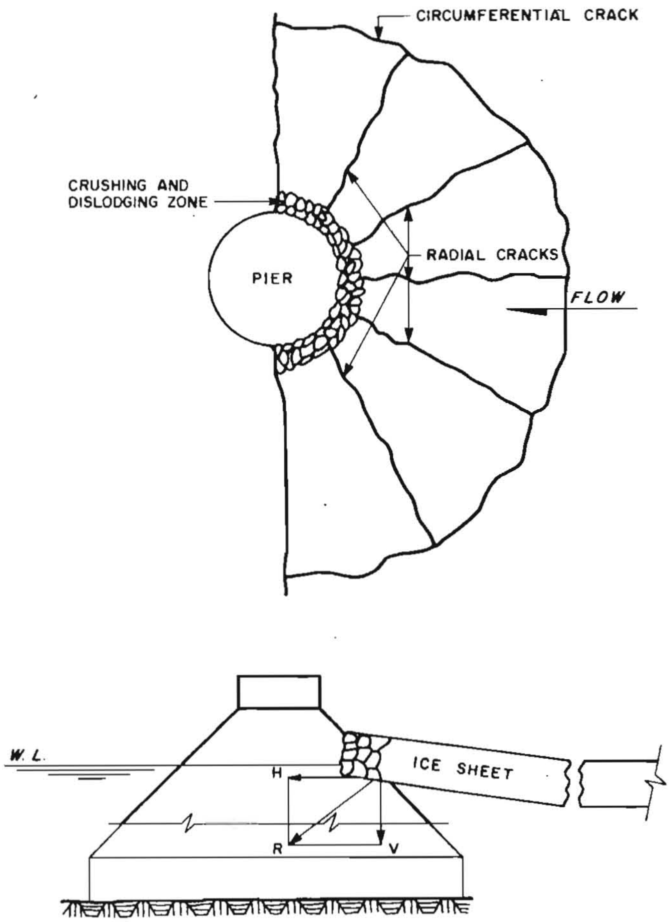


LEGEND
A = CONSOLIDATED BLOCKS
B = SOLID ICE SHEET
C = BLOCKS PARTIALLY CONSOLIDATED TO SHEET
D = LOOSE BLOCKS IN WATER
E = EXTENDED PEAK OR 2nd. RIDGE
AT 90° TO FIRST
h = THICKNESS OF ICE SHEET

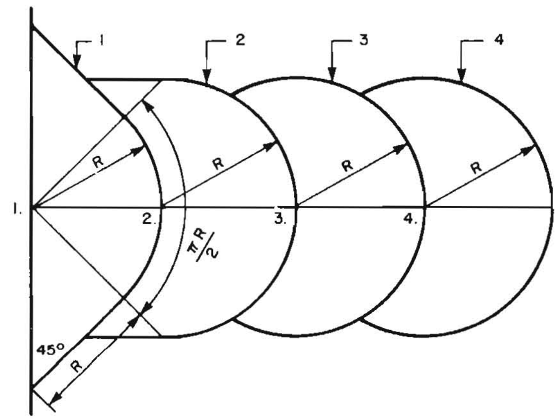
MODEL PRESSURE
RIDGE DIMENSIONS

FIGURE 1



TYPICAL CRACK PATTERN
ICE SHEET FAILING IN FLEXURE

FIGURE 2



FAILURE LINES
CONES BREAKING THROUGH ICE SHEETS

FIGURE 3

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THERMAL ICE PRESSURE

PRESSION THERMIQUE DE GLACE

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L'école Royale Polytechnique Supérieure,
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Single-axis tests with 7 x 7 x 20 cm ice prisms have been carried out. The relationship between pressure, deformation and time is used to determine the elasticity and viscosity of the ice. Double-axis tests, at temperature increase and with restricted expansion on an ice sample 80 cm in diameter and 7 cm thick have been carried out. The relationship between the pressure and speed of deformation is used to determine the viscosity of the ice. The dependence of the moduli of deformation on temperature, pressure, period of load and size of crystals is studied and approximate formulas are given for calculation. An example is given to show methods of calculating ice pressure at a given air temperature from the moduli of deformation obtained.

Des prismes de glace de dimensions 7 x 7 x 20 cm ont été soumis à des essais de compression monoaxiale. L'élasticité et la viscosité de la glace ont été déterminées en utilisant la relation entre pression, déformation et temps. Un échantillon de glace de diamètre 80 cm et d'épaisseur 7 cm a été soumis à des essais de compression biaxiale à température croissante et expansion restreinte. La viscosité de la glace a été déterminée en utilisant la relation entre pression et vitesse de déformation. La fonction liant module de déformation à température, pression, période de charge et taille des cristaux a été étudiée et des formules approchées sont proposées pour les calculs. Les méthodes de calcul de la pression de glace à partir du module de déformation pour une température donnée sont démontrées sur un exemple.

SYMBOLS

A	=	Area in cm^2
a	=	$\frac{\lambda}{c \cdot \gamma}$ = Thermal diffusivity in m^2/h
α	=	Heat transfer in $kcal/m^2 . h\,^oC$
c	=	Specific heat in $kcal/kg\,^oC$
γ	=	Density in kg/cm^3
P	=	Tractive force in kp
d	=	Thickness in cm
E_1	=	Modulus of elasticity in kp/cm^2 for instantaneous elastic deformation
E_2	=	Modulus of elasticity in kp/cm^2 for retarded elastic deformation
ϵ	=	Relative deformation
η_2	=	Modulus of viscosity in $kp . s/cm^2$ for damping of retarded elastic deformation
η_3	=	Modulus of viscosity in $kp . s/cm^2$
ϑ	=	Temperature in ^oC
k_n	=	Coefficients (n=1, 2, ...), also as indeces
λ	=	Coefficient of thermal conductivity in $kcal/mh\,^oC$
n	=	Index
ν	=	Coefficient of contraction
σ	=	Stress, ice pressure in kp/cm^2
t	=	Time in h for heat calculations, time in s for deformation calculations

1. THEORETICAL ANALYSIS OF THE DISTRIBUTION OF TEMPERATURE IN AN ICE SHEET

The temperature of the ice is governed by several factors, such as air temperature, speed of the wind, precipitation, insolation, thickness of the ice sheet, physical properties of the snow, thickness of the ice and physical properties of the ice.

The thickness and properties of the ice sheet affect the result of changes in air temperature on the upper surface of the ice. If the ice sheet is thick, a considerable rise in temperature would cause only a slight increase in the temperature of the upper surface of the ice. This increase in temperature is, moreover, displaced in time .

The distribution of temperature in the ice is governed by the change of temperature in the upper surface of the ice and by the thickness and physical properties of the ice. The temperature of the lower surface of the ice is hereafter assumed to be $0^{\circ}C$.

The distribution of temperature in an ice sheet with a constant surface temperature, taking into account the effect of latent heat of ice, can be studied with the help of the following equation given by JANSON, 1964.

$$\vartheta_x = \vartheta_o + (\vartheta_i - \vartheta_o)\,\frac{G\left(\frac{x}{\sqrt{4 a_1 t}}\right)}{G\left(\frac{-m}{\sqrt{4 a_1}}\right)} \qquad (1)$$

ϑ_x = temperature x m under the upper surface of the ice

ϑ_o =temperature of the upper surface of the ice

ϑ_i = temperature of the lower surface of the ice

$$G(f) = \frac{2}{\sqrt{\pi}}\left(f - \frac{1}{1!\cdot 3} f^3 + \dots\right), \qquad \text{GAUSS' error integral}$$

x = distance from the upper surface of the ice in m.

t = time in h from the starting of ice formation

$a_1 = \frac{\lambda}{c\,\gamma}$ = thermal diffusivity of ice in m^2/h

m = a factor which, in ice, is governed by the temperature of the upper surface of the ice.

If this equation is closely studied, one finds that deviation from the linear temperature distribution is very slight.

Temperature distribution in ice at an arbitrary change in air temperature can be appropriately calculated with a difference method. The calculation may be carried out numerically or graphically. Numerical calculation can be carried out by means of ADP.

The progress of temperature in a body with one-dimensional heat flow can be calculated by a difference method according to SCHMIDT, 1924, and PLANK, 1959.

As an example of the application of this difference method the change in temperature is calculated in the following 3 devised alternative cases.

The air temperature in all three cases is the temperature recorded at Arjeplog on 17/1 - 18/1, 1963, Fig. 1. The rise in temperature on 17/1 - - 18/1, 1963, from $-34^{\circ}C$ to $+2^{\circ}C$ is the largest recorded at Arjeplog in 1963 but cannot be said to be exceptional. This temperature increase is only used for the calculations.

The calculations are carried out numerically. From information collected from lakes, near Lake Hornavan, the thickness of the ice at Hornavan, near Arjeplog, on that date is estimated to have been about 60 cm.

The following ice sheet and types of weather are assumed for the three cases in question:-

	Alt. 1	Alt. 2	Alt. 3
Thickness of snow in m	0	0.05	0.40
Type of snow	-	newly fallen snow	granular snow
γ snow kg/cm^3	-	$0.10 \cdot 10^3$	$0.25 \cdot 10^3$
λ snow kcal/m.h$^\circ$C	-	0.04	0.12
Wind force m/s	16	16	0
α kcal/m^{2}h$^\circ$C	42	42	5

In the example it is assumed that heat flow on the 17/1 at 0.00 h was stationary.

In Fig. 2 the distribution of temperature in Alt. 1 at 12-hour intervals is shown. The temperature curves show that the change in air temperature is very quickly absorbed by the ice and, between the 24th and 36th hour, causes a temperature change of about 10°C in the ice, 0.1 metres under the surface.

Fig. 3 shows Alt. 2. The 0.05 m thick layer of snow heavily damps the effect of an increase in temperature on the ice. The temperature of the ice 0.1 m under the surface of the ice changes in this case only about 0.5°C between the 24th and 36th hour.

Fig. 4 shows Alt. 3. In this alternative the effect of the ice sheet is very great. The temperature of the upper surface of the ice changes only about 0.8°C during the 48 hours of the experiment.

2. DEFORMATION PROPERTIES OF ICE

The following rheological equation was used to evaluate experimental studies:-

$$\varepsilon = \frac{\sigma}{E_1} + \frac{\sigma}{E_2}\left(1 - e^{-\frac{E_2 \cdot t}{\eta_2}}\right) + \frac{\sigma \cdot t}{\eta_3} \qquad (2)$$

In this equation the material is called visco-elastic material. When the relative deformation of a material is linear because of the stress, the material is called linear visco-elastic material. Fig. 5 shows a time-deformation diagram for a visco-elastic material. The diagram shows deformation when a load is applied and when the load is removed after a specified time.

Several studies have shown that equation (2) does not give a complete picture of the deformation characteristics of ice. Ice is therefore not linearly visco-elastic. This means that ice is difficult to deal with in simple rheological equations. However, a simple form in accordance with equation (2) is used in the following. As ice is not linearly visco-elastic the moduli of deformation are therefore governed by stress.

3. EXPERIMENTAL INVESTIGATION - SINGLE-AXIS TESTS

To determine the relationship between deformation and time at constant compression stress 7 x 7 x 20 cm ice prisms were used. The ice prisms were subjected to load in the longitudinal direction. This direction was parallel to the surface of the ice. Deformation in the direction of the pressure was measured in a mechanical extensometer as shown in Fig. 6 and 7. Sixty ice prisms were used.

A deformation-time diagram for some of the tests is given in Fig. 8.

The different crystalline structures are shown in Fig. 9 and 10.

An interesting gliding surface on one of the tests is shown in Fig. 11.

On the basis of the deformation equation assumed, the moduli of deformation involved, E_1, E_2, η_2 and η_3, were studied with regard to the effect of different factors. Only a few tests were carried out and it is therefore only possible to show the general effect of different factors.

An evaluation of the tests has given the following approximate values of the moduli of deformation

$$E_1 = 66{,}000 - 800 \cdot \vartheta \text{ kp/cm}^2 \tag{3}$$

The relationship applies to $\vartheta = -0.5^\circ C - -20^\circ C$

E_1 seems to be independent of stress (2-14 kg/cm^2) and previous load. It was not possible to show the effect of the size of crystals. The modulus of elasticity E_1 was determined on the basis of the deformation occuring

30 s after the load was applied. If the effect of retarded elastic deformation during 30 s is eliminated, the value of E_1 would be about 3 per cent higher.

$$E_2 \sim 70,000 \text{ kp/cm}^2$$

The effect of stress, temperature, previous load and size of crystals could not be determined, as it is difficult to determine accurately the values of E_2

$$\eta_2 \sim 1.1 \cdot 10^8 \text{ kp.s/cm}^2$$

The modulus of viscosity η_2 is determined with very little accuracy.

$$\eta_3 = 18.5 \cdot \sigma^{-1} (0.2 - 0.08\vartheta) \cdot \left(\frac{t}{3600}\right)^{0.5} \cdot 10^8 \text{ kp}\cdot\text{s/cm}^2 \qquad (4)$$

The above relationship applies to a range of temperature between -0.5°C - -20°C, stress between 2-14 kp/cm² and load period 3,600-150,000 s. For temperatures, stress and loads outside these ranges the relationship should be used carefully.

The given values of the modulus of viscosity η_3 are applicable only to normal crystals such as those used for the experiments. The value of η_3 is considerably lower when the crystals are small and shows tendencies to decrease with the period of load. This can probably be explained as a result of gliding surfaces occuring when small crystals were used during the experiments. These surfaces had an accelerating effect on deformation.

4. EXPERIMENTAL INVESTIGATION DOUBLE-AXIS TESTS

Natural ice sheets have horizontal stretches which are large in comparison to their thickness. This means that stress caused by ice pressure becomes double-axled.

The following experiments were carried out with the intention of studying the size of the moduli of deformation at double-axled pressure and equal pressure in all directions.

A steel ring with an inner diameter of 80 cm was placed around a circular ice plate with a diameter of about 80 cm and a thickness of about 7 cm. Thereafter, the space in between was filled with water which froze, see Fig. 12. When the ice plate with the steel ring was subjected to rising temperature, pressure arose in the ice which, because of the higher length expansion coefficient of ice, caused stress in the steel ring. The stress was measured with a wire strain gauge. In this way the ice pressure and deformation characteristics of the ice at given temperature rises could be studied.

The experiments were started at a low temperature. The temperature of the ice and the steel ring increased relatively rapidly. The stress in the ring and, consequently, ice pressure increased very swiftly. The rise in the temperature of the ice was then regulated so that the relative deformation of the ice, caused by the restrictive effect of the steel ring, became linear with time. Fig. 13 shows the temperature of the ice and ice pressure during Experiment No. 1.

When the temperature is increased the ice expands

$$\varepsilon_{is} = \Delta\vartheta_{is} \cdot \alpha_{is} \qquad (5)$$

Equation (5) shows the expansion of the ice when movement is free. Movement is, however, restricted by the steel ring.

The steel ring expands when the temperature rises and when stress occurs.

$$\varepsilon_{steel} = \Delta\vartheta_{steel} \cdot \alpha_{steel} + \varepsilon_{\Delta P} \tag{6}$$

$\varepsilon_{\Delta P}$ is the relative change in the length of the steel ring caused by increased stress in the ring occuring when the ice is pressed against the ring.

$$\varepsilon_{\Delta P} = \frac{\Delta P_{steel}}{A_{steel} \cdot E_{steel}} \tag{7}$$

When the temperature is raised the ice tries to expand more than the steel ring.

In other words, because of the sustaining effect of the ring the ice must become deformed

$$\varepsilon_{is_{def}} = \varepsilon_{is} - \varepsilon_{steel} \tag{8}$$

During the experiments the temperature was raised so that $\varepsilon_{is_{def}}$ became linear with time. In this way the different values of the speed of relative deformation, ranging from about $10 \cdot 10^{-6}$/h to $100 \cdot 10^{-6}$/h, were examined. These deformation speeds correspond to temperature rise speeds of between 0.2 and 2.0° C/h when temperature expansion is completely restricted.

Curves showing the temperature of the ice, its relative deformation ($\varepsilon_{is_{def}}$) and ice pressure have been drawn up. In these experiments ice pressure was kept constant in all directions at one level. Some of the experiments are shown in Fig. 13-15.

The curves show that at the start of an experiment the pressure of ice increases very rapidly. This rapid increase in temperature occurs because at the beginning the deformation of ice is mainly elastic and delayed elastic while viscous deformation is comparatively small.

Gradually, the speed of the rise in pressure decreases. The reason for this is that the deformation of ice when pressure and temperature increase is more and more of the viscous type. A longer period of load results in decreased viscous deformation and this has the opposite effect, though it is of less importance.

Gradually, a maximum value of ice pressure is reached. At the maximum value, the following equation is true as ice pressure has been assumed to be equal in all directions on that level

$$\frac{\Delta \varepsilon_{is_{def}}}{\Delta t} = \frac{\sigma}{\eta_3}(1 - \nu_v) \qquad (9)$$

ν_v is the coefficient of contraction for viscous deformation.

At maximum point η_3 can thus be calculated when the speed of deformation and the ice pressure are known.

From the rheological equation for ice at single-axis pressure

$$\varepsilon = \frac{\sigma}{E_1} + \frac{\sigma}{E_2}\left(1 - e^{-\frac{E_2 \cdot t}{\eta_2}}\right) + \frac{\sigma \cdot t}{\eta_3} \qquad (10)$$

the moduli of deformation have been determined by double-axis experiments.

E_1 and E_2 have only been studied very generally during double-axis experiments. The experiments show that the values obtained in single-axis experiments are in general applicable to double-axis experiments on condition that ν is assumed to be approximately 0.36 during elastic deformation.

$$E_1 \sim 66{,}000 - 800\,\vartheta \text{ kp/cm}^2$$

$$E_2 \sim 70{,}000 \text{ kp/cm}^2$$

$$\eta_2 \sim 1.1 \cdot 10^8 \text{ kp}\cdot\text{s/cm}^2$$

The modulus of viscosity η_3 has, on the other hand, been studied rather carefully. If the coefficient of contraction ν_v is assumed to be 0.5 when the deformation is viscous, the following is obtained.

$$\eta_3 = 31 \cdot \sigma^{-1}(0.30 - 0.07\,\vartheta) \cdot \left(\frac{t}{3600}\right)^{0.25} \cdot 10^8 \text{kp}\cdot\text{s/cm}^2 \qquad (11)$$

The above relationship is applicable to a temperature range between -0.5°C and -20°C, stress between 5-16 kp/cm² and load period between 7,200 - 72,000 s.

At temperatures, stress and loads outside these ranges the equations must be used carefully.

Fig. 16 shows η_3 as a function of ice pressure.

The effect of smaller crystals has not been studied.

The modulus of viscosity η_3 is thus considerably lower in single-axis experiments than in double-axis experiments. One of the reasons for this is that the average of the moduli of viscosity in single-axis experiments was affected by several experiments with a low modulus of viscosity, probably caused by the occurrence of gliding surfaces in the ice prisms.

5. COMPARISON WITH PREVIOUS ICE PRESSURE DETERMINATIONS

From the basis of the moduli of deformation reported the ice pressure-time-diagram at a constant speed of increase in temperature has been calculated. The calculations were carried out with difference calculus applied to double-axled stress with temperature increases of 0.1, 0.2, 0.5, 1.0 and 2.0 $^{\circ}$C/h. The starting temperature was -5 $^{\circ}$C, -10 $^{\circ}$C and -20 $^{\circ}$C. The diagrams are shown in Fig. 17-19. The 0.1 $^{\circ}$C/h increase in temperature is lower than thouse used in the experiments and the corresponding curve is therefore uncertain. The above calculations from the bases of a diagram which shows the maximum ice pressure as a function of the speed of raised temperature. The diagram shown in Fig. 20 deals with double-axled stress when expansion is restricted in two directions.

From the ice pressure - temperature diagram given by LÖFQUIST some correlated values of max. ice pressure and estimated speed of temperature rise have been calculated and reported in Fig. 20. According to LÖFQUIST, given ice pressure value should be increased by 25 per cent in order to correspond to the effect of the simultaneous expansion of the surrounding concrete barrel. The values given by LÖFQUIST have therefore been corrected accordingly.

Fig. 20 shows that the estimated maximum pressure is somewhat higher than the results obtained by LÖFQUIST in experiments with corresponding double-axled stress. This may be explained by the possible occurence of open cracks or tension in the ice during LÖFQUIST´s experiments.

A comparison with preceding experiments reported by MONFORE with single-axis stress show similar results. These are shown in Fig. 21.

6. EXAMPLES OF CALCULATION AND COMMENTS

The previous paragraphs give calculations of the change in the temperature of the ice in three alternative cases under different conditions. It would be of interest to calculate maximum ice pressure during these changes of temperature. Ice pressure can be determined by difference calculus. Moduli of deformation as stated above can be used when expansion is restricted in two directions. Calculations of this type are, however, very extensive and should be processed by means of ADP.

The moduli of deformation obtained during the experiments are spread over a wide range. This means that calculations of the values of maximum ice pressure are somewhat unreliable. With this in mind, rough estimates can be used to assess maximum ice pressure. With the help of Fig. 16--18 an approximate value of the maximum ice pressure can be calculated. It is assumed that expansion is restricted in two directions, that ice pressure is 0 at 0.00 h, and that the ice is not cracked.

The ice sheet is divided into layers, and pressure at different times is calculated for each layer, depending on temperature changes in the layer in question. The sum of ice pressure in the various layers is calculated following which maximum ice pressure can be obtained. The maximum ice pressure when conditions are those applicable to Alt. 1 is then $0.6 \cdot 77 = 46$ Mp/m.

In the same way ice pressure in Alt. 2 and 3 is 12 Mp/m and 5 Mp/m respectively.

Although a snow cover on the ice has a reducing effect on the maximum ice pressure one can conclude from the above that the substantial reduction in temperature which occurs when the ice is covered with snow does not cause a corresponding rate of reduction of ice pressure. This can also be seen in Fig. 20, where it is shown that max. ice pressure acquires a value which, approximately, is a function of the root of the speed of temperature rises.

In the above calculations of ice pressure a number of approximations and assumptions have been made. The approximative method mentioned can only give basic values for ice pressure under ideal conditions. Actual conditions in nature must, of course, be taken into account.

Field research on ice pressure and ice temperature and determination of exceptional increases in temperature and simultaneous snow sheets must therefore be carried out. Field research, together with laboratory experiments of the types described above, should provide the bases for determining the ice pressure to be taken into consideration for dimensioning hydraulic construction works.

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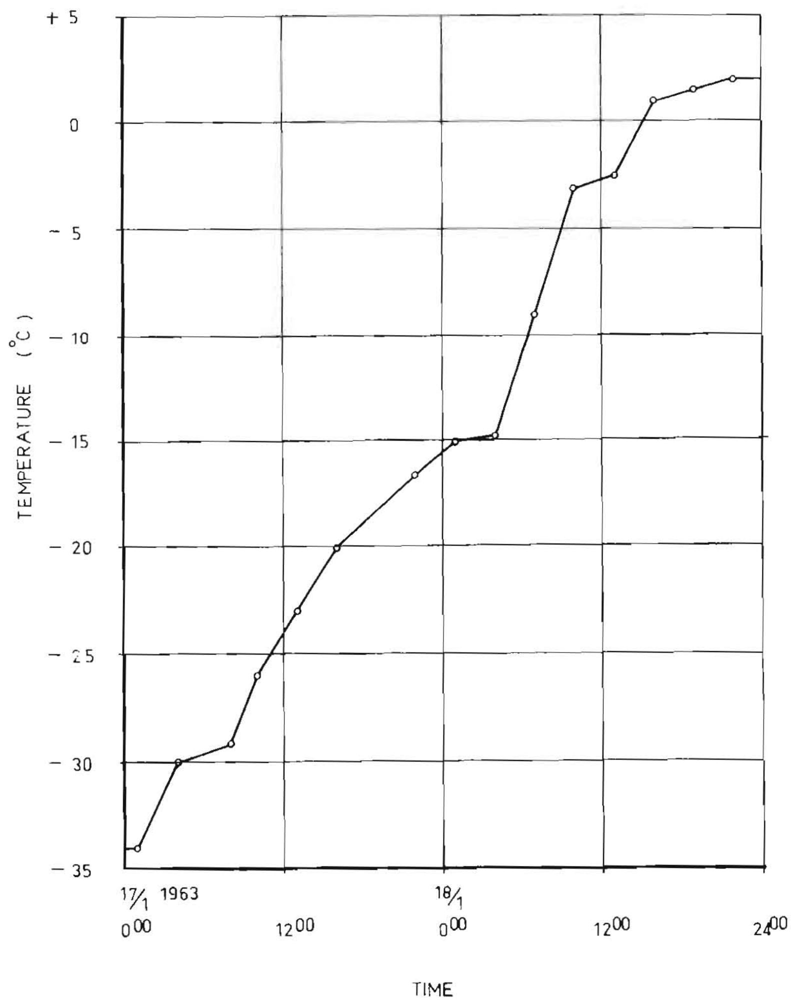
AIR TEMPERATURE IN ARJEPLOG
THE 17 TH AND 18 TH JANUARY, 1963

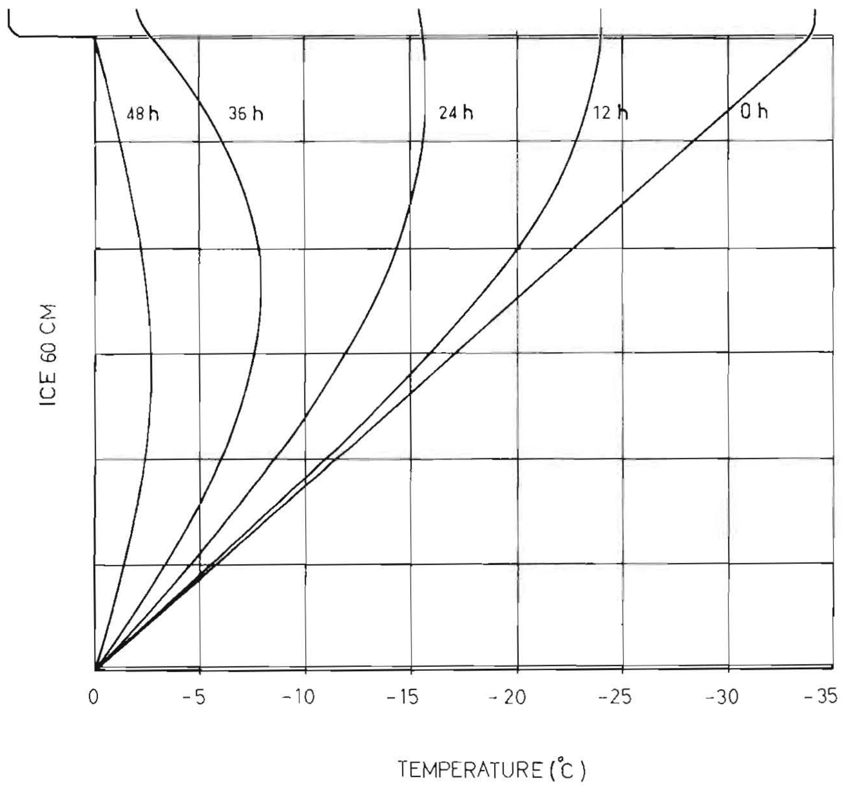
FIG. 1



6.7

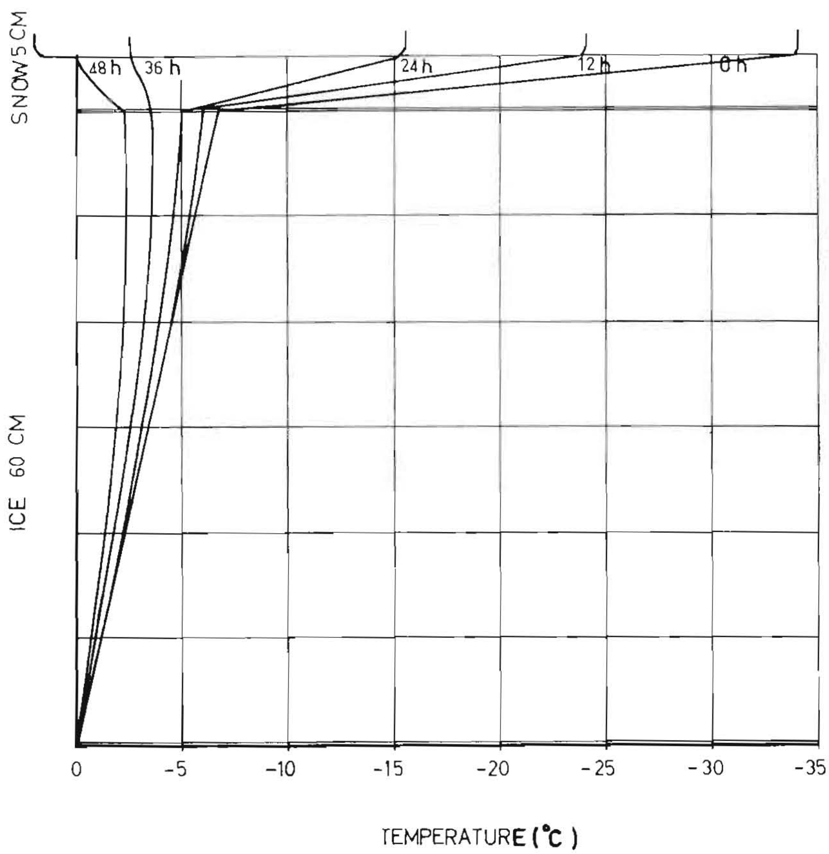
DISTRIBUTION OF TEMPERATURE
ALTERNATIVE 1

FIG. 2.



DISTRIBUTION OF TEMPERATURE
ALTERNATIVE 2

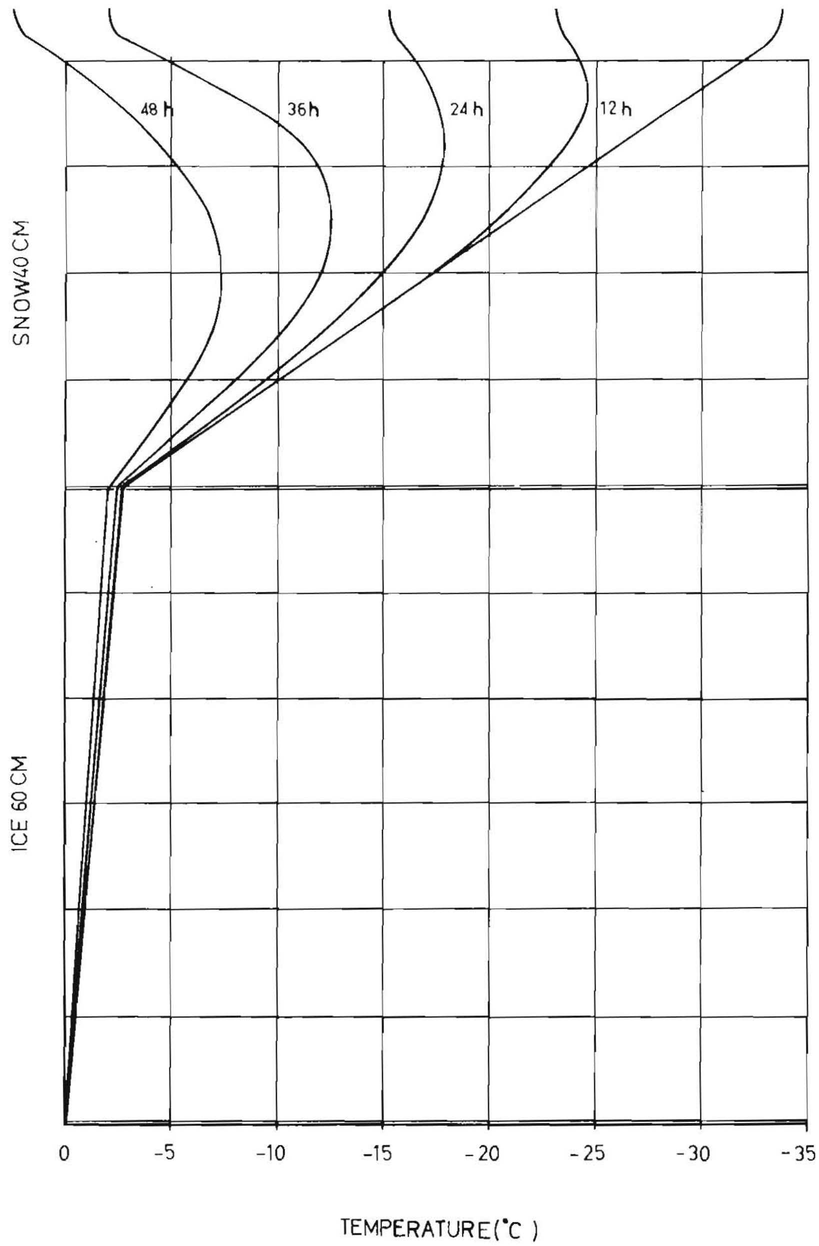
FIG. 3



6.7

DISTRIBUTION OF TEMPERATURE
ALTERNATIVE 3

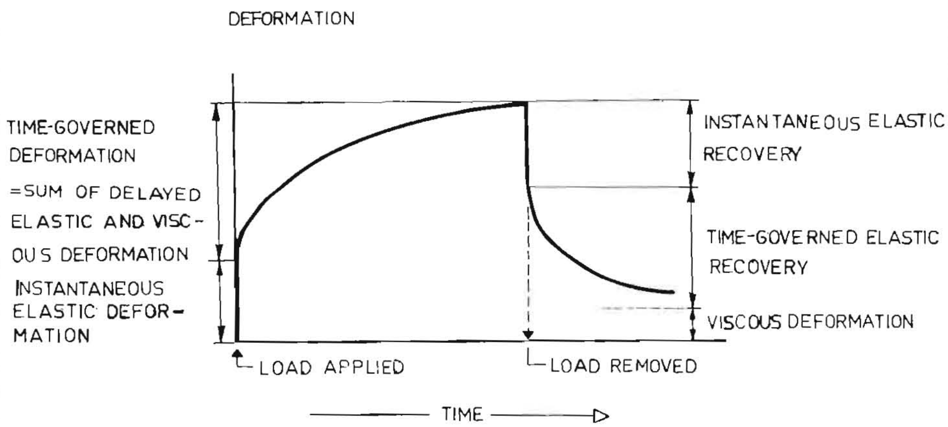
FIG. 4.



6.7

TIME DEFORMATION DIAGRAM FOR ICE

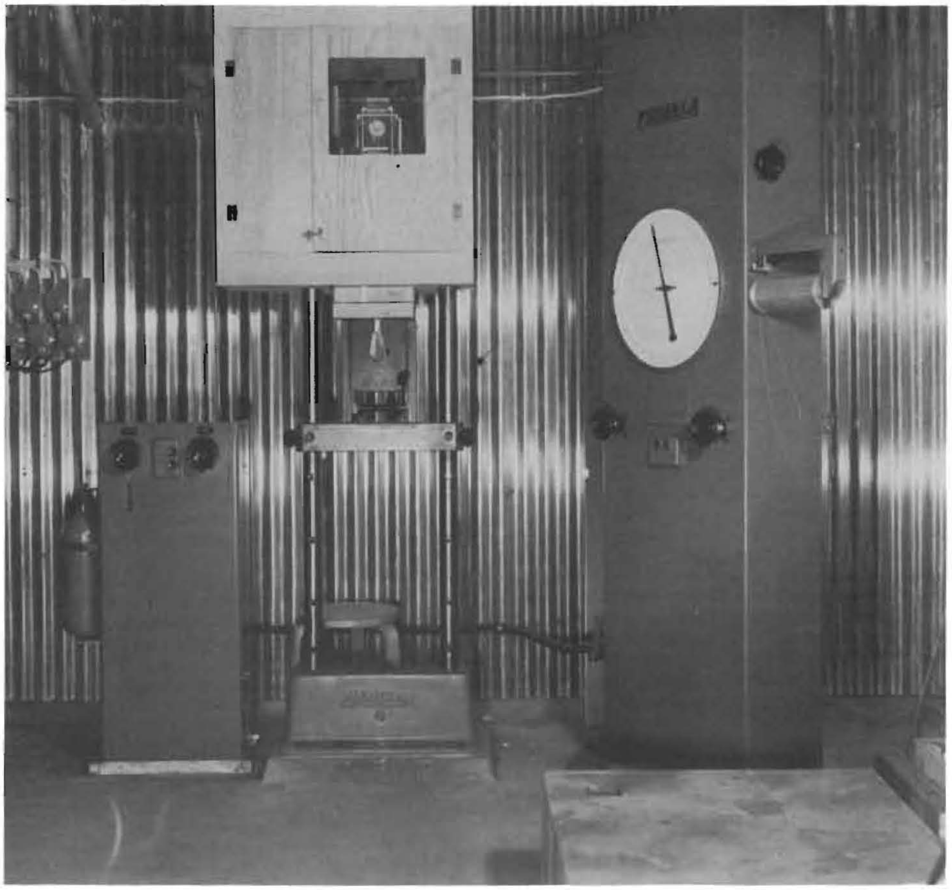
FIG. 5.



6.7

SINGLE-AXIS TEST
PRESSURE TESTING APPARATUS

Fig. 6

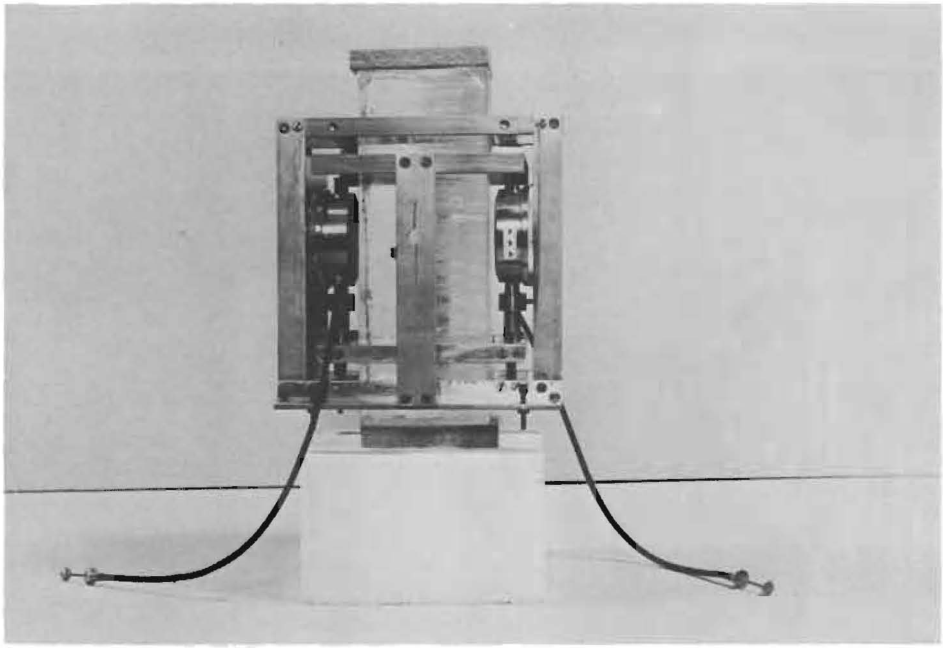


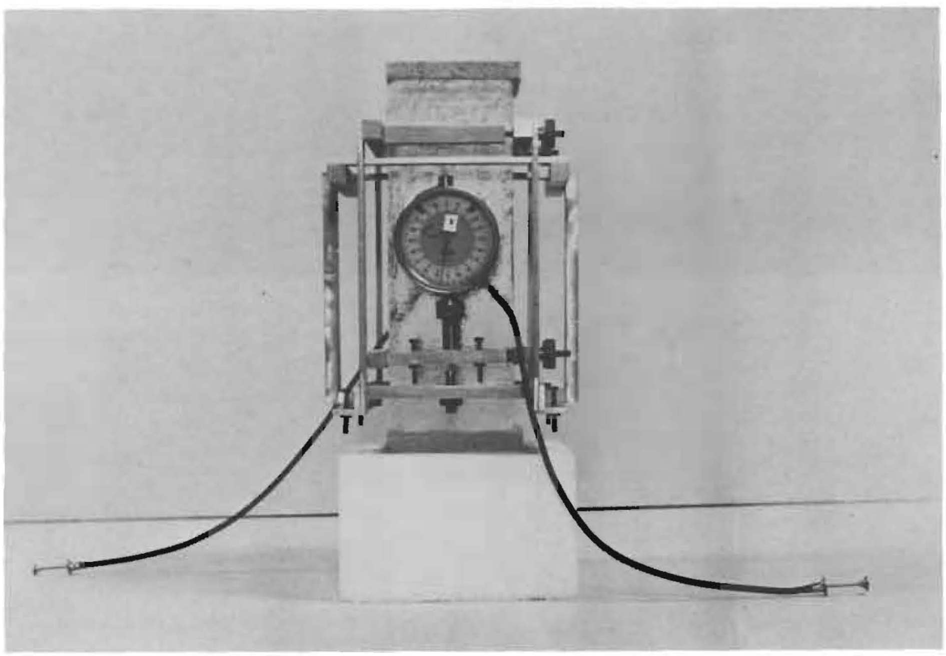
6.7

SINGLE-AXIS TEST

MECHANICAL EXTENSOMETER

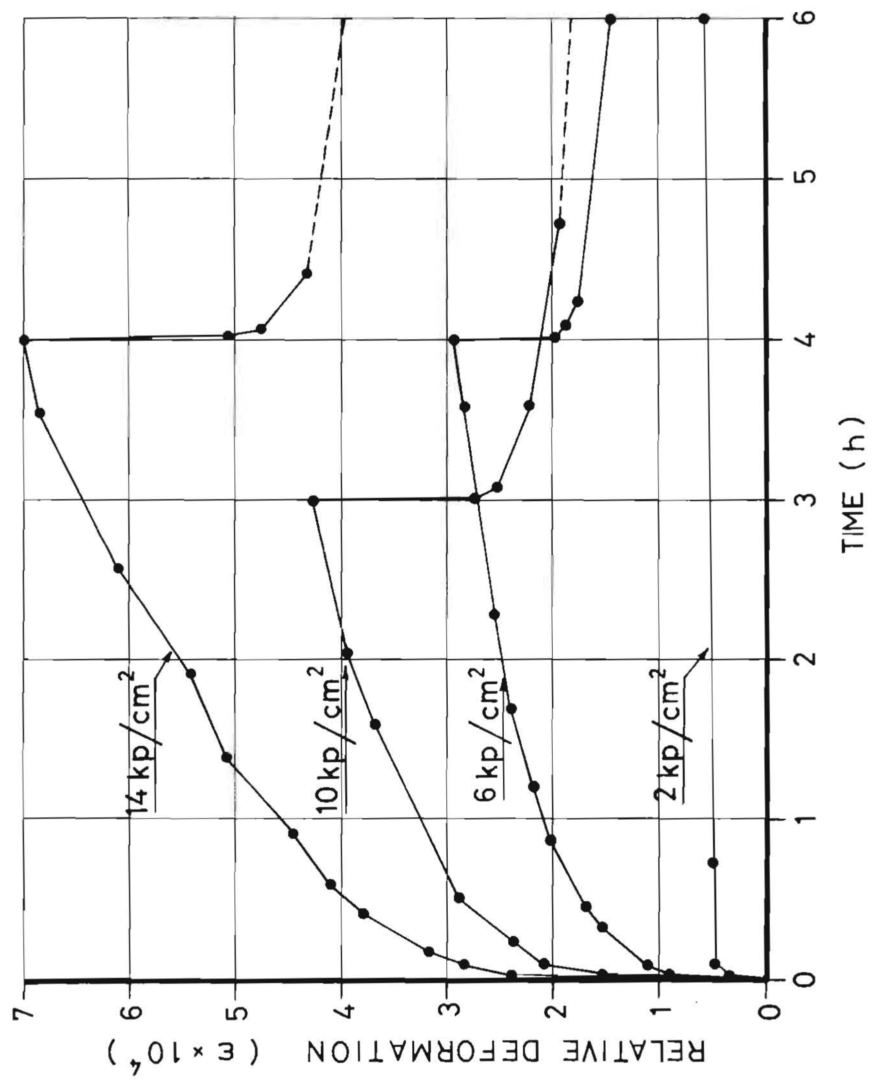
Fig. 7





SINGLE-AXIS TEST
DEFORMATION — TIME-DIAGRAM
TEST 6

Fig 8.

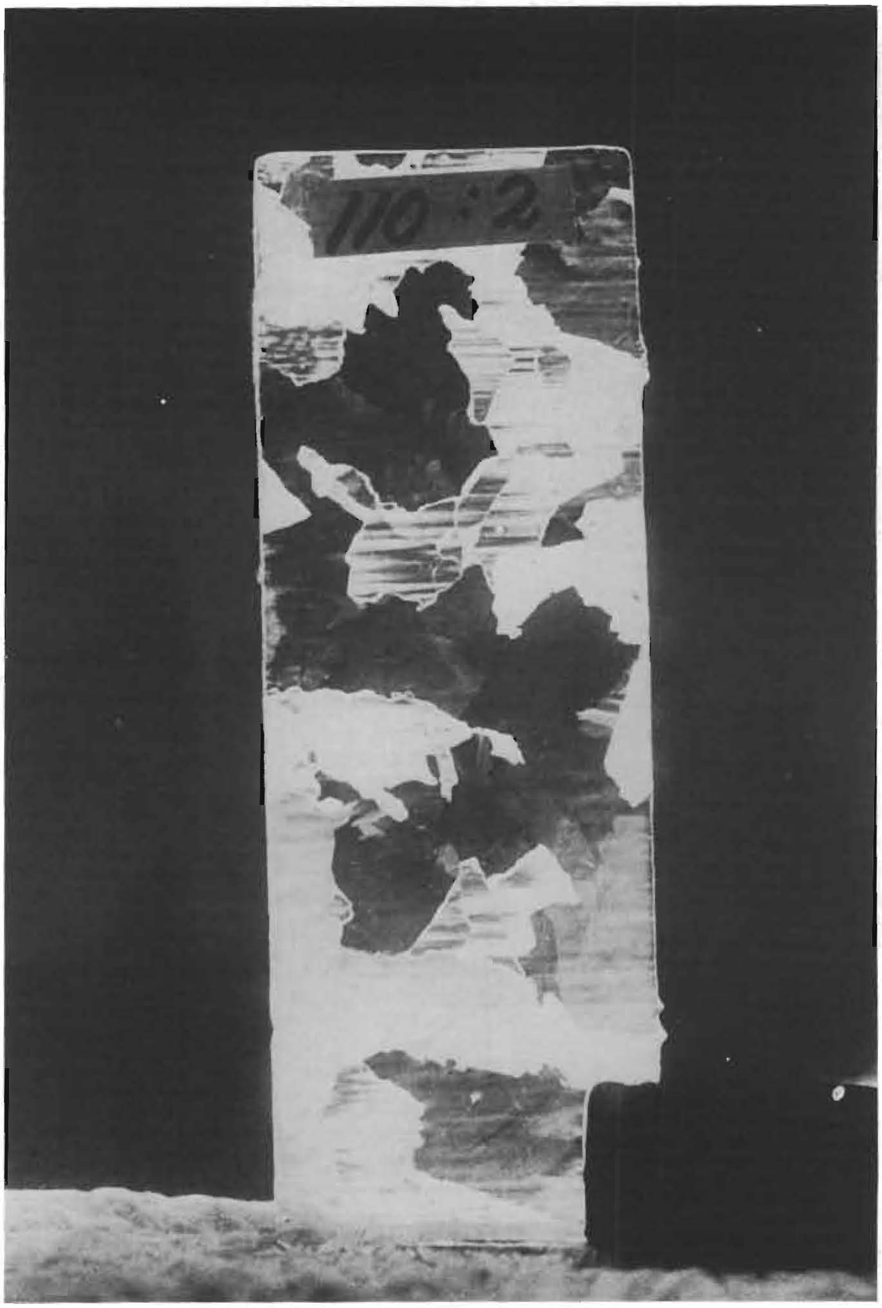


6.7

SINGLE-AXIS TEST

NORMAL CRYSTALS

Fig 9



6.7

SINGLE AXIS TEST

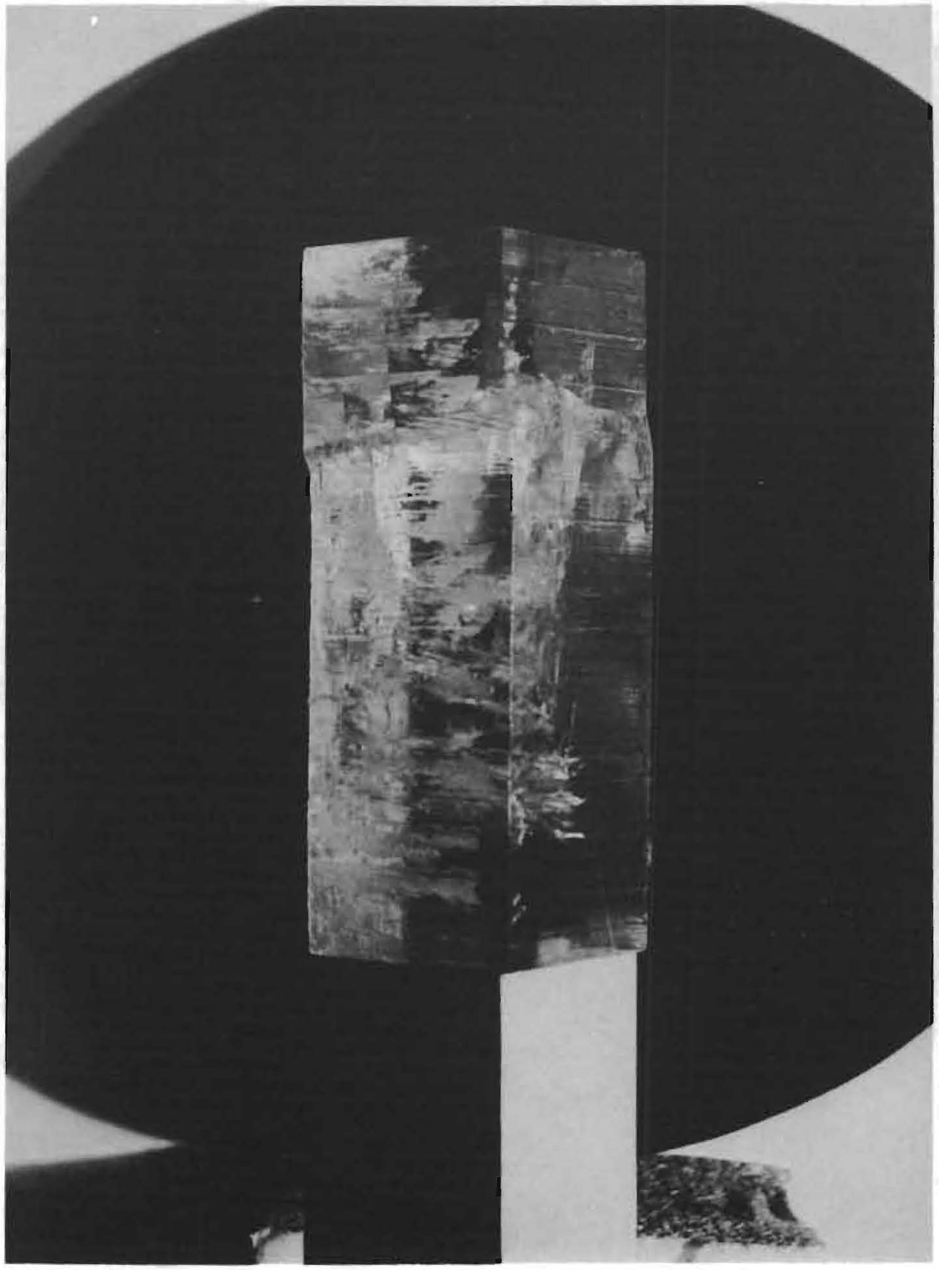
SMALL CRYSTALS

Fig. 10

6.7

SINGLE AXIS TEST
GLIDING SURFACE
WITH ENLARGED DEFORMATION

Fig. 11

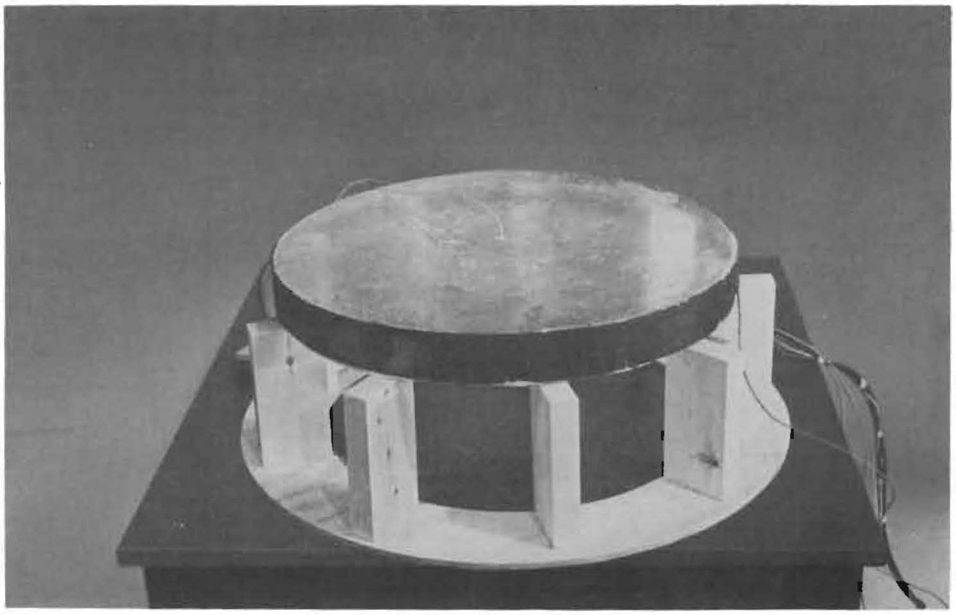


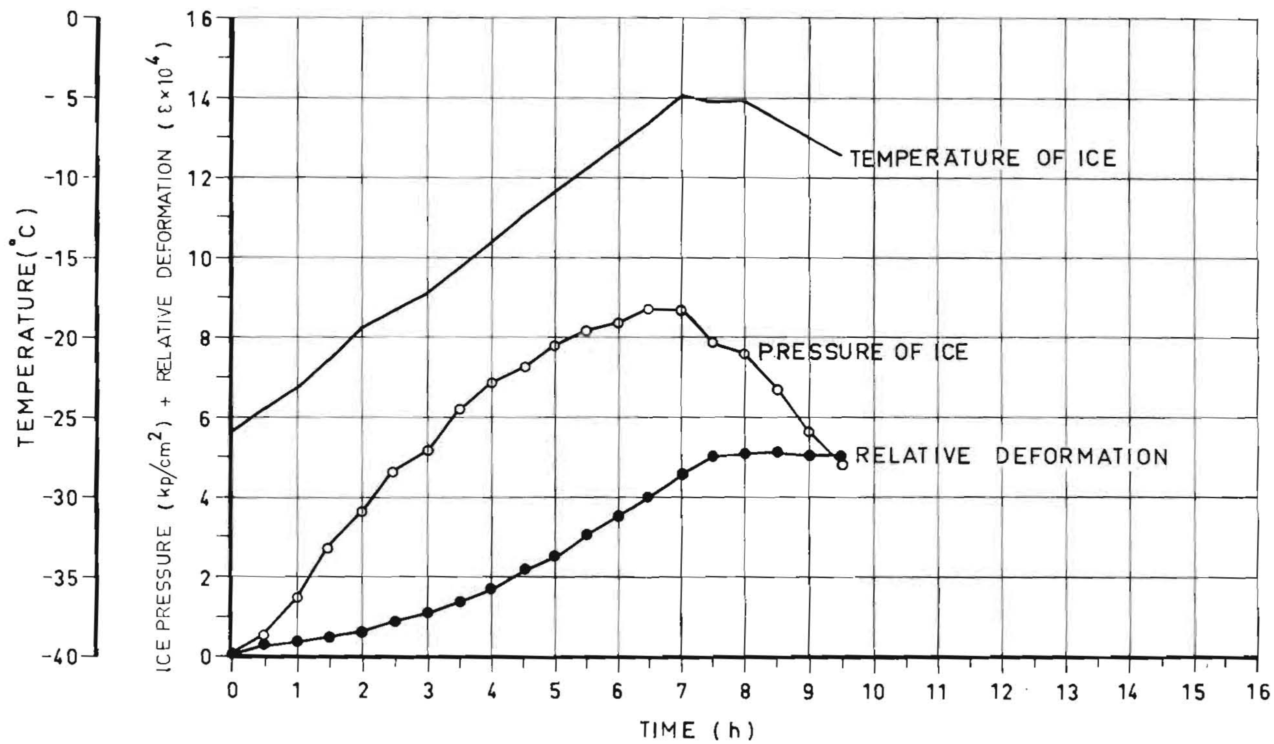
6.7

DOUBLE-AXIS TEST

Fig.12

ICE PLATE WITH STEEL RING

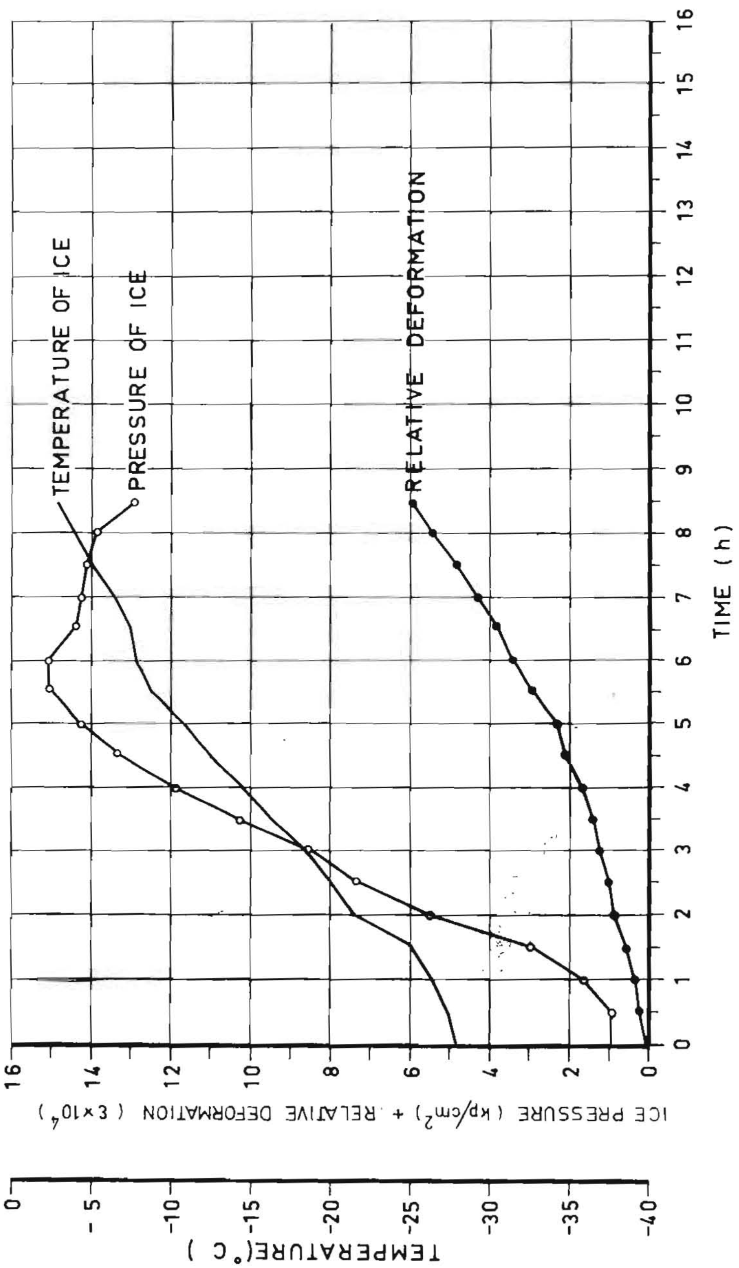




DOUBLE-AXIS TEST
PRESSURE-DEFORMATION-TIME-DIAGRAM
TEST 1

Fig. 13

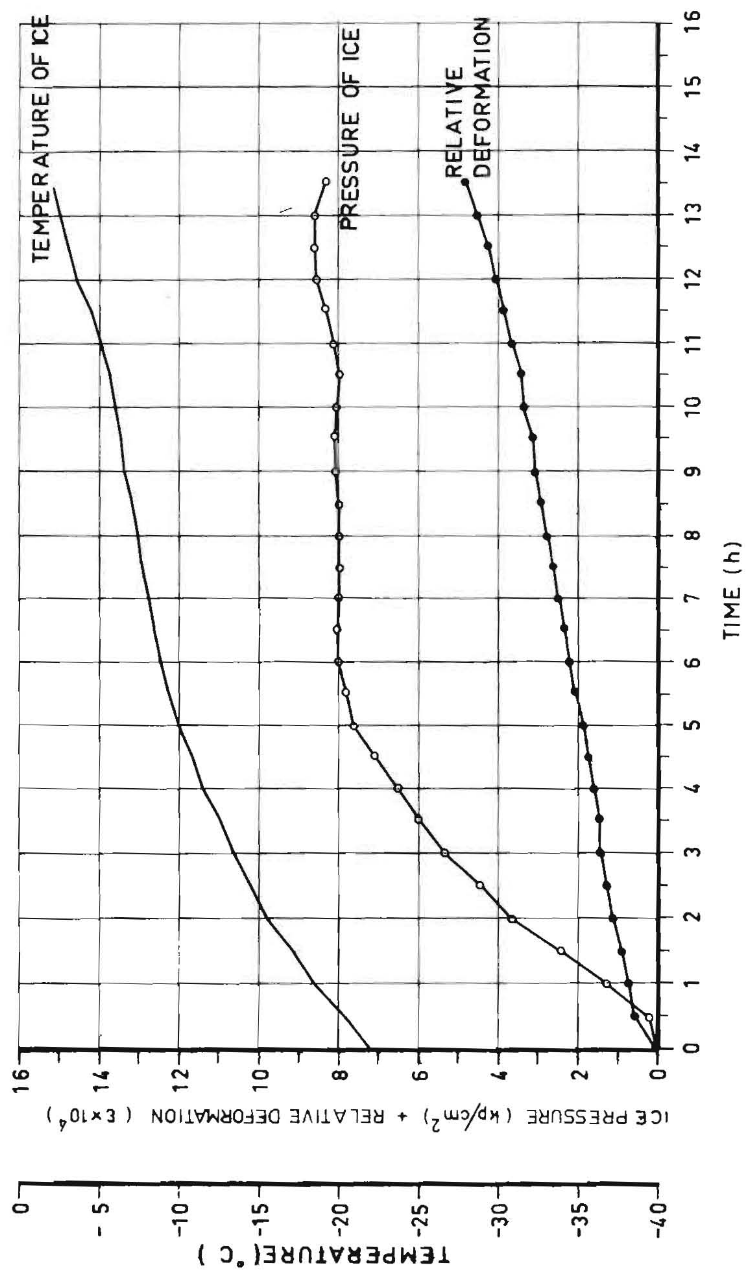
	DOUBLE-AXIS TEST PRESSURE-DEFORMATION-TIME-DIAGRAM TEST 3	Fig. 14.



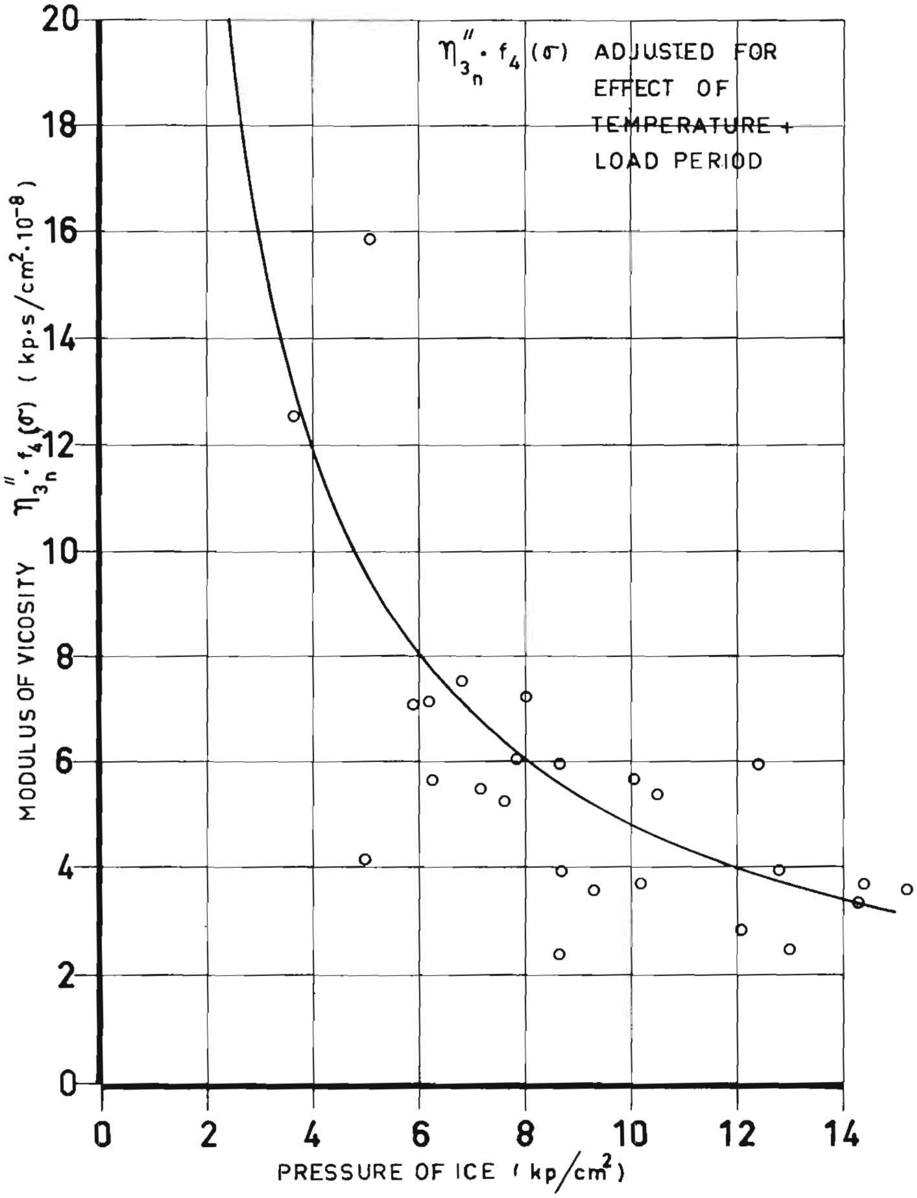
6.7

DOUBLE-AXIS TEST
PRESSURE-DEFORMATION-TIME-DIAGRAM
TEST 7

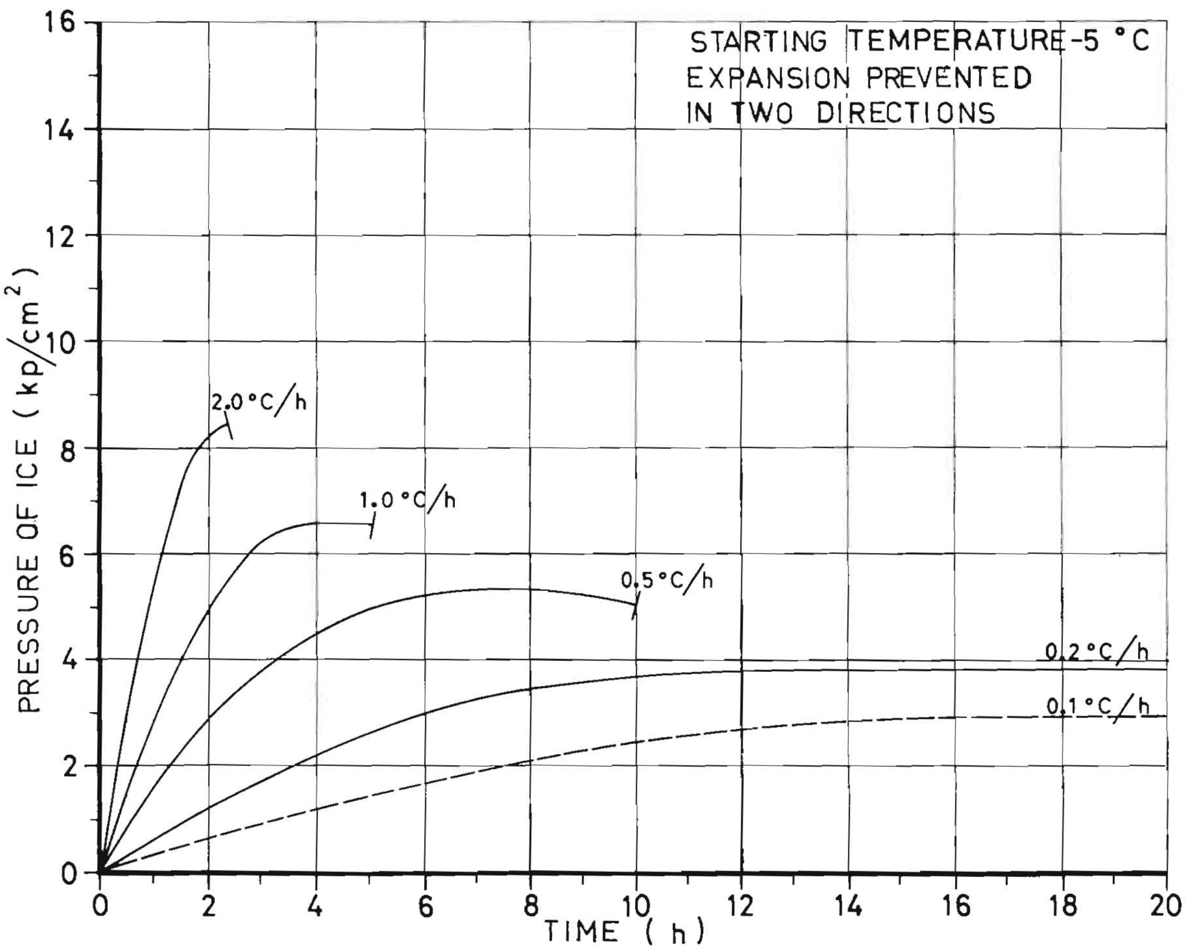
Fig.15



6.7

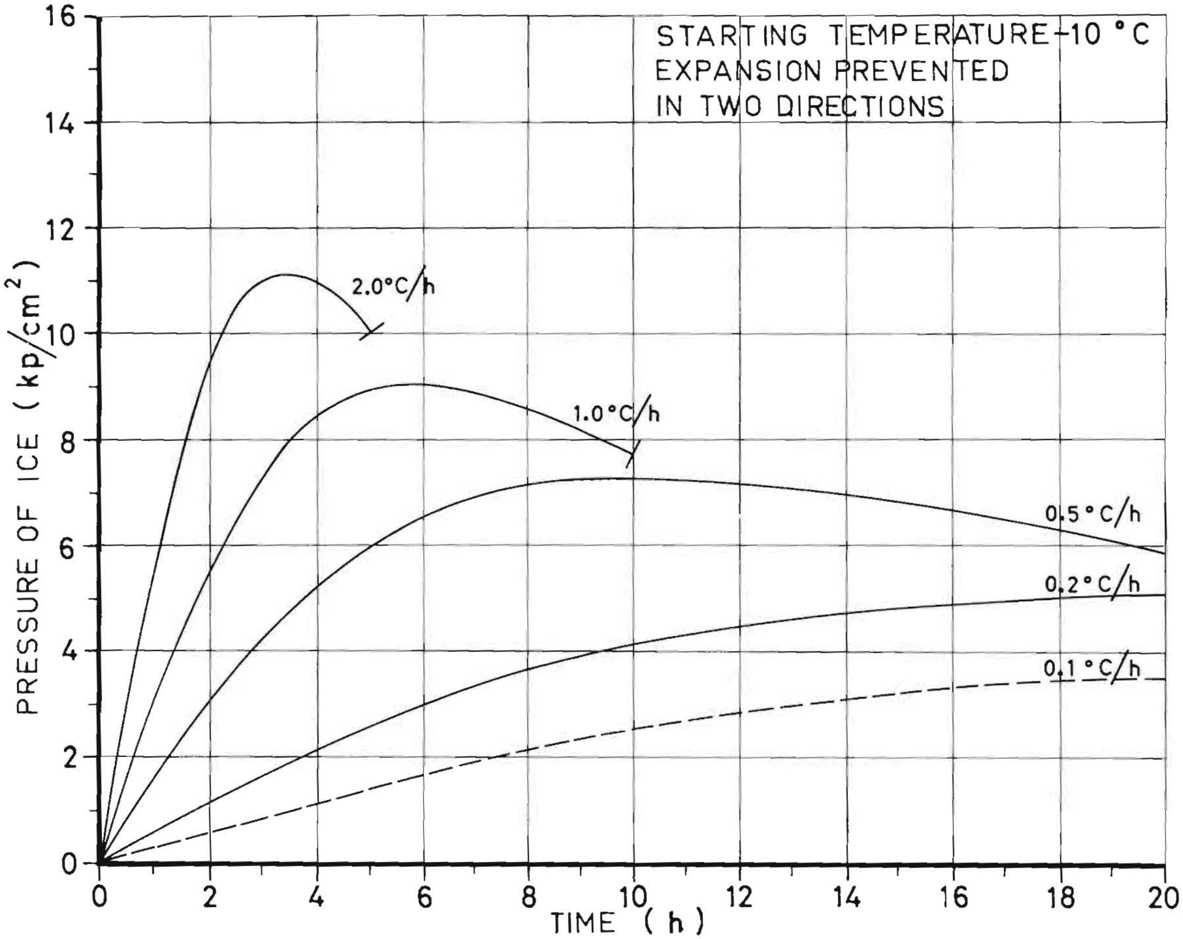

$\eta''_{3_n} \cdot f_4(\sigma)$ ADJUSTED FOR EFFECT OF TEMPERATURE + LOAD PERIOD
MODULUS OF VICOSITY $\eta''_{3_n} \cdot f_4(\sigma)$ (kp·s/cm^2·10^{-8})
PRESSURE OF ICE (kp/cm^2)
20
18
16
14
12
10
8
6
4
2
0
0
2
4
6
8
10
12
14

6.7

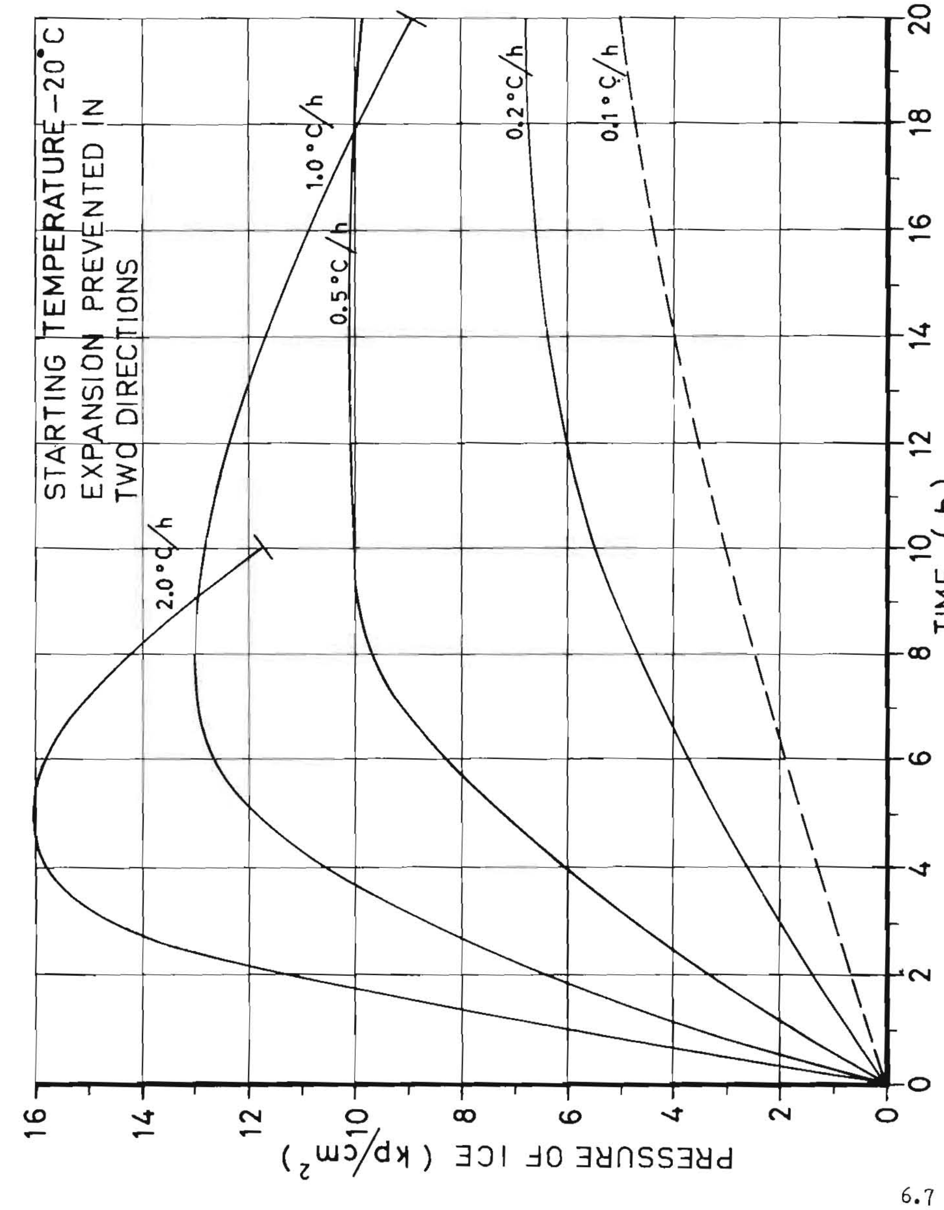

STARTING TEMPERATURE -5 °C
EXPANSION PREVENTED
IN TWO DIRECTIONS
PRESSURE OF ICE (kp/cm2)
TIME (h)
2.0 °C/h
1.0 °C/h
0.5 °C/h
0.2 °C/h
0.1 °C/h
0
2
4
6
8
10
12
14
16
18
20

6.7

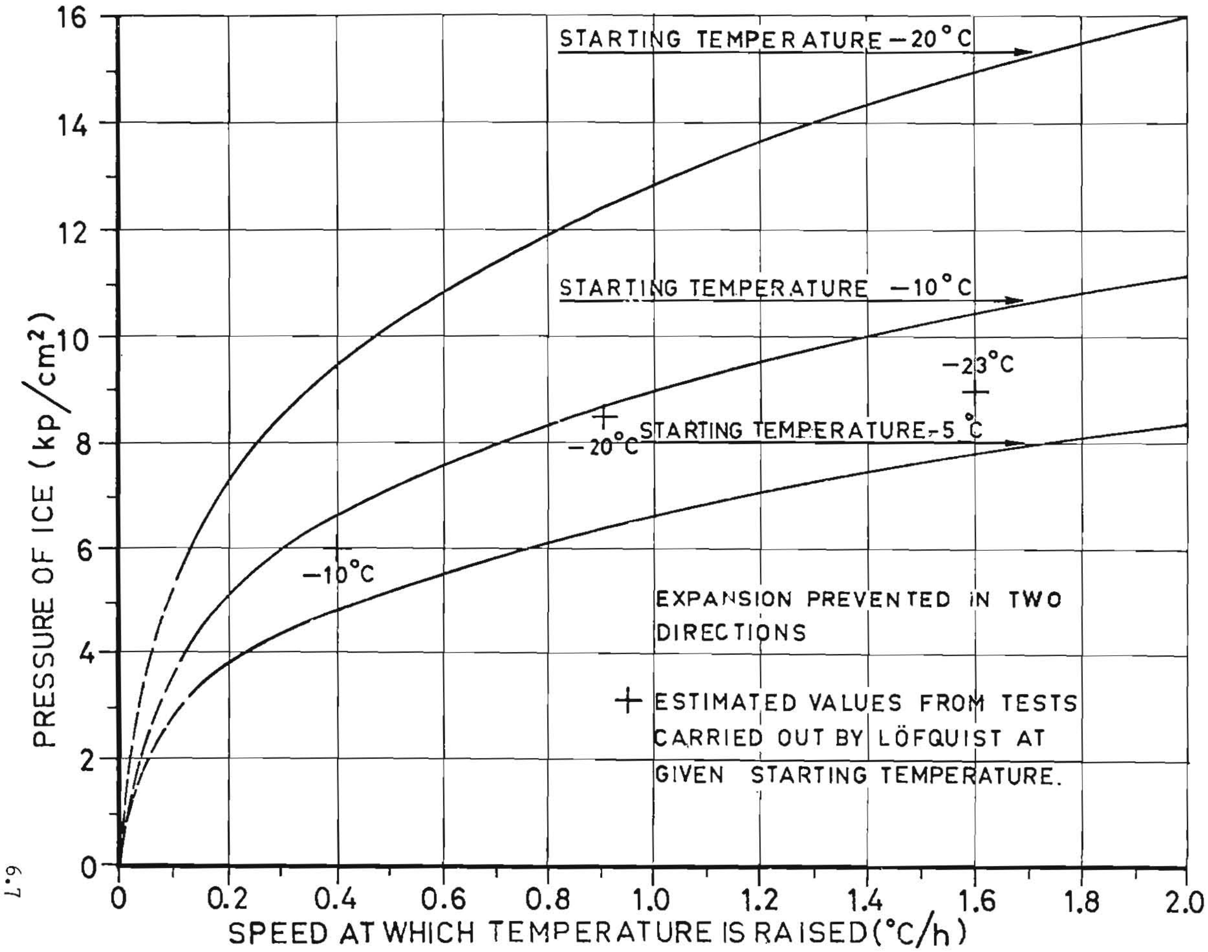
37


STARTING TEMPERATURE -10 °C
EXPANSION PREVENTED
IN TWO DIRECTIONS
2.0°C/h
1.0°C/h
0.5°C/h
0.2°C/h
0.1°C/h
PRESSURE OF ICE (kp/cm²)
TIME (h)
0
2
4
6
8
10
12
14
16
18
20

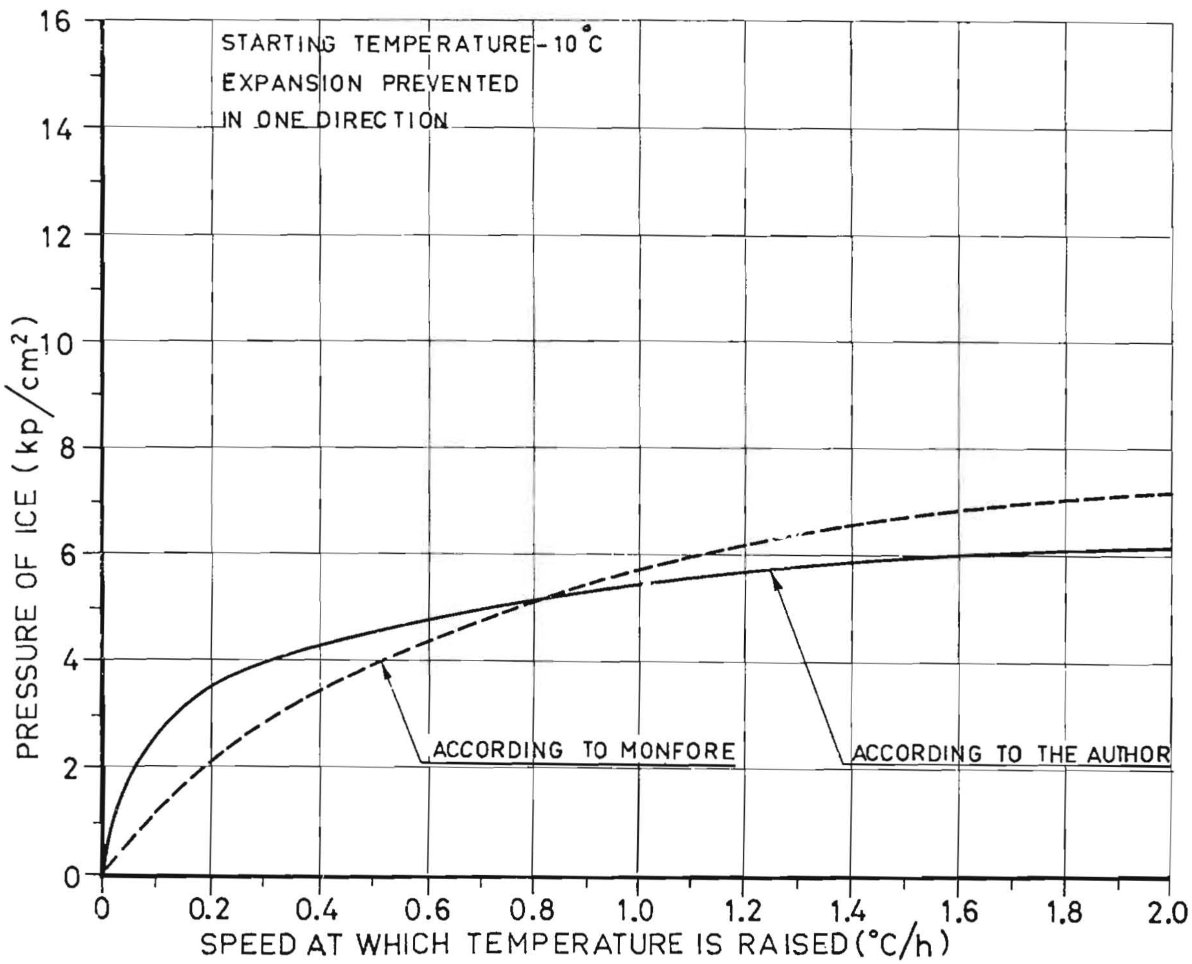
6.7


STARTING TEMPERATURE −20°C
EXPANSION PREVENTED IN
TWO DIRECTIONS
2.0 °C/h
1.0 °C/h
0.5 °C/h
0.2 °C/h
0.1 °C/h
TIME (h)
0 2 4 6 8 10 12 14 16 18 20
PRESSURE OF ICE (kp/cm²)
0 2 4 6 8 10 12 14 16

6.7


STARTING TEMPERATURE −20°C
STARTING TEMPERATURE −10°C
STARTING TEMPERATURE −5°C
−23°C
−20°C
−10°C
EXPANSION PREVENTED IN TWO DIRECTIONS
ESTIMATED VALUES FROM TESTS CARRIED OUT BY LÖFQUIST AT GIVEN STARTING TEMPERATURE.
PRESSURE OF ICE (kp/cm²)
0
2
4
6
8
10
12
14
16
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
SPEED AT WHICH TEMPERATURE IS RAISED (°C/h)

6.7


STARTING TEMPERATURE -10 °C
EXPANSION PREVENTED
IN ONE DIRECTION
ACCORDING TO MONFORE
ACCORDING TO THE AUTHOR
PRESSURE OF ICE (kp/cm²)
SPEED AT WHICH TEMPERATURE IS RAISED (°C/h)
0
2
4
6
8
10
12
14
16
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0

6.7