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POAC 81

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Vol. I
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POAC 81

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Sixième conférence internationale sur le génie maritime dans l’Arctique

Québec, Canada

Du 27 au 31 juillet 1981

Proceedings
Comptes rendus
Volume I

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PREFACE

The International Conferences on Port and Ocean Engineering under Arctic Conditions have originated in 1971, from the personal initiative of Professor Per Bruun of the Norwegian Institute of Technology, and they were an instant success.

This has been so because of intensive ocean engineering developments in the North Sea and later on in the Canadian and American Arctic waters, and also because the meetings present the right mixture of research and development work bringing together both research scientists and engineering users. Thus, in these conferences, the transfer of new knowledge is direct and results of research can be put to immediate use. This is quite a feat in the history of research where basic scientific work is usually buried in scientific journals for many years before it can be applied; and this could be done at “POAC” both because this is a new field of development that needed immediate technical solutions and because of the initiative of its founder.

From an engineering point of view, the Arctic Ocean and North Atlantic waters have many problems similar to those of other oceans. There is, however, a major difference in the environment and this is the presence of floating ice. In the previous “POAC” conferences, a major orientation was given to problems in the North Sea, then in full exploration bloom. However, the present conference is mainly directed towards the Canadian and American Arctic waters. Thus the papers are heavily weighed on ice questions, which is the major new aspect of development in these regions.

The preeminence of ice in this conference is so large that the word ice or its equivalent appears in the title of 95 papers out of 120. Consequently, several sessions had to be organized on ice action on the subjects of: “Ice Mechanics”, “Sea Ice Conditions” and “Marine Structures”. Except for those on “Meteorology and Oceanography” and “Wave Mechanics” all other sessions deal mostly with ice. They are: “Navigation in Cold Regions”, “Remote Surveillance and Instrumentation”, “Marine Foundation and Scour”, “Oil Spills”, “Sea Ice Drift”, “Interaction between Ice and Shore”, “Icebergs” and “Ice Control Measures”.

It is an honor for the City of Québec to host this conference. Our city has a long history of commercial navigation in ice covered waters dating back to the 18th century. In 1874, for example, the whole commercial fleet, which was anchored under winter conditions in Québec Harbor, was crushed by ice at the spring breakup. River and estuarine ice is a way of living with Québec people and it can be seen every winter, at our doors in the St. Lawrence River. This is why people in our Province, have one of the longest experience in research and ship operation in ice-covered waters.

This conference was jointly organized with the direct financial support of Environnement Québec of the Provincial Government and the administrative contribution of the Université Laval. Many Canadian organizations have joined with us in various manners to help make it a success. Amongst these, we would like to mention, from the Federal Government: the National Research Council of Canada, the Department of Transport, the Canadian Coast Guard, and the Department of Northern and Indian Affairs. Private groups have also been very responsive to our request for collaboration. They are: the Canadian Society for Civil Engineering, the Order of Engineers of Québec, the Canadian Committee on...

Our various organizing committees have been very active and did an excellent job in preparing the meeting. The authors’ contribution has been more extensive than ever before. All the people of these groups should be commended for their important contribution to the success of this conference.

Bernard Michel
PRÉFACE

Les conférences internationales de génie maritime dans l'Arctique (POAC) ont initialement été mises sur pied en 1971, sous l'initiative personnelle du professeur Per Bruun de l'Institut norvégien de technologie, et elles eurent dès lors un succès immédiat.

Ce succès est attribuable en partie aux développements en cours en génie maritime, d'abord dans la mer du Nord et ensuite dans les eaux arctiques canadiennes et américaines. De plus, ces conférences ont provoqué des échanges entre ingénieurs et chercheurs scientifiques et une interaction entre les travaux de développement et la recherche scientifique. Elles ont donc mené à l'application technique immédiate de nouvelles connaissances et des résultats de la recherche scientifique. Ceci est une réalisation remarquable dans l'histoire de la recherche scientifique où la diffusion du travail des chercheurs est très restreinte et reste souvent confinée aux publications et journaux scientifiques (spécialisés) pendant de nombreuses années. Si POAC a apporté ces résultats, c'est d'abord grâce à l'initiative de son fondateur mais aussi parce que le génie maritime dans l'Arctique était un nouveau domaine de recherche et de développement qui nécessitait des solutions techniques immédiates.

Au niveau technique du génie maritime, l'océan Arctique ainsi que les mers du nord de l'océan Atlantique ont en commun avec d'autres océans un nombre de problèmes fondamentaux. Par contre, la présence de glaces flottantes dans les mers froides du nord apparaît comme une différence environnementale majeure. Lors des dernières conférences de POAC, l'orientation principale était accordée aux problèmes de la mer du Nord qui était alors en voie de développement intensif. La conférence actuelle est, par ailleurs, orientée plus particulièrement vers les questions reliées aux eaux arctiques, américaines et canadiennes; en conséquence, un grand nombre d'articles traitent de questions touchant les glaces, ce qui est un important aspect du développement de ces régions.


C'est un privilège pour la ville de Québec que d'être l'hôte de cette conférence. Du point de vue historique, une navigation commerciale s'effectue sur des eaux couvertes de glaces vers notre ville depuis le 18e siècle. En 1874, par exemple, toute la flotte commerciale qui était alors amarrée dans le port de Québec pendant la saison hivernale, fut écrasée sous les glaces lors du dégel printanier. La présence de glaces sur les rivières et estuaires fait partie de la vie des Québécois puisqu'on les voit tous les hivers sur le fleuve Saint-Laurent. C'est pourquoi on a entrepris depuis longtemps au Québec des recherches sur la navigation sous conditions de glace.
Cette conférence a été organisée avec le support financier du ministère de l'Environnement de la province de Québec ainsi que la contribution administrative de l'Université Laval. Différents organismes canadiens se sont joints à nous pour en faire un succès. Parmi ces derniers, nous voudrions mentionner, de la part du gouvernement fédéral: le Conseil national de recherches du Canada, le ministère des Transports, la Garde côtière canadienne, le ministère des Affaires indiennes et du Nord. Plusieurs organismes privés ont aussi répondu avec beaucoup d'enthousiasme à notre requête sollicitant leur collaboration. Parmi ces derniers, on remarque: la Société canadienne de génie civil, l'Ordre des ingénieurs du Québec, le Comité canadien sur l'océanographie, l'Association des opérateurs pétroliers de l'Arctique et l'Association des opérateurs pétroliers de l'Est. Les différents comités d'organisation ont été particulièrement actifs et remarquablement efficaces dans la préparation de cette conférence. La contribution des auteurs a aussi été beaucoup plus considérable que prévue. Nous remercions tous ces groupes pour leur contribution importante au succès de cette conférence.

Bernard Michel
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DESIGN CRITERIA FOR NEARSHORE AND OFFSHORE STRUCTURES UNDER ARCTIC CONDITIONS

Per Bruun, Professor
Geir Moe, Associate Professor
Norwegian Institute of Technology, Trondheim

ABSTRACT

A review of the probability based reliability analyses for design of offshore structures is given. Reliability analyses with varying degree of sofistication are discussed, concentrating on the simplest method, the so-called Level I method, which utilizes so-called characteristic values and partial safety factors. Problems with this method is discussed. Then a state of the art review is given on design of coastal and harbour structures in the arctic. Finally the coastal engineering practices are discussed in light of the above mentioned reliability analysis method, Level I. It is strongly recommended that work is undertaken to implement this method.

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INTRODUCTION

One may ask what is the difference between arctic and non-arctic conditions. Apparently this must be related to physical conditions including climatic factors. That means temperatures, winds, waves, currents and their mutual interference causing deviations from non-arctic conditions. This includes changes in transfer of action and reaction of forces. Low temperatures transforms water to ice. Ice is solid water and behave very different from water, influencing flow pattern circulations and force interaction. When waves and ice join, wave action changes and so do wave forces which are now a combination of forces by water and by solid mass. Material properties also change character. Most materials become more brittle. Also tear and wear of materials change. Effluents behave differently and pollutants will usually break down much slower. Ice always puts extra loads on the structure and the entire load distribution for design changes compared to a no-ice condition.

For all offshore, harbour or other marine installations the availability of the facilities for practical use is mandatory. For non arctic conditions certain criteria on waves, tide, current and sedimentary conditions including scour must be fulfilled. The same conditions must be respected in the arctic, but here the cold climate,
ice and snow complicate the problems.

BASIS OF DESIGN CRITERIA

Probabilistic methods

During the last few decades large efforts have been spent to put design criteria and design codes on a firm and systematic foundation. The goal has been to obtain adequate, but not unnecessarily high reliabilities, for all failure modes. Also consistency in the safety margins has been desired.

A prerequisite to this work is the acceptance of the idea that there is a small, but finite probability that the structure may fail. How large the probability of failure should be, ought to depend on the costs to increase the reliability and on the consequences of the failure. As an example concerning steel structures it may be mentioned that it makes little sense to have the same reliability on joint fasteners as on major structural members, since it is quite cheap to increase the reliability of the fasteners.

For offshore platforms it is often attempted to obtain the same, low probability of failure for each platform during one year of usage. The value $10^{-5}$ is sometimes mentioned, (Refs. 33,34). This at least provides a basis for comparison between several structures under similar conditions, but is a rough criterion and does not provide any guidance on how to distribute the risks, cfr. the example concerning the cheap fasteners.

Another method consists in minimization of the expected value of the total costs. The cost of course also involves the sum of the probabilities of any failures times the total costs of that failure. Here all indirect costs must be included, e.g. for a production platform also loss of revenue due to a production stop. For an analysis along these lines to be possible, loss of human life must be evaluated in terms of money. Unpleasant as this may seem such an evaluation is implicit in any design, since a choice must always be made concerning the magnitude of reliability to be built into a structure.

For extremely simple problems it may be possible to carry out a mathematical minimization of such a cost function by means of some nonlinear programming procedure. For any practical case one would
be content to compare the expected costs of different design alternatives, however. Even for such a comparison one would have to deal with many properties that are extremely difficult to assess. Even so, it is felt that this approach is the most rational one by which to obtain the lowest risk for a given amount of money. Risk is here meant to be the probability of failure times the economic consequence of failure. The method may even offer some guidance as to the proper balance between risk and initial investment, but considering the uncertainties of the factors involved, any such analysis should be viewed with a healthy portion of skepticism. Finally, since the risk to human life is not commensurable with material losses, a separate check should be made to see that the low expected value of the total cost does not involve unacceptably high risks to human life.

There is an extensive literature on this subject. Here will be mentioned work by Cornell, Lind and Ditlevsen (Refs. 34, 35, 36, 37).

Assessment of reliabilities

The above method, though conceptually clean and logical, may be very difficult to use in actual design. Possibly the main problem is the determination of the probabilities of the different failures modes.

The reliability of a system is defined as the probability that it does not fail during its lifetime. Failure is defined by so-called limit states. Usually 2 broad categories are recognized, namely

- ultimate limit states
- serviceability limit state

The former pertains to a situation in which large plastic deformations or even total collapse occur, the latter to a situation making the structure unfit for normal use. Unfortunately, some phenomena does not easily fit into either of these 2 universally recognized limit states. The offshore regulatory agencies in Norway, The Norwegian Petroleum Directorate (NPD, Ref. 38), and Det norske Veritas (DnV, Ref. 39), have therefore defined two additional limit stated, namely

- the fatigue limit state
- the limit state of progressive collapse

The former relates to the effects of repeated loadings, the latter to whether a failure will propagate, once certain parts of the
structure is destroyed. The initial destruction may be caused by falling loads, impacts from ships, explosions or the like. Those design rules that have only 2 limit states must still cover fatigue and progressive collapse, the difference is therefore mainly one of rule editing.

**Level III safety checking** - To investigate the reliability of a structure, ideally a full probability distribution is required. This involves the knowledge of the joint distribution of failure for all design variables. Then the reliability of the structure can be evaluated integrating the joint probability over the safe domain expressed in terms of the design variables. This approach is denoted a Level III safety check (Ref. 34).

**Level II safety checks** - Here the safety is checked for a selected number of cases. The reliability is expressed by a safety index $\beta$, defined as

$$\beta = \frac{E[Z]}{\sigma_Z}$$

where $Z = Z(X_1, X_2 ... X_n)$ is a failure criterion and $E[Z]$ and $\sigma_Z^2$ are mean and variance of the variable $Z$. We may for a simple, one-dimensional case think of $Z$ as the difference between the resistance $R$ and the loading $S$. Thus $\beta$ expresses the number of standard deviations that the mean of $Z$ exceeds $Z = 0$, or $R = S$. If the probability density of $Z$ is known to be $p(Z)$, then one can easily calculate the probability that $Z < 0$, i.e. failure. If $p(Z)$ is not known, $\beta$ still provides a rough estimate of reliability. (Ref. 34).

**Level I safety checking** - This method is by far the most common in use today. It is built upon the definition of characteristic values and partial coefficients. The characteristic loads are defined separately for each type of loading and the characteristic strengths are defined depending on material, etc. The values used for the loading are meant to be the most probable highest value of the loading during the design life of the structure. For the permanent loads deviations from the design load value can be avoided by careful supervision, thus the characteristic load need not be much larger than the expected load. The live loads for an offshore structure should likewise be supervised closely, enabling accurate weight estimates, and thus accurate characteristic load estimates. Deformation loads may also be estimated fairly accurately. Environmental loads e.g. the expected highest wave should be
determined on the basis of the theory of stochastic processes. Similarly the expected largest wave generated stresses on a large volume structure (Ref. 40) or on a slender structure (Ref. 41) may be estimated. Both ought to be considered as characteristic values. Special problems arise concerning the simultaneous action of waves, currents and earthquakes. The final group of loading consists of accidental loadings, for which characteristic values are extremely difficult to specify.

The characteristic strength of any given material is according to the DnV-rules "to be based upon the 5th or 95th percentile of the test results, whichever is the most unfavourable". Partial material factors by which to divide the characteristic strength to obtain the design strength is now given by the regulatory agencies for the different limit states.

Finally the design stresses at any point of the structure may be checked against the design strengths for all design limit stated, (ultimate, serviceability etc.). To fix ideas, Table 4.1 in the NPD rules (Ref. 38) gives the combinations to be tested for the ultimate limit state.

<table>
<thead>
<tr>
<th>Load combination</th>
<th>P</th>
<th>L</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary</td>
<td>1.3</td>
<td>1.3</td>
<td>1.0</td>
<td>0.7</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
</tr>
</tbody>
</table>

P - permanent loads
L - live loads
D - deformation loads
E - environmental loads

In the opinion of the authors the labels "ordinary" and "extreme" are somewhat misleading in this context. The idea is that the satisfaction of both requirements will always yield sufficient margins of safety, no matter whether L, P or E dominate. This may be seen most easily if plotted in a 3-dimensional space (Fig. 3) in which the two requirements "ordinary" and "extreme" represents two planes. The surface representing the more conservative of these two requirements is shown in full lines in the figure and is seen to always lie above the plane P+L+E, i.e. the case for which all load factors are equal to 1. The lowest margins occur when permanent
loads and/or live loads on the one hand and environmental load on the other hand both contribute significantly to the total load. It is more unlikely that two independent random variables shall both obtain quite high values, than that one will, thus this feature seems reasonable.

PROBLEMS WITH THE LEVEL I PROBABILISTIC METHODS

For offshore structures the probabilistic methods is in fairly extensive use. This is true not only for well defined analytical subfields such as design waves, extreme forces or extreme responses, but also for the design synthesis problems discussed previously. Here Level I safety checking based upon characteristic strengths and characteristic loads in conjunction with partial strength or load factors is by now the most common approach.

In the opinion of the authors, this has provided a good basis for application of sound and consistent engineering judgement to structural design in this area. Obviously such methods do not solve all problems, however. Many of the physical phenomena involved are namely only incompletely understood. A few examples are: Particle velocity due to breaking of irregular waves, drag forces due to irregular waves, slamming forces and hydroelastic behaviour (which may dominate in the design of deep water multiple production risers).

Less obvious, but probably equally important is another problem area having to do with the Level I method itself. As an example consider a floating platform with relatively slender members so that the wave forces must be determined by use of the Morison’s formula. Let us assume that the wave force on the platform is of the form

\[ F = K_D (u-x) |u-x| + K_M (\dot{u}-\dot{x}) + V \dot{u} \]

Here \( u, \dot{u} \) is velocity and acceleration of a representative particle and \( \dot{x}, \ddot{x} \) is velocity and acceleration of the structure, idealized as a one degree of freedom system. \( K_D, K_M \) and \( V \) are constants. The use of relative velocities in Morison’s formula has been heavily criticized (e.g. Ref. 42), but that is another story. The problem in this context is that it may be unobvious how to define characteristic values and at what stage to apply partial load factors. If \( \dot{x} = \ddot{x} = 0 \) then a quadratic loading is obtained and it seems most reasonable to apply the partial load factor to \( u \), starting from a
characteristic value of u. If this factor is 1.3 then the load factor on F will become 1.69. For the general case above, $\ddot{x}$ and $\dddot{x}$ not equal to zero, the hydrodynamic force F will also contribute to the damping. Then when applying a load factor to u one will in effect also increase the damping by the factor 1.3. If e.g. wind loading provides most of the dynamic loading at resonance, and hydrodynamic wave forces provide most of the damping, then it is obvious that an increased damping due to application of a load factor on u is unacceptable. Similar problems may occur for other phenomena where loading and response are intermingled. Mathieu instabilities or hydroelastic problems (vortex induced vibrations, galloping, etc.) may be mentioned as examples. Indeed the Level I methods seem tailormade for static or quasistatic problems. For dynamic problems it is less obvious e.g. what magnitude of factors to apply to the mass plus added mass. The NPD rules (Ref. 38), Sect. 4.2.2, states that the computational models for dynamic systems shall be based on expected average values of stiffness and mass. This seems a realistic approach for most cases.

PROBABILISTIC METHODS IN COASTAL ENGINEERING DESIGN

The coastal engineering problems span a wide range of diverse problems and statistical methods are sometimes in extensive use. Still the overall design synthesis problem is normally not treated on this basis. Instead deterministic models or even empirical design formulae rule the ground. We will therefore proceed to a state of the art description of the design criteria now in common use for coastal structures, giving also comparisons to offshore problems. Then follows a short discussion, including some suggestions concerning future work to improve coastal design practices.

COASTAL DESIGN - BASIC PARAMETERS

To discuss basic parameters and thereby designs criteria it is interesting to look into the recently issued recommendations and standards by the Permanent International Association of Navigation Congresses on physical, navigational, structural aspects and in this way distinguish between non-arctic (normal) and arctic conditions. Physical factors include winds, waves, tides, currents, visibility (humidity) sediment transport and certain chemical actions. To design a harbour or marine structure for arctic conditions all these factors must be taken into consideration, plus some others including the low temperatures, winter darkness, snow, ice and their
mutual interference and influence on water bodies and structures. In the following each force factor is mentioned separately and the difference between its normal and arctic performance if any, is pointed out. Next the influence on the layout and design of structures is dealt with briefly. There is of course much similarity between nearshore and offshore physical factors but their action is different.

FORCES ACTING ON A VESSEL OR STRUCTURE

Wind forces

The wind forces $F_V$ (in Newtons) may be expressed as:

$$F_V = \frac{1}{2} \rho_e C A V_e^2$$

$\rho_e$ = the density of air (1.225 kg/m$^3$)
$A$ = the area exposed (m$^2$)
$V_e$ = the wind velocity (m/sec)
$C$ = a dimensionless form factor which also depends upon the actual size of the area exposed. For a head wind $C \sim 0.8$, for a beam wind $C \sim 1.1$.

It is normal practice to measure mean wind velocities at 10 m height. The velocity at $z$ meters height may be expressed by a logarithmic or a power law (Ref. 19).

Gustiness increases wind forces. It is therefore common to increase the wind velocity to be used for design by 20-30 per cent above the average velocity. This factor is often higher in the arctic violent storms. 3 min gusts may exceed about 50%.

There is some difference between $C$ under arctic and non-arctic conditions, $C$ increasing with lowering of temperatures.

Currents forces

The current forces, $F_S$ (Newton) by a beam (cross) current on a vessel may be expressed as:

$$F_S = \frac{1}{2} \rho_v C_D A V_s^2$$

$\rho_v$ = the density of water (about 1000 kg/m$^3$)

decreasing in temperatures below about 4°C
A = the area exposed to currents (m²)  
$v_s$ = current velocity (m/s)  
$C_D$ = a dimensionless form factor which also depends upon the actual size and draught of the vessel. It increases with decreasing water depth from about 1 in deep water to about 3.5 where keel clearance is very small (as it can happen in icefilled waters). Normally one may use $C_D = 2.5$ for a vessel at berth.

Current forces, $F_f$ (Newton) by a head current may be expressed as:

$$F_f = \left| \frac{0.075}{(\log R-2)} \right| \frac{1}{2} \frac{1}{2} \rho_v S v_s^2$$

$R$ = Reynolds number = $\frac{v s L_s}{\nu}$  
$\nu$ = the kinematic viscosity (m²/s)  
$\rho_v$ = the density of water (1000 kg/m³)  
$S$ = the wetted area of the ships hull (m²)  
$v_s$ = current velocity (m/sec)  
$L_s$ = length of ship (m)

With respect to forces by icefilled currents there is of course a difference. First the Reynolds Number changes and this will have some relatively minor influence. The main difference of course lies in the ice-filled current and its content of various types of ice from loose to pack-ice.

Obviously the speed of a vessel running through ice decreases. The actual speed is calculated by finding the crossing point of the available thrust, also including the open water resistance.

The speed depends upon a great variety of factors including ice characteristics and ships geometry, its propellers, their number, distribution and placements. Most icebreakers are provided with bow propellers which suck the ice at the bow down. In the USSR vibrating bows have been tested. In Finland air bubbles have been successfully used to reduce the resistance between the hull and the ice which in effect squeezes the vessel in between two ice walls. Numerous tests have been run and the result has been a still increasing efficiency. Due to the availability of nuclear power icebreakers can penetrate any ice covered water.
In the field of marine structures under Arctic Conditions we are facing the opposite situation. The structure is hit by the ice — it does hit the ice itself. And this makes a great deal of difference in the modes and forces of icebreaking. A slope breaks the ice easier than a vertical wall whether the "slope" is in an Icebreaker bow or is induced in a breakwater. Ice climbs up the slope and the vessel climbs (to some extent) up on the ice. No wonder therefore that there is a similarity in geometry between some cone-shaped platforms and some icebreakers.

Vibrations occur in offshore structures subjected to wave actions as well as in ice-breakers and all kinds of torques occur. Consequently some of the techniques developed are transferable from vessels to "towers" and vice versa.

Wave forces (head sea)

Wave forces on a vessel are more difficult to compute. They may become a major consideration in the case of long period waves penetrating into a harbour basin so that resonance between basin period and wave period occurs and as mentioned later this is often the cases in arctic waters.

The wave force by a head sea \( F_b \) (Newton) may be expressed as:

\[
F_b = mg H/2 \ k \ \xi \ \beta \ \cos \omega t
\]

\( m \) = the "added mass" (pressure which of course could be much higher in arctic waters with loose or packed ice)

\( g \) = the acceleration in gravity (m/sec\(^2\))

\( H \) = wave height (m)

\( k = \frac{2\pi}{L} \) (m\(^{-1}\)) wave number

\( L \) = wave length (m)

\( \xi = \frac{1}{kD} \frac{\sinh kd - \sinh kh}{\cosh k \ d} \)

\( d \) = water depth (m)

\( D \) = draught of vessel (m)

\( h \) = keel clearance (m) which could be a problem in itself for passages in arctic waters

\( \beta = \frac{3(\sin KL_s/2 - KL_s/2 \cos KL_s/2)}{(KL_s/2)^3} \)

\( L_s \) = length of vessel (m)

\( \omega = \frac{2\pi}{T} \) (sec\(^{-1}\))

\( T \) = wave period
appears to be the dominating factor. For very long waves the maximum force along the vessel will be the weight of the vessel multiplied by the slope of the water table. This is the force which may cause the vessel to move fore and back and in case of berthing it may cause breaking of ships moorings.

Forces by waves on vessels may be written in innumerable other ways depending upon vessel characteristics. In ice filled waters waves hardly ever exists, but icebergs of course present a great danger to Arctic navigation which comprehensive ice observations and navigation equipment like radar are able to solve.

The PIANC report (Ref. 19) also deals with the subject of "slowdrift" which in particular is important for oscillations of moored large volume structures. Time and space is not available for treatment of this subject which of course in icefilled waters solely becomes a current problem. Also the problem of visibility is dealt with by the PIANC commission (Ref. 19) and this is of course a problem of much more common occurrence in arctic waters than in waters which are not subjected to snow, ice and storms like the arctic. Modern navigation equipment including position equipment and radar have, however, improved this situation considerably (Ref. 20).

The main problem and the problem where the arctic differs from the non-arctic of course is in the ice itself. In the preceding section forces by winds, currents and waves have been mentioned briefly. The design criteria, however, differ little between ice free and ice-filled waters apart of course where it comes to ice laden currents which have different flow patterns, drag and inertia coefficients. These are known from the ice-breaker technique.

Ice and ice navigation

Due to the exploration for and exploitation of oil and gas in the Arctic, the interest in navigation in icefilled water is gaining still increasing interest (Refs. 12, 14, 15). An example of that is the "Manhattan-cruise" from the US East Coast to the Prudho Bay in Alaska in 1970 (Ref. 9).

In order to plan and carry out future navigation in icefilled waters (Ref. 14), it is necessary to know the ice condition including the character of the ice, which may be classified in accordance with its appearance, degree of cover and structural characteristics.
Ice may occur as "land-fast" ice and pack ice from closed to open in concentration, and in thickness that in polar regions vary from about 0.5 to about 2 meters. Navigation in ice-filled waters may advance through uniform ice, broken ice, brash and slush ice of varying concentrations. Pressure ridges have proved to be the worst obstacle against navigation as ridges of 10 to 40 meters total height may occur, presenting almost insuperable barriers to navigation (Ref. 8).

Icebergs occur where glaciers discharge ice in the sea by downbreaking. This happens in particular in Greenland waters, at certain parts of Spitsbergen, in Canada mainly at the Ellesmere Island and in Antarctica. In Canada and in the US considerable research is in progress on profiling of iceberg and to explore the ice mass as function of location, depth and wind conditions (Ref. 2). As collisions with icebergs are very dangerous an operational watch is kept on icebergs off the coasts of Greenland and East Canada. Earlier such ice observations were carried out partly by ships and partly by specially equipped airplanes which made regular sweeps over sea territories which were particularly important for navigation. During recent years ice movements have also been recorded in Alaska by comprehensive observations using radio, radar, laser. An important invention in ice navigation, however, was the use of satellite imagery (Ref. 13) which keeps a constant watch on ice conditions making it possible to predict the relative density of ice covers over large areas with a practical range of about 20 kilometers. As an example of research on ice conditions using satellites should be mentioned the microwave imagery from the LANDSAT 1 satellite in the Western part of the Davis Strait. In addition side-looking airborne radar (SLAR) has been used to determine with greater accuracy the margins of fast ice (Ref. 28). Satellite radiant temperature surveys are now in use for surveys of ice thickness (Ref. 29).

Planning and design of any structure in icefilled waters must needless to say, be based on adequate knowledge on the frequency of occurrence of covers of ice of certain magnitudes including the geometry and volume of ice bergs.

Iceberg geometry may be classified as shown in Table 1 (Ref. 7), indicating the ratio of draught to height.
**Type** | **Ratio of Draught to Height**
---|---
Tabular or Rectangular (blocky) | 5:1
Rounded (domed) | 4:1
Pyramidal | 3:1
Pinacled | 2:1
Winged or Horned (drydock) | 1:1

**TABLE 1** Type of iceberg and ratio of draught to height (Ref. 7)

The US Coast Guard found the following law (Ref. 21):

For icebergs 10–60 meters height: The height if the berg is related to the draught-to-height ratio by the power law:

\[
\text{RATIO} = 49.4 \times (\text{HEIGHT})^{-0.8}
\]

This ratio can be compared with the ratios of Table 1:

<table>
<thead>
<tr>
<th>H (m)</th>
<th>(H^{-0.8})</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.1585</td>
<td>7.9 : 1</td>
</tr>
<tr>
<td>20</td>
<td>0.0910</td>
<td>4.5 : 1</td>
</tr>
<tr>
<td>30</td>
<td>0.0658</td>
<td>3.3 : 1</td>
</tr>
<tr>
<td>40</td>
<td>0.0523</td>
<td>2.6 : 1</td>
</tr>
<tr>
<td>50</td>
<td>0.0437</td>
<td>2.2 : 1</td>
</tr>
<tr>
<td>60</td>
<td>0.0380</td>
<td>1.9 : 1</td>
</tr>
</tbody>
</table>

A more comprehensive classification of "under-ice profile data" is given in Ref. 13, which includes the following parameters:

a) linear frequency of ice keels found at depths greater than 2, 5, 10 and 15 m

b) frequency of occurrence of keels as a function of depth
c) frequency of occurrence of ice as a function of depth
d) frequency of occurrence of ice masses of given unit volumes (where an ice mass is defined as a contiguous profile area larger than 3 m)
e) the frequency of occurrence of ice mass base widths and f) the relationship between the unit volume of a given ice mass to the unit area of its base.

As an example of statistical evaluations is the results of surveys on the occurrence of icebergs of various parallels (Ref. 7). It is now known e.g. that 2500 icebergs/year cross the 60 parallel, 1000 icebergs/year cross the 55 parallel and that 400 icebergs/year
melt on the Grand Banks off New Foundland. Their drift speed down towards the banks is 10-12 miles/day. Bergs up to 23 mill. tons have been observed on the Grand Banks. The average size is 0.25 mill. tons or about 150 x 60 x 30 meters.

The US, the Canadian, the USSR and the Danish government carry out considerable observation on ice in Arctic waters. In Antarctica interests is - so far - mainly of research character by a number of nations claiming land rights in this region.

By co-operation of Baltic States winter charts are published giving ice conditions in Bothnian Bay, the Gulf of Finland, the Baltic and its approaches by agencies in Finland, Sweden and the Fed.Rep. of Germany (DHI in Hamburg). Usually ice does not coincide with other hindering phenomena.

Navigation in ice filled waters needless to say, does not only depend upon the character of the ice but also upon vessel characteristics including size, geometry and horsepower. Ships should be classified for ice navigation and ice breaker assistance is often needed.

The 105,000 DWT tanker "Manhattan" proved that navigation in the arctic waters of the North West passage is possible although ice ridges of a recorded maximum magnitude of up to about 40 meters (Bering Strait) may present an almost impenetrable hindrance which then has to be circumvented. Recent years have brought considerable discussion on improvements in ice breaking techniques including various kinds of mechanical cutting, water and steam jet.

Ref. 12 concludes that mechanical ice breaking by ice breaker combined with a low pressure steam jet device is the most effective and economical method of ice breaking.

HARBOURS UNDER ARCTIC CONDITIONS

There are basically three kinds of harbours under arctic conditions: the natural harbour, the breakwater protected harbour and the artificial island which is a fairly new invention.

NATURAL HARBOURS

It is realized that harbour structures are not too common in the
waters of the Arctic which is filled with ice most part of the year. In such areas most harbours are located in fiords and bays and the basic design criteria are that the natural harbour area is an area that is relatively well protected against ice, which means that operation in the harbour is not too difficult as long as it is possible at all to enter harbour areas. There are places on the arctic shores of Canada and the United States as well as in East Greenland where the natural harbours are only accessible in 1-2 months. The harbours have no other breakwaters than natural fiords, headland islands and reefs. They are exposed to some wave action for a small part of the year only. The Design Criteria therefore solely requires the best protection against ice for the wharf pier or other loading facility which is built. The way this problem is solved is by field observations through a number of years. During recent years not only experience but aerial photography, including satelite pictures (Ref. 13) have proven to be a great help in this respect, as well as to ice navigation in general.

When a site has been selected the next question is the layout. In this respect the report by Group IV of the PIANC, ICOBELS (Ref. 20) gives adequate information but it does not mention the ice problem at all. The solution to this question, however, is the same as for the site selection. One must observe where a quay wall, whether a solid (preferable) or a piled structure can best be built with maximum protection against waves and ice. With respect to waves one must remember that "waves" include "short period" (wind) as well as long period waves (swells T > 30 sec).

In the arctic fiords long period waves (60 sec up to several hundred seconds) often play an important role due to concentrations of wave energy, reflections and the natural geometry. (See Refs. 10 and 25.)

MAN MADE HARBOURS, DESIGN

First of all one must remember that ice filled waters are not always icefilled, even though icebergs may be floating around at any time of the year in some of the arctic waters where harbours or other marine structures have been installed.

The differences in the forces are due to changes in flow behavior and force coefficients which are caused by changes in viscosity and changes in roughness. They are relatively small and not important.
The most important change, however, is in the action of ice. Currents have very different force coefficient in ice filled and not ice filled waters. Passage of a vessel in a fair-lane or approach channel is entirely different in waters with no ice and in waters which are more or less filled with ice. This influences navigation very considerably. If the ice floes are not very much packed, the influence on navigation will be small apart from perhaps it may cause more trouble to keep the ship on its right course. If ice is packed the situation is different but the situation depends upon the ships power. The slower it moves, the more will the ice sidetrack it. It may be necessary to count on considerable deviations in the ship's path. This is important when the vessel passes through an entrance or similar narrow spot.

Proceedings of POAC 5th Conference in Trondheim, Norway, 1979, Vol.2 (Ref. 30), includes several papers on Breakwater Design as well as construction including evaluations of the most dangerous wave actions. Groupings of waves are considered as well as special sequences of waves (time series) by Günbak, evaluation of failure probability by Losada, Random Wave Attacks on a Composite Breakwater by Yamamoto et al, Experimental Analysis of Wave Interaction with Rockfill Embankments with impervious cores by Næser, Scale effects discussed by Tørum and two very practical papers one by Ali Mesta on Prototype Behavior of a Breakwater and another on Practical Views on the Design and Construction of Mound Breakwaters by Bruun and Kjelstrup. No reason is seen to repeat the results of these papers here but a brief comparison of earlier practices with new practices is mentioned below.

BREAKWATERS
Earlier Practice
Earlier Practices were strictly of empirical nature. Although the problem is of detailed hydrodynamic nature and includes soils as well as (very important) interior solid aspects they were all left out of consideration.

The application of weight formulas generally used were all of the type

\[ W = \frac{H^3}{(Sr-1)^3} \tan \alpha K_D \]

where \( W \) is weight of block, \( H \) = wave height, \( Sr \) is specific gravity of armor and \( K_D \) a fraction of great variability.
As explained by the PIANC's (1976) Wave Committee (Ref. 17), such formulae are not reliable and should only be used for preliminary design. Ref. 13 shows a "Christmas tree" of results of various formulae.

**New Practice**

A much more reliable design procedure is recommended by the PIANC 2nd Wave Committee (Ref. 17). It is based on the use of the 

\[ \xi = \tan \alpha \cdot \frac{\sqrt{H}}{L_0} \]  

(Iribarren's breaking criteria number) where \( \alpha \) is slope angle, \( H \) = wave height in front of the structure and \( L_0 \) = wave length. \( H \) may be interchanged by \( H_b \). This is all described in Ref. 5 and in Ref. 30, POAC-79 Proc. Vol. 2 and in Ref. 32.

Interest is now much more concentrated on the zero damage stability number \( N_{ZD} = H_{ZD}/(W_{50}/\gamma r)^{1/3}(S_r-1) \) versus \( \xi \) relation or interaction. This refers to stability as well as to uprush and downrush. A number of results are available and more are coming.

The new design principles are based on a clear distinction between the forces which are acting on the armor layer as a whole as well as on the single armor unit.

It is very clear that stability as well as up- and downrushes depend upon the \( \xi \)-factor, that means on the character of the breaking wave, and that the plunging wave which occur for

\[ 0.4 < \xi_b < 2.0 \]  

(Ref. 5)

where \( \xi_b \) refer to the breaking wave condition is the most dangerous wave, particularly when in groups of the same period as the wave uprush/downrush period. But other special types of waves sequences (time series) and single huge waves of solitary wave character may be as dangerous or rather worse (POAC-79, Ref. 30, Günbak).

This is all explained in the above mentioned papers in Vol. 2 of the POAC-79 Conference and in addition is given suggestions for probabilistic approaches and risk analyses. The investigation of the stability of a breakwater must of course include hydrodynamic as well as structural aspects. While the former has been realized for long, the latter has not been recognized until very recently.
That the single unit of course must be stable in the armor layer seems obvious. If not the departure of a single unit could cause the departure of other units. That means the units must stick together by friction (like natural rock) or interknitting (like multilegged blocks). But that's not enough. The armor layer as a whole must not slide down, but be well tied in with its sublayer. And the entire armor layer must of course have a solid toe protection which partly supports the armor against scour and partly support the armor against an overall sliding which could occur if the frictional connection between armor layer and first sublayer(s) is not intimate enough (Fig. 1). It is of course also important that the volume weight of the armor + water is not too low because the void ratio is high. After all we cannot get by with an armor which is mainly water. Some of the multilegged concrete blocks have void ratio's exceeding 50% which makes sluidization easier than with natural rock where void ratio is rather 25-35% depending upon the character of the rock.

To decrease the possibility for sliding we must provide adequate friction between the armor and the sublayer (POAC-79 paper by Bruun and Kjelstrup, Ref. 30). This has not been realized until recently.

Hydrodynamic Aspects

The progress made during the last two years may best be summarized by reviewing what a model study of a breakwater must comprise of:
- Tests on perpendicular waves
- Tests on waves coming in under an angle
- Introductory test using regular waves
- Tests using irregular waves based on time series from actual recordings. Attempts should be made to upscale these series which include wave groupings, waves of solitary character, succession of waves of particularly dangerous character to stability (time series). Wave action will be varied for various sections of the breakwater to the extent this is warranted and justified by technical-economical estimates. Breakwater heads must be subjected to special tests because they are more exposed.

A test programme like the one outlined preliminary above is of course comprehensive and all justifiable efforts should be made to limit it in space and time. The following measures or methods are available.

Tests in a relatively small scale e.g. 1:80 All introductory tests
of "discriminative nature" may be run in a relatively small scale with regular and/or irregular waves. It will only be an advantage to compare the results of the two different series in some introductory tests with special reference to the effect of wave grouping. With respect to scale effects, they must of course be considered. In this respect progress is slow. See e.g. Proceedings of the Santander University Seminar, Spain, on Breakwaters in 1980. At this time it is wisest to assume that the model is not conservative.

A very considerable amount of information can be gained by such preliminary tests where less practical or less economical solutions may be eliminated from being considered in time-consuming and expensive tests. All "common reasons for breakdown" should be considered (Ref. 6).

In construction of the model, it is very essential that the construction method in the model corresponds to the method which will be used. This is to secure realistic tolerances. It is suggested that design engineers as well as constructors advise on this.

It is of no importance to include winds in such tests to evaluate overrun and splash.

Use of the no-overtopping criteria which is common is considered as being impractical in this case. The criteria must be:

1) Overtopping must not contribute to increase of damage of any part of the breakwater. The crown as well as the back slope onward must be sufficiently strong to stand up to any kind of overtopping including solitary waves of considerable magnitude which often occur at the end of storms;

2) Overtopping must not cause wave action of any practical importance in the entrance or in the outer harbour which could present a danger (or nuisance) to navigation.

A special problem which sometimes becomes of great importance is the scour problem at the toe and in front of the breakwater. To meet this problem, the entire bottom in front of the breakwater should be photographed to locate weak spots where scour could occur, for which reason the toe of the breakwater should be designed accordingly.
In this respect, it should be remembered that the reflection coefficient from a breakwater varies but may be of the order of about 25% for long waves. This increases bottom shear stress by about 50% and the scour ability up to 3 times. This should be taken into consideration in the toe where the situation is that the combined shear stress, including stresses by downrush, could cause considerable scour. It is easy, however, to avoid by just extending the toe as a mattress in front of the main toe structure. The width of the mattress could be as much as 10-20 meters (model studies).

It is self-explanatory that:

a) Tests should be run to the highest exposure limit expected with little or no damage to the structure - while overtopping as mentioned above should be permitted to an acceptable extent;

b) Tests should be extended beyond that limit to study damage development and breakdown pattern.

When the results of the preliminary tests are available, they should be subjected to technical as well as economical scrutiny by the client, the designers (including laboratory), and the contractors. Economy analyses of a preliminary nature should also be undertaken before selected tests on a large scale are run.

**Selected tests in large scale (> 1 in 30).** These tests should be run in a scale no less than 1 in 30. It is essential that measures against the effects of wave reflection are taken, that winds are included in the tests and that time series are reproduced correctly. This limits at this time the availability of laboratories which are able to run such tests. With meticulous and thorough planning it should, however, be possible to limit the number of tests, thereby cutting down time and cost of the experiments.

It is emphasized again that tests must include such time series which are a result of the combination of waves overtaking each other or coming from various directions, refracted and defracted. The actual records therefore are important for the evaluation of these waves. Equipment is available to produce such situations but it may become necessary to combine two of the major laboratories in Europe because one has the physical geometry facilities and the other has developed the technique for producing a certain time series at a certain point at a certain time. Such tests are of very
short duration and must therefore be repeated until, by probability considerations, one can feel sure that the total occurrences in a peak storm have been exceeded in the tests.

This does not, in any way, conflict with the normal time series tests which shall be run as they occur in trains. It is the solitary and odd wave situation which consists of one, perhaps two or three waves which are of particular interest. They may become determining for the stability. The "fatigue" (usually grouped) waves are another source but the fatigue may come fast.

**Structural Aspects**

That the single unit shall be structurally sound and solid seems obvious — until it was proven, by unpleasant experiences, that this was not true for some of the multilegged blocks. Their single members were simply not strong enough to carry the load from neighboring members. As all multilegged blocks are placed on steep slopes this means that some blocks or block-members became subjected to forces which simply broke the block perhaps even _before they had ever been subjected to wave action_. That the greatest number of broken blocks are usually found in the lower part of the slope or often around the mean water or low water table is proof enough that the block, that means one of its single beams or members, may have been subjected to just static forces strong enough to break them by the neighbouring blocks gravity (Fig. 2). And the situation does not improve by wave or — even worse — ice action. In other words this may amount to a major structural mistake, difficult to correct but still correctable by avoiding too steep high slopes e.g. by means of a berm in front if applicable due to depth and wave conditions (resonance effects).

**Conclusions regarding breakwaters in the Arctics**

It is by far the best to avoid them! If necessary they shall be designed for the maximum ice-free wave situation and for the expected ice pile-up elevation climbing them during the ice season. This usually causes little damage apart from perhaps moving some blocks in the crown or in the wave screen. Layout should be based on local observations of ice movements.

**ICE VERSUS WAVE ACTION**

If ice predominates most of the year it at least cuts the fetch so
that the situation is that waves are less in the fringes of ice covers and perhaps non-existing during the ice-period. This is the case in Alaska, Canada, Greenland, in the US and Canadian Great Lakes. What is surprising under such condition is that it is the no-ice condition which is determining for the stability of the breakwater or wharf - if any.

The movement of ice up on a breakwater or other slope is dealt with by Tryde (Refs. 26, 27). Bruun (Ref. 4) gives data for the height of ice pilings at various types of structures which are often overrun by ice possibly causing some damage to the upper structure.

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 elevation of 10-15 m above still water level. Local observations and comparisons should confirm the local criteria.

2) Vertical walls or steep 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. This is of particular importance in harbour basins as mentioned below. Ice pressure must not penetrate to deck level in a piled structure.

ICE RIDGES. DIMENSIONS AND FORCES

A problem in entering a harbour, whether natural or breakwater protected, is the existence of ice ridges which often tend to build up in entrances, particularly if they are narrow. In many cases ice ridges build up on the bottom in front of the structure - as they do in the open sea and may even reach thicknesses exceeding 40 meters which is non-penetrable for all normal vessels, including the "Manhattan".

Sackinger in Ref. 24 mentions that stresses up to about 10 kg/cm² were measured in grounded pressure ridges.

Ref. 3 (Correll, Blidberg and Westheat) mention maximum thickness of ice up to 15 to 17 m. The POAC Proceedings have numerous other papers on this subject.
Such ridges block all navigation without the assistance of ice-breakers. But normal ridges are 2-4 meters only.

But otherwise navigation usually is possible for vessels with larger power and special bows to meet the reaction forces by the ice. The winter or early spring traffic in the St. Lawrence river is an example of this.

The movement of the ice covered waters in the open ocean is usually of very modest magnitude.

According to Ref. 1 major winter landfast ice movements are associated with coastal storms when winds exceed 10-13 m/sec. When ice is occurring in floes movements may be faster but still not in any way detrimental to navigation. According to Ross Peters (Ref. 16), the average floe size in the Labrador Sea was about 10 m with a few up to 30 m.

The movements of ice blocks suspended in water in low concentration (the blocks don't generally touch each other) have the same velocity as the water movement. Fast ice moves much slower, perhaps a few meters per hour but strong currents may speed up the movement considerably according to Speddings observations, mentioned in Proc. POAC-79, Ref. 30, Vol. 1.

While ice ridges in the open ocean is definitely a severe hindrance to navigation because they may be very difficult or impossible to penetrate, ice ridges building up in front of breakwaters may sometimes be considered a blessing because they stop the continued building up of ice against the breakwater and thereby overtopping, see Fig. 2. This happens e.g. in Great Lakes harbours (Ref. 4).

ARTIFICIAL ISLANDS

Artificial Island has already for some years been built in Alaskan Waters partly as sand fill island on 10-20 meters depth and partly on stranded icebergs. In either case they have been used as drilling platforms.

The forces these islands are exposed to are ice-forces. Such islands may still be considered to be on the testing stage, but they have an advantage over other platforms (including cone-shaped)
that they cover a considerable area useful for production of products.

Design principles of artificial islands are mentioned in Ref. 18 with no reference to ice condition. All POAC Proceedings discuss this subject and test results.

FORCES BY ICE ON LARGE OBJECTS

The literature available on forces and pressure by ice is very comprehensive, including 5 sets of POAC Proceedings with almost 100 papers. Many factors play a role including the character of the ice, its movement and the geometry and size of the structure. The ultimate strength increases with the thickness of ice to width of structural member and forces may vary from 30 to 100 t/m² from slim to solid structures.

A paper by Reddy et al (Ref. 22) gives a very thorough review of non-stationary response of offshore towers to ice forces. Harbour structures, however, are hardly ever exposed to such forces.

THE BEHAVIOR OF ICE IN A HARBOUR WITH SPECIAL REFERENCE TO BASINS WITH QUAY-WALL OR SIMILAR INSTALLATIONS

Ice creates many problems in harbours all over the arctic Ocean as well as in the Baltic with adjoining seas. Every winter these harbours freeze over for a shorter or longer period. Canada, the US, the USSR and the Scandinavian countries, apart from Norway, have large ice breaker fleets which keep navigation moving in the icy period. Norway's West and North Coast is blessed with the Gulf Stream and Norway has little ice problems apart from occasionally in the Oslo Fiord and the interior of some fiords.

Ice problems in a harbour basin may be solved by icebreaking. The worst problem always occur along the quay wall where ice tends to thicken making it difficult to berth a vessel. Such ice may still be removed by propellers from tug boats if it does not build up too thick. The problem is often what to do with the ice. To discharge warm (cooling) water in the harbour is an advantage or to push it out by generating outgoing currents. An ice boom at the entrance may be practical for small ports but it has to be very strong too. The advantage, however, is often limited if the sea outside the
harbour is frozen too. In some cases broken channels may, however, be able to stay open for days and weeks if there is enough traffic through them to hinder new ice formation. The rate at which the ice grows varies greatly. One method for ice thickness predictions is described in recent reports by Sandkvist in Ref. 23.

The static growth of ice is well described by the degree-day method, which is:

\[ h(t) = a \sqrt{D} \]

where \( D \) = summation of daily negative mean temperatures
\( h(t) \) = the static ice thickness
\( a \) = an empirical coefficient depending on wind conditions and especially on the thickness and condition of the snow cover on the ice

The best curve fitting was obtained when a degree-day coefficient of 2.2 cm \((^\circ\text{C}-\text{day})^{\frac{1}{2}}\) was used for the conditions he examined. The estimated degree-day coefficient agrees with coefficients determined for lakes of northern Sweden.

A few practical experiences from harbours are mentioned below referring to a report by Bratteland in Norwegian (1977) including:

1) Massive structures cause less icing than piling structures.
   The use of steam or rubber sheets is helpful in removing the ice cover.

2) Straight vertical walls with deep water in front will tend to force the ice down while sloping structures will build up ice.

3) One may attempt to keep the ice away from piles and columns e.g. by sawing holes around the pile to avoid heavy uplift forces on the pile. Uplift forces may become very large e.g. 1-2 t/m with 0.5 m ice thickness and about 0.5 m rise of tide.

4) The harbour should not be left without any attempt to break the ice as this may cause strong horizontal forces due to thermal expansion (up to 30 t/m² measured in the USSR).

5) Piles may be provided with steel-caps to protect the concrete against corrosion by the ice.

MATERIALS FOR CONSTRUCTION

Regarding the proper materials to be used for harbour structures under arctic (cold water) conditions several factors play a role.
They include:

1) The availability of material
2) Its resistance against attacks by sea water of sometimes varying salinity
3) The tidal ranges (exposure to air)
4) The cost of the material

Summarizing the information from the POAC Proceedings and other information one may conclude that:

materials for breakwaters if needed at all should, if possible, be native rock of as great resistance as possible. This occasionally create problems regarding availability. Some rock is stratified and tend to split by freezing and thawing. Another problem is the distance to the available source and the size of rock needed. The latter refers to exposed locations where rock exceeding 10-20 t is needed. This problem does not exist with offshore structures where steel or concrete are the preferred materials.

The resistance to corrosion, whether steel or concrete, is a problem, however.

At certain places e.g. in Greenland, soils conditions have only allowed the use of steel and double-anchored wharves have been used (Ref. 30, POAC-71, Greenland Session). This has caused some corrosion problems but as driving of piles is very difficult and tidal ranges are high, a primary experience has been that a main consideration of the design engineer should be to try to reduce underwater work as much as possible and to make the structure as independent of the bottom conditions as possible.

If steel is used for various purposes in marine structures it is necessary to corrosion-protect it e.g. by placing a protective cover. Protective measures could range from concrete encasement in asphalt-bonded corrugated culvert pipe to use of anti-corrosive measures like cathodic protection of various kinds, common in offshore platforms.

In Greenland Greenhart has been used in certain cases in open as well as closed structures (POAC-71, Greenland Session, Ref. 30).

Concrete has proven to be a material that is very adaptable to arctic conditions. Comprehensive research in Norway reported in Ref. 30,
Material sessions by Gjørv, concludes as follows:

"On the basis of the comprehensive field investigation carried out along the Norwegian seaboard, the following conclusions appear to be warranted:

1. Chemical deterioration of concrete in seawater may also represent a problem under arctic conditions, depending on the C$_3$A-content of the cement and the permeability of the concrete. For ordinary portland cement with high C$_3$A-contents, a water-cement ratio of not more than 0.50 should be used.

2. For underwater placing of concrete where the permeability of the concrete may rise locally during the construction, a sulfate resisting cement should always be employed.

3. In the deterioration taking place in the tidal zone freezing and thawing is supposed to be the most important mechanism. However, the observations indicate that chemical deterioration also plays an important role in this breaking-down process.

4. Apart from the deterioration observed in the tidal zone, damage due to freezing and thawing was mainly confined to a small extent of scaling. However, even better conditions would have been observed if air-entrained concrete had been more widely used.

5. Corrosion of reinforcing steel represents the most extensive and serious problem to the structures inspected. Damage was only observed above spring high water level, but this does not exclude that a certain extent of steel corrosion also took place in the submerged parts of the structures.

6. A study of the pattern of the damage due to steel corrosion as well as of the mechanism of the electrochemical corrosion, indicates that all those parts of marine concrete structures which are the most exposed to intermittent wetting and drying, also will be the most vulnerable to steel corrosion.

7. Detrimental amounts of chloride-ions were found to have penetrated into high quality concrete beyond what would be a practical limit for the thickness of a concrete cover. Thus,
attention to additional precautions for inhibiting steel corrosion should be paid when concrete structures are exposed to severe marine environments.

8. The observations further indicate that repairs of damage due to steel corrosion should never be considered as being just a question of recovering the steel where the concrete cover has spalled off. To obtain a more effective control of a deterioration due to steel corrosion, all the consequences of changed electrolytical conditions should be considered before any repairs or any coatings of the structures are carried out."

Prestressed concrete seems to have been successful both in the real arctics and in the North Sea Waters as e.g. reported on in paper by Ben Gerwick in Ref. 30, (POAC-71, Materials Session) and in numerous cases e.g. the gravity platforms in the North Sea including Condeeps. As expressed by Gerwick "High quality prestressed concrete appears to be especially well suited for the construction of a wide variety of arctic structures. It possesses excellent properties for resisting the environmental loadings and conditions and is inherently practicable and economical. As further development of the Arctic Ocean and its adjoining waters takes place, prestressed concrete will undoubtedly play a major role." It did! It is also explained how various "coatings" can be used.

HYDRAULIC MODELLING OF ICE COVERED WATER

As explained by Kotras et al (Ref. 11) hydraulic modelling can play an important role in solving problems associated with ice-covered waters if it is shown to scale prototype conditions accurately. Their paper presents an unified set of scaling laws for both distorted and undistorted hydraulic models of ice-covered waters. Next a unique modelling material which exhibits similitude with prototype ice properties is introduced and examples of its use in hydraulic models of portions of the St. Lawrence seaways are described and the general role that hydraulic modelling can play in solving ice-related problems is examined.

DISCUSSION OF COASTAL ENGINEERING PRACTICES

Many aspects of the coastal engineering design practices could easily be incorporated in a Level I design code. Thus the wave and current forces discussed previously could easily be recast in a form yielding...
characteristic loads. There are hardly any principal problems involved with the ice loads, either, thus the same philosophy could be adopted there. There is at the present time a lack of ice data on which to base such characteristic ice loads. Still it would be beneficial to adopt a common theoretical base for coastal design work, at the same time providing a common framework for presentation of the ice force data to be obtained later.

Probably the most difficult coastal problem involves breakwaters subjected to waves. These structures usually are of vital economic importance, and several failures have occurred during the last few years. It may be safe to say that formulae relating wave height to block weight are unsatisfactory. Many other features of the breakwater are important than just block weight, and many other features of the wave train are important than the wave height. Such formulae make no attempt to distinguish between the different failure modes, nor is anything said about the intended margins of safety.

The loads on a breakwater could be inferred from a dimensioning seastate which preferably should be determined by means of actual measurements on the site. Time series could then be made to correspond to the power spectral shape of the given seastate. (Several realizations possible, so-called simulation). This would be the characteristic loading which ought to be increased by a certain, fairly low load factor to account for the fact that the most unfavourable combination of individual waves may not have been obtained by the chosen time series. The characteristic strength may be even more difficult to obtain, and much research remains to be done before this problem is completely understood. What must be done is to identify the failure mechanisms, as for instance removal of individual blocks of the outer layer, sliding of the outer layer, crushing of blocks, fluidization accompanied by pull-outs, etc. Each failure mechanism follows its own model law, thus model tests are rather unreliable unless failure mechanisms are considered. The application of partial strength factors would require an analytical model so that one could predict what a division by 1.3 would mean for block strength, etc.

Unwieldy as the problem may seem, it is still urged that work is undertaken to implement a Level I reliability analysis also to breakwaters. Indeed soil mechanics problems exhibit many points of similarity to breakwater design, and the Level I reliability ideas
are well established here.

CONCLUSIONS

The probabilistically based methods of reliability analysis will in
the opinion of the authors dominate in all future engineering
design rules, and no reason is found why coastal and harbour
engineering should be an exception. The simplest version of these
reliability methods is the so-called Level I reliability analysis,
which is by far the most common in practice. Thus it is urged that
work is undertaken to implement the Level I method to coastal and
harbour problems. Even though the establishment of such method
solves few problems by themselves, it will at the very least provide
a reference framework for future work.

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SCHEMATICS

CONNECTION BETWEEN ARMOR AND SUBLAYER

FIG. 1

MULTILEGGED BLOCKS ON STEEP SLOPES ARE SUBJECT TO HIGH PRESSURES BY OTHER BLOCKS PARTICULAR IN ITS LOWERMOST LAYER: THE RESULT IS BREAKAGES OF ELEMENTS, TRUNKS, FLUES ETC.
SCHEMATICS
ICE PILE-UP ON STRAIGHT SLOPE

SCHEMATICS
ICE PILE-UP ON BERM OR REEF SECTION

FIG. 2
FIG. 3

- Nonactual surface
- All load factors equal to 1
- Actual surface
PRODUCTION SYSTEM IN ARCTIC WATERS

BY USING A FULLY INTEGRATED TSG PLATFORM

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ABSTRACT

The problem of installing a fixed platform for continuous production systems in arctic waters might be solved in some cases by using suitable auxiliary systems as a defense against ice action.

In other cases it might be economically convenient to interrupt the production and recover the platform in danger time intervals.

These cases can be solved by a production system based on the use of the Tecnomare Steel Gravity (TSG) platform, modified to assure quick positioning and connection on a proper subsea completion and manifolding system, and very quick recovery in case of danger.

This solution uses the basic technology already applied in "early production" systems developed by Tecnomare for the North Sea.

The implementations of the basic TSG platform are presented and discussed with particular reference to:

- modification to the structural configuration
- equipment and procedure for a very quick structural and functional connection/disconnection
- mooring and loading point integrated in the platform
- special configuration of the bases of the platform.

The paper deals with the technical aspects relevant to the above solution.
INTRODUCTION

Exploration drilling in the Labrador Sea has proven that this is a potentially high hydrocarbon area. Presence of pack ice and icebergs prevents the use of conventional production platforms.

A proper production system must have the capability to move off location to avoid any iceberg collision.

The proposed approach utilizes existing and proven technology, with proper modifications, for the solution of the problem of iceberg collisions.

The Tecnomare Stell Gravity Platform (TSG) has already been used for the development of the Loango field (offshore Congo) and of the Maureen field (North Sea).

Fig. 1 shows one of the four TSG drilling and production platforms installed at Loango field during towing from the construction yard to the installation site.

Fig. 2 shows the axonometric view of the TSG used for the Maureen field, an application of an integrated drilling production platform with storage capacity to be positioned over a sea bed template for early production.
Among the main features of the TSG platform the most important characteristics are:

- Capability of transportation of the fully equipped deck
- Capability to be positioned over a sea bed template with high accuracy
- Possibility of recovery whenever necessary
- Integrated storage capacity.

The above features with proper modifications and improvements have been utilized to develop a TSG configuration for Arctic Sea applications.

POSSIBLE PRODUCTION SYSTEMS

The basic concept of this TSG configuration is that the platform is positioned over a subsea cluster of underwater wellheads positioned inside an excavation of proper dimensions to be protected against iceberg scouring.

A production riser connects the control subsea manifold to the deck workover production facilities.

The TSG may be utilized in different configurations to comply with many production schemes. Both seasonal and all year round production possibilities are foreseen.

As for the product transportation, if favourable conditions permit the use of a sea line properly protected from iceberg scouring, the transport of oil or gas could be carried out directly from the central subsea manifold to shore.
As an alternative to the sealine, storage capacity can be integrated in the production platform.

Storage is obtained by using part of the cylinders and/or separate underwater storage tank integrated with the foundation bases of the platform.

The solution described in the following foresees oil storage integrated in the platform cylinders. The oil loading system is integrated in the platform deck: a proper reel system provides pay in/out of flexible hoses, while the tankers are moored to one of the four mooring points necessary for the platform positioning.

Fig. 3 shows a general view of the proposed mooring/loading system.

- Fig. 3 -

**DESIGN CONDITIONS**

In order to show the suitability and flexibility of the TSG concept in Arctic Sea, a range of water depths (100–200 m) and installed payloads (7500–15000 m) for typical environmental and soil conditions offshore Labrador have been investigated.

A representative case will be shown in detail in the next section and refers to the following design data:
Production rate : 100,000 bbls/day
Pay load : 10,000 t
Deck area : 3,500 m²
Water depth : 125 m
100 year storm
wave height : 29 m
wave period : 15 s
wind (1 min sust.) : 36 m/s
Max operative storm
wave height : 18.7 m
wave period : 13.7 s
Current
surface : 1.3 m/s
bottom : 0.3 m/s
Soil : Morenic silty sand
Storage capacity : 200,000 bbls

PLATFORM CONFIGURATION

The proposed TSG for Arctic Seas is an extension of the original Tecnomare tripod steel gravity platform.

The basic design philosophy for this TSG platform is as follows:

- To provide a production workover platform in which a storage capacity is built in.
- Capability of being towed, installed and recovered with fully equipped deck in vertical position.
- Good seakeeping performances.
- Minimization of hydrodynamic and ice loads.
- Adoption of three foundation bases in order to minimize the effect of possible soil unevenness and differential settlements.
- Capability of positioning of the platform on a sea bed cluster accommodating underwater wellheads.
- Capability of a quick disconnection and recovery to avoid iceberg collision.

Fig. 4 shows the general configuration of the basic case and relevant main characteristics.

Mainly the structure is composed of:

- Hexagonal lattice tower.
- Three cylindrical tanks that provide storage at location as well as buoyancy while floating.
- Three bases at the lower part of the cylinder accommodating solid ballast which provides the platform with the necessary stability while floating.
- Three bearing pads as gravity foundation of the platform.

Tower, cylinders and bases, structurally connected, represent the recoverable part of the TSG system, while the three pads once installed act as fixed support on sea bottom (See Fig. 5).

The interface bases/pads represent the key point of the TSG for Arctic Seas and it is based on the suction effect inside a proper seabed chamber between bases and pads.

Fig. 6 shows the general configuration of the interface base/pad and a general flow sheet of the suction system.

The suction system, activated during the mating, assures the in life platform on bottom stability.

In case of possible iceberg collision the system is used for water injection into the chamber to permit quick disconnection and refloating of the platform.

This procedure permits the reduction to a minimum of platform refloating since it is possible to evaluate the possible collision routes when the iceberg is at a short distance from the platform.
Fig. 5

BASE/PAD INTERFACE

Fig. 6
A possible TSG installation is shown in Fig. 7.
Alternatively the three pads can be towed under the bases and installed together with the platform. The choice of the proper solution is made as function of the adopted construction procedure and of the actual soil conditions.
Fig. 8 shows the platform refloating procedure in case of possible iceberg collision.
Fig. 9 shows the platform weight trend as function of the installed payload and of the water depth with respect to the basic case.

CONSTRUCTION AND ASSEMBLING PROCEDURE
The modular configuration of the TSG platform permits the distribution of the prefabrication of the main structural blocks in several yards and their assembling procedure may be adapted to the different techniques and characteristics of the selected yards.
As for the assembling procedure of the platform both a construction in dry dock and in a wet sheltered site can be followed.
The fully equipped deck can be built in a different yard and mated with the platform in a deep sheltered site supported on two barges in a catamaran configuration.

PLATFORM INSTALLATION

- Fig. 7 -
PLATFORM RECOVERY

1. Riser Disconnecting and Platform Deballasting
2. Underbase Water Injection and Platform Disengagement

- Fig. 8 -

3. Platform Deballasting
4. Platform Towing to Recovery

- Fig. 9 -

PLATFORM WEIGHT TREND

- Fig. 9 -
CONCLUSIONS

Production of hydrocarbon in iceberg infested waters by means of TSG platform can be achieved by the application of existing and proven technology with modifications required by the particular operating and environmental conditions.

The key feature of TSG concept for Arctic Seas is the base/pad connection/disconnection system which permits the quick recovery of the platform in case of possible danger.

The TSG platform weight trend with respect to water depth and operating payload has been presented for a preliminary budgetary cost estimate of the structure.

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ABSTRACT

Since 1973, Panarctic Oils Ltd. have drilled sixteen offshore wells in the Canadian High Arctic with the use of ice platforms. In the usual situation the ice platform is built through successive floods of ocean water to provide the required buoyancy and strength to support the 1630 tonne and greater long-term offshore rig loads. However, over the past three years the magnitude of the loading has increased substantially with the result that the construction of the platforms required more and more time. Since the drilling period is limited to the winter season, the increased construction period reduced the time available to complete the well. As a solution to this problem it was decided to provide partial platform buoyancy through the use of rigid urethane foam frozen into the central area of the platform which would allow for a reduced required ice thickness.

As a trial to the theories proposed to predict the performance of such platforms it was decided to perform a test program at an exploratory well drilled by Panarctic Oils Ltd. at Char G-07. The results from this test program and a further load case at Maclean I-72 are discussed in this paper.
INTRODUCTION

In the summer of 1979, Panarctic Oils Ltd. engaged FENCO CONSULTANTS LTD. to conceive and study methods which could be used to improve the construction and performance of arctic offshore ice platforms. These ice platforms have been used to support the heavy drilling rigs which are used to drill offshore exploratory wells. Since the commencement of the program in 1973, the trend has been to use larger rigs to drill to greater depths. At present, the largest rig operating in the Arctic Islands area imposes a long-term load on the ice platform of 1630 tonne. When it is noted that even the operation of a fully loaded flatbed truck, which weights approximately 40 tonne, requires caution over arctic ice, the scale of the rig load is realized.

In the usual situation of past years, simple flooding has been used to produce a tapered oval-shaped structure. Under reasonable weather conditions a flood was possible about every four hours, after which freezing was allowed to occur. In this way the required thickness was built layer by layer. However, with the rigs becoming heavier and heavier, it soon became apparent that the construction of the ice platform could not be completed by the mid-January date required to allow sufficient time to drill the well to total depth.

Since the very heavy weights on the rig were all necessary and no reduction was possible in these loads, it was proposed to remove some of the weight of the ice platform and thereby increase the ice platform load-carrying capacity. For this purpose, rigid urethane foam could be placed in the neutral axis area of the ice platform. Since the urethane foam has a usual density of approximately 30 kg/m$^3$ and ice has a density of 920 kg/m$^3$, for every m$^3$ of ice displaced by foam, 890 kg of additional load could be carried by the ice platform.

The above explanation is, however, a simple approach to a very complex time-dependent strain problem. In order to test the theories which were used to predict the performance of urethane foam platforms it was decided to freeze a known volume of foam into the platform at Char G-07 and study platform performance.
DESIGN CRITERIA

Essentially when an ice platform is designed, two criteria must be met.

1. The long term deflections must not exceed available freeboard.

2. Stresses due to any possible combination of long-term loads and short-term loads should not exceed 500 kPa.

At present the formula, as presented by Masterson et al [1], is used to predict long-term deflections, of a standard ice platform. This is a semi-empirical formula that uses directly the results of analysis of case histories. The equation is given by:

\[ W_1 = W_0 \left( \frac{h_0}{h_1} \right)^n \left( \frac{P_1}{P_0} \right)^m I_f \]  

(1)

where

- \( W_0, W_1 \) are deflections for previous load case 0 and design load case 1;
- \( h_0, h_1 \) are ice thicknesses for previous load case 0 and design load case 1;
- \( P_0, P_1 \) are loads for previous load case 0 and design load case 1;
- \( I_f \) represents a first-year ice factor of 1.25.
- \( n \) and \( m \) represent constants.

Long-term deflection data gathered since the start of the ice platform projects have indicated that approximately 25% more deflection will occur with a first-year ice platform relative to a multi-year ice platform.

The formula which was proposed for use with the urethane foamed ice platform is given by:

\[ W_1 = W_0 \left( \frac{h_0}{h_1} \right)^n \left( \frac{P_1 - P_f}{P_0} \right)^m I_f \]  

(2)

where \( P_f \) is the calculated reduction in weight of the ice platform through the use of the foam. This term is also referred to as the
foam uplift load as it is in effect an upward force which has been imposed on the platform at the neutral axis area.

A graphical presentation of equation (2) is presented in Figure (1) for two load cases. The 678 tonne load case represents the long-term load case for Rig 4 which drilled the exploratory well at Char G-07 and the 428 tonne load case represents the Rig 4 long-term load minus the 250 tonne uplift provided by the urethane foam. These predictions are based on two 678 tonne Rig 4 load cases which are indicated on the graph.

A problem which was foreseen with the use of equation (2) was that the foam uplift was applied to the ice platform approximately 30 days before the rig load was applied. However, it was also noted that the flooding on a four-hour basis was in fact imposing a downward load on the platform which would in effect partly balance out the foam uplift. During an average day, approximately 290 m$^3$ of water would be flooded onto the platform every four hours. This 290 tonne water load would granted be a frozen mass at the end of this four-hour period but for most of this period it is a viscous fluid.

Since equation (2) is an extension of equation (1) then the foam upward load should be applied at the same time that the rig load is applied. This, however, is not possible. But, as we have shown above, the water load should in part act like a platform load to balance out the foam load until the rig load is applied.

A second factor which also should be considered is that the rig load is also gradually imposed on the platform over approximately a 30-day period. If we consider the situation where an upward foam load $P_f$ is applied, then the situation illustrated in Figure (2) is achieved. The important point to note is that after 30 days of rig-loading, the platform would have returned to its deflection and stress level as if neither foam load nor rig load were present. Therefore after approximately 30 days of rig-loading, the platform should behave as if a load equal to the rig weight minus the foam uplift were present.
DATA ANALYSIS CHAR G-07 LOAD CASE

The initial load case, on which the urethane foam cells were employed, was on an ice platform from which the exploratory well Char G-07 was drilled. This load case was not an extremely heavy ice platform load and the foam was not necessary in order to complete the platform on schedule but the foam was used in order to test the theories as presented.

The initial surveys at the Char G-07 site indicated that the site was a first-year ice area and the ice platform was constructed to meet first-year ice requirements.

Prior to the load being applied on the Char G-07 platform, thickness and freeboard measurements were recorded at ten different stations over the design area of the platform. An average ice thickness of 5.01 m and an average freeboard of 0.685 m, which represents 13.7% of the average thickness, were recorded. Over the past years we have found that the completed ice platforms retain approximately 11% freeboard and deflection predictions from equations (1) and (2) are based on 11% freeboard. The excess freeboard is definitely due to the
presence of the urethane foam and it is estimated that the platform would rise to above the 13.7% level if the rig load were not imposed. At any rate the long-term deflection data for the Char platform are presented in Figure (3a). Also the predicted deflection below the 11% platform level is indicated. As can be seen, the degree of accuracy is very good but it should be emphasized that in usual platform work we get approximately 10% accuracy in long-term deflection predictions and that a certain amount of luck was involved in getting the near-exact prediction for Char. However, this does reinforce our confidence in the assumption that the platform up-lift provided through use of the urethane foam can be subtracted directly from the rig long-term load as far as long-term deflections are concerned. This means that deflections predicted through use of equation (2) are essentially correct.

\[ w = \phi \ t^m \]

where \( w \) is deflection, \( t \) is time, and \( m \) and \( \phi \) are constants.
The three possible types of deflection curves are illustrated in Figure (4).

If we complete a linear regression on the deflection time data over a constant load period, we can solve for the value of m. The following can be concluded about the value of m.

\[ m < 1 \text{ Primary curve decreasing deflection rate} \]
\[ m = 1 \text{ Secondary curve constant deflection rate} \]
\[ m > 1 \text{ Tertiary curve increasing deflection rate} \]

The calculated values for m for the Char G-07 platform are presented in Figure (3a). These values are based on deflection data for a particular day, plus the data from the nine previous days. Two important conclusions can be made about the values of m for the Char G-07 platform:

1. Only near day 30 did the value of m go above 1. This occurrence has been attributed to an increase in the ice platform load.

2. There exists a general downward trend in the value of m as time progresses. This fact is important during the initial stages of loading, e.g., if the value of m was above 1 but decreasing we would know that the platform would soon enter a safe platform deflection mode. However, if we observed a value of m greater than 1 and increasing, we would know that the platform is in a dangerous deflection mode.

DATA ANALYSIS MACLEAN I-72 LOAD CASE

Upon the successful completion of the Char G-07 test load case the urethane foam was used at the Maclean I-72 site to help support a
1630 tonne long-term load. At this site the load support created by the foam cells has played an important role in allowing the platform to be completed by early January. An area of concern which was not studied during the Char G-07 load case was the deflection of the ice platform during the final flooding stages and initial loading of the platform. For this purpose a float gauge was installed at Maclean on December 30, 1980, and the results are presented in Figure (5). Indicated on the graph are the final flood and start of rig-up times. Important points to note are:

1. During two periods of 'no flooding' on December 30 and after the last flood, the platform was seen to rise 2 to 4 mm.

2. The exact time of the start of rig-up is not exactly known other than that it was during the final hours of January 1, 1981.
URETHANE FOAM CELL CONSTRUCTION CHAR G-07

The foam cells were constructed by Engineered Urethanes Ltd. for the Char G-07 platform during the period November 23, 1979 to December 7, 1979. The equipment and chemicals to produce the urethane foam were mobilized from Edmonton and transported from Edmonton to the Char G-07 site by aircraft. The reasons for selecting the urethane foam to construct the cells were as follows.

1. The equipment required weighs less than 570 kg and therefore is easily aircraft transportable.

2. The manufactured foam has a density of approximately 30 kg/m$^3$ and therefore the entire 275 m$^3$ of foam requires only 8250 kg of material.

3. The manufactured foam has a closed cell structure and therefore does not allow water penetration.

4. The manufactured foam is flammable and therefore during the summer after the foam is used, crews can dispose of the foam through burning.

The foam cells were built in a 6 m X 12 m Hypalon clad prefab with plywood over planks, and 2 in. pearl board insulation for flooring. Heat was supplied by a 1,000 BTU Lister Air Heater (external). A single vent located mid-span of the building, six feet off the floor, serviced the shop. Electric power was 208 V, 3-phase/110 V 20 Kw (60 amps) service. Two air compressors, each rated at 8.6 c/m at 110 psi, provided air for the Gusmer H-II proportioner, Graco transfer pumps and fresh air for the foam mechanic.

The CIL rigid urethane foam required for the manufacture of the foam cells was processed using a Gusmer H-II proportioning unit and Gusmer Model D gun with #70 tip. The H-II uses a double acting counter opposed positive feed pump to proportion the isocyanate to resin ratio at 1:1 by weight.
In-line heaters maintained the temperature of the chemicals measured at the gun between 21.5°C and 29.5°C. The block primary heaters were run at 27.5°C to 32.2°C. Graco 2:1 air actuated transfer pumps feed the chemicals from the drums to the H-II.

Maximum flow rate obtained by the H-II was 3.64 kg/min, although with the D gun and #70 tip the H-II is capable of 5.5 kg/min to 7.3 kg/min. The reason for the poor rate was the high viscosity of the cold chemicals. The surface temperatures of the chemicals in the drums was 9.1°C, while the bottoms of the drums (3 in. off the floor) were -4.4°C. The chemicals should be stored at about 18°C. Below this temperature, besides causing feed problems due to high viscosity, the chemicals will begin to break down, their various components separate and poor quality foam results.

Typically the H-II unit generates pressures of 850 psi in the chemical lines to the gun, but because of the cold chemicals the unit would only produce 450 - 650 psi.

The foam reaction generates a considerable amount of heat and care must be exercised that the reacted foam does not overheat. Because the heat being dissipated from previous layers increased the maximum temperature reached by the next layer applied, the temperature of the surface layer reached 160°C, foaming would be suspended and the block allowed to cool. Scorching, which destroys the foam structure, begins at 177°C and must be prevented. Rather than let the blocks cool down to room temperature (26°C) which took between 2½ to 3½ hours, the blocks were cooled to 120°C. This would take up to an hour at which time a single layer would be applied and the process would be repeated. It was observed that each consecutive layer raised the maximum temperature experienced by 22°C to 28°C. Typically, on a cool (room temperature) substrate, 3 to 4 layers or 60 cm of foam could be sprayed before cooling was required. Note, too, that the surface layer of foam would experience the maximum temperature of the block and that the foam reaches its maximum temperature within 15 minutes of spraying.
SUMMARY

The test load case at Char G-07 illustrated that the weight reduction of the ice platform through the use of foam could be assumed to be replaced by an equal amount of rig load as far as long-term deflections are concerned.

A study of the deflection mode at both the Char G-07 and Maclean I-72 ice platform load cases showed that the deflection rates at these sites was safe. A linear regression analysis proved to be a useful tool in determining whether the platform was in a safe deflection mode.

Careful monitoring and control of the urethane foam chemical proportions and temperatures was found to be necessary in order to produce a foam product of suitable quality.

ACKNOWLEDGEMENTS

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REFERENCES


Numerous designs ranging from sand fill and caisson retained islands to monopods, monocones, barges and ships, have been put forward as exploration systems for arctic conditions. All have certain restrictions, be they related to water depth, seasonality of operation, ice conditions or cost. The purpose of this study was to arrive at a solution for a year round, mobile platform for operation in water depths ranging from 30' to 50 meters. The design requirement was that the platform be capable of drilling 3 deviated wells without major resupply or, alternatively, for a single well case, to be able to achieve two moves during the short arctic open water season. For this latter requirement, the number of operations associated with moving the platform have been kept to a minimum.

A form of monocone is proposed with a base in the form of a series of radial "spokes". The area between the spokes are filled with sand which provides a large proportion of the mass required to resist the high incident ice loads.

1. Introduction

The encouraging results of the exploration activity in the Canadian Beaufort Sea have stimulated the development of numerous concepts of drilling and production systems for ice covered waters. In early 1979, Gulf Canada Resources Inc. provided the terms of reference for a conceptual engineering study for a
moveable gravity platform to be used initially as an exploratory drilling system in water depths ranging from 20 to 50 m. The underlying philosophy in these terms of reference was that every effort should be made to develop a system that has all season capability so that the technology needed in the production phase would be refined and proven during the exploratory phase.

Other requirements to be met as far as possible included:

a) Lightship draft was limited to 6m firstly to permit the tow from a West coast shipyard around Pt. Barrow in the near shore region not normally intruded by the summer Arctic pack and secondly to facilitate any dry dock repairs in the Beaufort Sea.

b) A minimum installation and removal time was desired to achieve two moves per year, weather permitting. With the considerable variability in the summer weather window, any measures that would permit these operations during moderate waves or partial ice cover would be incorporated.

c) Maximum use would be made of locally available fill materials (clay, sand and gravel).

d) Maximum base pressures were to be limited to 95kPa. Since it was decided at the outset that some form of dredging support would be required, removal of small quantities of unsuitable sea bed material would be possible.

2. The Concept

After assessing a number of alternatives, a form of steel monocone was put forward as being the most promising solution. A perspective view of the proposed concept is shown in Figure 1. Monocones are not new and have two major desirable features, viz, the relatively small area subjected to ice loading which minimizes the ice forces, and the conical shape which assists failure of the ice in bending. The shape put forward is a double angle 45°/70° cone, the 70° section providing a transition into a vertical cylindrical stem. In water depths up to 30 m, ice sheets or ridges fail in bending against the 45° cone where the angle is such as to give a pronounced failure pattern. In depths between 30 m and 40 m, failure is still in bending but the 70° cone provides a much less pronounced mechanism. In greater depths, failure is by crushing against the vertical faces of the stem.
OPEN BASE BETWEEN SPOKES & TANKS
SPLAYS FOR PROTECTION FROM ICE
PREPARED SAND MATTRESS
ANCHOR COMPARTMENTS (9 - ON UNDERSIDE SPOKE)
FLOW LINE TUNNELS (4 TOTAL)
POSSIBLE FUTURE CONCRETE RIM
DECK EQUIPMENT & MODULES
ENCLOSED DECK (STORAGE, WORKSHOP, CONTROL CONSOLE & ANCHOR WINCHES)
STEM
70° CONE CONE SECTION (STORAGE AREA UNDER)
45° CONE SPOKES – WATER BALLAST AREA
SAND BALLAST INNER PERIPHERAL TANKS
OPEN BASE BETWEEN SPOKES & TANKS WATER BALLAST AREAS
SPLAYS FOR PROTECTION FROM ICE OUTER PERIPHERAL TANKS
PREPARED SAND MATTRESS
ANCHOR COMPARTMENTS (9 - ON UNDERSIDE SPOKE)
FLOW LINE TUNNELS (4 TOTAL)
POSSIBLE FUTURE CONCRETE RIM
FIGURE 1 PERSPECTIVE OF MOBILE ARCTIC GRAVITY PLATFORM
The ice, whether in the form of sheets, ridges or rubble, generates large horizontal forces, which in previous designs have been resisted by the mass of the structure. A novel feature is put forward in this design by introducing a "spoke" configuration in the base, the spokes being themselves water ballast tanks which connect to circumferential ballast tanks on the base periphery. All compartments between the spokes and tanks are open, without base plates, and filled with sand ballast which is contained by the spokes to provide a significant proportion of the lateral resistance. In this manner, the structural mass, while still substantial, can be reduced.

The structure will sit on a prepared sand mattress, of which the thickness of which will depend on the strength of the underlying seabed materials. The spoke concept will not require accurate mattress preparation as this type of base will "bed itself down" by successive local failure of high spots. Figure 2 shows several key phases in the platform operation.

Setting and raising any monocone shape entails difficulty because of the instability created by the reduction in waterplane area in the cone section. In water depths up to about 30 m this can be overcome by setting the structure by tilting, but in greater depths this could not be achieved in a controlled manner. To achieve control, massive anchor blocks have been incorporated on the periphery of eight of the spokes. These are operated by deck winches and are used to "pull down" the structure in a level mode. This requires careful control during ballasting to maintain positive buoyancy while at the same time avoiding excessive heeling.

On set down, the open compartments between the spokes and tanks are filled with sand ballast from a nearby stockpile. Prior to raising, a proportion of this sand ballast is removed by a small dredge or airlift to minimize negative skin friction effects, the remainder staying on the bed after removal of the structure. To dissipate any suction forces which might be present under the base, a water jetting system has been incorporated which would be activated prior to raising. A further contingency could be to make use of a ballasted barge to assist in providing uplift. Because substantial ice forces do not exist in the summer, no sand ballast is required and this assists in possibly achieving two moves per season.

The drilling equipment and consumable items are based on a 3-well programme and two principal areas are used for storage; the deck and the cone section.
Bulk items such as mud, cement and fuel have been located in the cone in custom made silos. The remainder, and all drilling equipment, are either inside the deck structure or on top of the deck in modularized components.

The question of whether two moves can be accomplished in the summer season is essentially a function of the time taken to drill the summer well. Using the figure of 75 days, it is estimated that the two move operation can be accomplished in about 113 days. This, therefore, requires extension of the average season, which is approximately 100 days, and icebreaker support would appear to be essential.

3. Specific Design Problems

Of the numerous design considerations in a structure of this kind, the following five subjects appeared to dominate.

3.1 Ice Forces

The major problem of gravity structures in ice covered waters is the provision of adequate sliding resistance. The maximum resistance that can be mobilized is governed by the structure weight and the foundation properties. In this concept, overturning was not a problem because of the large base diameter to water depth ratio.

The design cycle consisted of initial sizing and a calculation of the ice forces resulting from three ice structure scenarios at the various setting depths. The structure dimensions were not optimized subsequent to these calculations because of the degree of uncertainty in the ice forces themselves. However base diameters from 137 to 172 m were considered before arriving at 161 m as the final figure.

The three ice loading scenarios considered were:

a) Uniformly 16 m thick ice sheet with unlimited driving force

b) First and multiyear ice ridges driven by a thick 1st year pack.

c) Rubble formations grounded on the structure and interacting with the first year pack.
In the above cases, the procedure followed was to compute the gross ice force components acting on the structure for the range of setting depths. The water depth and structure geometry influenced, to some degree, the geometry of the worst ice feature. As expected, the rubble dominated the shallow water-settings whereas the ridges dominated the deep.

One particular area of concern was the effect that a ridge keel would have if it became lodged between the spokes and either moved transversely against a spoke or, worse still, moved towards the cone, creating a wedging action. Rather than delve into the interaction mechanics at this early stage, it was arbitrarily decided to provide sufficient ice strengthening on all potentially exposed surfaces and to chamfer the sides of the spokes to assist any ice feature to ride up over the spoke.

With respect to localized loads, a curve of normal ice pressure versus loaded area was used. This is an empirical in-house relationship based upon the small to medium scale ice test results of others and some theoretical work on large scale ice pack pressures. The structural system adopted here for those areas subject to ice contact was a four element system, consisting of bulkheads at 3 meter centers, stringers at 1.5-2.0 meters, ribs at 0.5 meters and heavy plate.

3.2 Floating Stability

The most critical periods during which floating stability is a factor are the sea tow to the Beaufort Sea and setting/raising.

Preliminary stability computations using the Boeing Ship Hull Characteristics computer program confirmed that draft and metacentric values were satisfactory in both the lightship and fully laden conditions with all tanks deballasted.

The effect of wash in the openings between the spokes during tow cannot be readily evaluated without model tests and it is quite possible that a temporary bottom would be required to reduce drag.

Although setting by tilting was examined and found satisfactory up to water depths of 30 meters, the more preferred method is the level setting mode using anchors. For most monocone geometries, there is a gradual loss of stability with reduction in waterplane area until at some point the structure
passes from a condition of positive stability to a negative one. To counteract this instability the structure is essentially pulled down to the bottom using a multipoint anchor system as shown in figure 2. This requires careful co-ordination between the ballasting and winching operations so that positive buoyancy is maintained at all times. The critical element here is the capability of the anchors to keep a level setting under the action of wind and waves once the point of negative stability is reached. However, should the anchoring system fail, it would be impossible for the structure to overturn because the toe of the base would ground and prevent further overturning.

The raising operation is basically the reverse of the setting procedure with the added consideration of overcoming base suction and the skin friction of the sand ballast between the spokes. The structure does have sufficient buoyancy to raise with full ballast, but this method would result in an uncontrolled ascent once the cavities between the spokes were emptied. It is therefore proposed to partially dredge the sand ballast to reduce the frictional forces against the side walls. As a measure to reduce base suction, a jetting system has been incorporated into the base.

3.3 Soil Mechanics

The mobility requirement in this design precluded the use of skirts or piles to enhance foundation stability. This requirement and conservative lower bound values for the foundation (undrained shear strength of 24kPa) resulted in a base diameter considerably larger than that required for a fixed structure.

The structure will not be set directly onto the seabed material but founded on a mattress of good, clean, free-draining sand. The question to be determined for any location will be whether the mattress can be placed directly onto the seabed, or whether a surface layer has to be removed. It is expected that a thick mattress could increase the depth capability of this concept by 10 m to a total water depth of 60 m.

To achieve the highest possible base friction coefficient i.e., equal to the internal sand friction, some roughening in the form of limited size skirts or keys will be necessary. The configuration must allow suction to be relieved prior to raising.
a) SETTING & RAISING BY TILTING (POSSIBLE UP TO 30m DEPTHS)

b) SETTING & RAISING BY ANCHORS (POSSIBLE ALL WATER DEPTHS UP TO 50 m)

c) RANGE OF OPERATING DEPTHS SHOWING FILL PLACEMENT

d) SETTING CAISSON THROUGH CENTRAL SHAFT

e) FLOWLINE PULL AND CONNECTION

f) WELLHEAD CONFIGURATION AFTER RAISING

FIGURE 2
KEY PHASES OF MOBILE ARCTIC GRAVITY PLATFORM OPERATION
3.4 Equipment and Consumables

The requirement to store sufficient consumables to drill three deviated wells without major resupply was an attempt to ensure that, once a field was deemed commercial, it could be drilled and brought on production as quickly as possible. Whatever the final configuration of the production platform, it is likely that satellite subsea wells (single or cluster) will be required to reach the reservoir outside the limit of deviated wells. This platform concept will permit many wells to be drilled before the production facilities are on stream.

The cone section contains 64 bulk storage tanks having a total capacity of 3895 cubic meters. Thirty two tanks have been designated as bulk mud (barytes, bentonite) and the remaining 32 as cement, although all can be used for either purpose. The remaining sack materials and casing will be stored on deck. The total weight of mud, cement and chemicals for three wells is 7.7 Million Kilograms. A 10 months supply of diesel fuel amounting to 3225 cubic meters is also contained in the cone.

3.5 Subsea Wellheads

The final major requirement of this drilling system was the ability to install subsea wellheads and connect flowlines. Within the 12.5 m shaft inside diameter, it was possible to install only 3 wellheads with their associated TFL (through flow line) loops. It was contemplated that a bundle of three flowlines will be connected to a flowline skid which will be pulled under one of the platform base spokes. The skid will be designed to mate with the trees at seabed level. All wells will be controlled remotely using a multiplexed control system with sequenced hydraulic back-up. Divers will not be required for running, retrieving or servicing the equipment except for minor maintenance or inspection. Should a major workover involving tubing removal be necessary, this work can be performed from a floating vessel during the summer without the use of divers.

As shown in figure 2 a protective berm surrounds the wellheads and caisson after the platform is removed.
Conclusions

This preliminary investigation showed that although a moveable gravity concept is a viable one for the Beaufort Sea environment, more work is required to define the ice-structure interaction and the capability of the anchoring system during setting/raising. This study was jointly performed by Albery, Pullerits, Dickson & Associates, Peter Hatfield Ltd. and Triocean. The authors are grateful to Gulf Canada Resources for permission to publicize this concept.
Abstract

Information on the winter regime in Great Lakes boat harbors, and design criteria and recommendations for boat harbors piling and structures subject to ice are presented. Included are estimates of stationary and moving horizontal and vertical forces, types of constructions and materials, facility layouts to mitigate ice problems, and methods of ice suppression. Design recommendations are based on actual design experiences, field investigations in two hundred and fifty US and Canadian Great Lakes harbors over a period of years, and laboratory studies.

Introduction

Piling and harbor structures in northern areas are damaged by ice. Supporting and mooring pilings are pulled from harbor bottoms. Thermally induced ice expansive forces act on dockages not removed for the winter. Wood structures are abraded. Deicing systems to protect structures from ice damage fail when not carefully designed and maintained. The winter regimes are hostile environments challenging the technical abilities of marine engineers and contractors. Figure 1 shows pilings and harbor structures that have been damaged by ice.
Ice for purposes of this paper is primarily stationary lake ice. River ice, ice floes and sea ice are not specifically dealt with. They may present additional and somewhat different problems. Harbors are customarily built in sheltered areas away from large moving ice masses. From a structural design standpoint, brackish and sea ice in harbors should present problems no worse than those associated with sound lake ice.

The writer presented some data and design recommendations for boat harbor structures at the PDAC 79 Conference, Wortley [1]. This paper supplements that information and presents some new data, particularly with respect to uplift on pilings. A companion paper in these proceedings discusses design of floating dockages in ice conditions.

Review of Data on the Winter Regime in Great Lakes Boat Harbors

For the past five years measurements and observations have been made in about two hundred and fifty Great Lakes harbors. The winter climate is characterized by freezing index degree days between 300 and 1200°C, minimum temperatures of -40°C, and snowfall ranging from 1 to nearly 10 m.

The harbor ice may be sound and firm for its full depth, may contain pockets of trapped water and air, or may be soft and melting if buried under a heavy snow cover. Under pile supported docks, 1.4 m of sound hard ice has been encountered. In harbors where storm conditions break and blow the ice away, new ice forms and may characteristically be only 0.5 m or less throughout the winter. In general, 1 m of ice can be expected in Great Lakes harbors.

Because of thermal forces and water level fluctuations (Great Lakes seiche action), the ice cover is cracked. The extent of cracking varies from discrete ice plates refreezing together after a storm to hairline cracks in glare ice during cold
spells. Structures in the ice perforate it and cracks in sheets 50 cm to 75 cm thick are frequently seen connecting structures 5 to 10 m apart.

Harbor geometry causes ice cracking. Almost always there is a crack that parallels the shoreline of a harbor basin. When the harbor is long and narrow this shoreline crack will be a single crack down the center of the basin. It is believed the shoreline crack is the result of the ice sheet failing in bending near where the ice is grounded or shorefast.

During warm periods, expansion in the ice causes a small pressure ridge ice crack along the longitudinal axis of the basin or down aisles between rows of boat slips. An entire row of boat slips can be separated from the rest of the harbor ice cover by an encircling crack connecting the outer pilings or structures. These cracks are found to be wet or dry. Around individual pilings and gravity-type constructions an active wet crack is usually found. This crack relieves the ice plate uplift forces and may exist throughout the entire winter.

A Whitney portable thermometer with lead weight and graduated cable is used to measure water temperature with depth. The thermometer circuit has a thermally cycled and pre-aged thermistor bridge with a reading accuracy of 0.1°C.

Water temperatures in Great Lakes boat harbors are usually very near the ice melting point. In many harbors values above 0.2°C are rare. Also the water is well mixed and isothermal. Water in the range of 0.5° to 2.0°C occasionally is encountered in certain "warm" harbors (near power plants or perhaps fed by springs or rivers).

Thicknesses of ice, crack patterns, water temperatures, ice-related structure damages, and the general winter regime in boat harbors are found to be fairly consistent year after year at specific sites. This would suggest to the structure designer, that if he observes conditions at his site, he can design constructions that will withstand the winters (even if he does not have complete quantitative numbers and values for the design parameters). In the next sections some recommendations for design are given.

Design of Harbor Structures in Ice Using Suppression Methods

An effective method of dealing with ice in harbors is to suppress or eliminate it, usually through melting. Compressed air distributed to diffusers on the harbor bottom will bubble up to the underside of the ice sheet and melt it out. Figure 2 shows this type of melting action.
Ashton's [2] monograph presents an analytical model for ice suppression with compressed air. Wortley [1] presents a trial and error design procedure based on the monograph and applicable to boat harbors. The quantity of air required is a function of the depth and temperature of the harbor water, the ambient air temperature, the amount of snow cover, the wind conditions, and the reduced ice cover thickness that can be tolerated. A quantity in the range of 0.00003 to 0.00009 m$^2$s$^{-1}$ per meter of diffuser tube length is usually adequate for water 2 to 5 m deep. The larger value would be used for the Great Lakes where the water is near freezing and the ambient temperature might be -20°C.

Suppression systems in ice covered rivers are believed to be ineffective. River currents destroy or displace bubble patterns. Thermal mixing of river water is a natural process and a river that has formed an ice cover has already dissipated nearly all available heat; otherwise the cover would not form but be melted out.

Ice can be suppressed with propeller systems that agitate the water surface and cause circulation. These systems appear to work well in areas where the water is fairly warm, in the range of 0.5°C to 2.0°C. In colder water it appears that more suppression can be obtained with less expended energy by using compressed air on the bottom and natural bubble buoyancy.

Design of Harbor Structures in Ice Without Ice Suppression

Harbor structures not protected with ice suppression systems must be designed to withstand horizontal and vertical forces. At this time these forces can only be approximated. Based on observations of piling supported boat docks in protected harbors, where blocks of ice move about, design loads are significantly less than the crushing strength of ice. The applied forces are the result of a stable cover breaking up under wind and surge, and are not from a sustained ice floe. The blocks of ice exert impact loads on supporting pileings or dock cribs but do not crush on them. It appears from structures presently built, that horizontal forces from moving ice pieces do not exceed the mooring forces for which the docks have been designed.

Methods to estimate, or measured values for thermal thrusts on individual pilings, have not been published. Again, from observations in boat harbors, these forces
are believed to be less than mooring forces for which the docks have been designed. The deflection of flexible piling supporting docks will be a matter of centimeters and adequate allowance in all structural connections must be provided.

Free standing mooring pilings have been permanently deflected when located in harbor basins with confining vertical sheet pile bulkheading. At Milwaukee, Wisconsin, concrete-filled 30 cm diameter pipe piles were deflected shoreward about \( \frac{1}{2} \) m. The piles are embedded 6 m into a firm silt and clay bottom and stand in 3 m of water.

Gravity-type crib structures will experience lateral shoving and must be designed to withstand the thermal forces. Laboratory studies by Drouin and Michel [3] have measured values for these forces. Although the work is a laboratory study, it is believed pertinent to the design of harbor structures.

Because of cracks, faults and discontinuities, field ice will be weaker than laboratory ice. Additionally, any snow on the ice will reduce the thermal responsiveness of the sheet. Thin ice is not capable of exerting significant thrusts. It buckles first. Thick ice tends to be self-insulating, i.e. the effects of a sustained temperature rise are rapidly attenuated with depth in the sheet. Therefore, thickness of the ice is not a critical factor in estimating thermal forces.

For the above reasons and based on observations of cribs in the Great Lakes a design value of 150 kN/m is recommended for thermal thrust on gravity type crib structures. Values one-half as much would be appropriate in areas with large snowfalls or weak unsound ice. On the other hand, 300 kN/m would be an appropriate estimate for clear ice, in a very confined boat harbor (without sloping banks) and under an unusually warm period following very cold weather.

In the next sections some recommendations for uplift forces and design against them are given.

Design for Uplift on Harbor Structures

In a harbor, pilings and other structures frozen into the ice cover will experience vertical forces from water level fluctuations. The case of most concern is a water level rise which lifts pilings from the bottom causing great damage. When the water (and ice) rises, either the piles embedded therein, are pulled from the bottom or the ice slips or fails near the piling. If the pile is lifted, the soil at the tip of the pile sloughs into the void created. When the water level recedes, the piling cannot return to its former depth. The ice eventually breaks away from the piling, drops, and refreezes at a lower level to the "jacked" pile. Figure 3 shows a single boat pier severely damaged from vertical ice forces. Entire marinas and docks have been similarly lifted.
However, when large water level drops occur, the ice loses all buoyancy and becomes a hanging dead weight spanning between "supporting pilings". Pilings should be designed for this full dead weight applied as an ultimate load.

In tidal cases, cyclic water levels can coat pilings and structural members, particularly inclined or horizontal bracing. In a manner similar to dipping a candle in wax, large accumulations of heavy ice can form and cause bending and shear failures in the harbor structure. Wind driven spray can similarly cause large accumulations. Problems of this type can be minimized by not framing structural members too close to one another.

Estimates of minimum ice uplift loads can be derived theoretically from a first crack elastic analysis of an infinite, floating, thin, homogeneous ice plate pierced by a round structure. The differential equation formulating this problem has been solved [4, 5] for boundary conditions describing a circumferential crack located a distance out from the center of the piling. When an ice sheet pulls upward on a strong well embedded piling, a circumferential crack does occur. For a steel piling this crack is usually 15 cm out from the face of the piling, and somewhat less for a wood piling. The ice is thicker next to the piling because of heat transfer through the piling. An ice collar forms around the piling.

The minimum ice sheet uplift loads for strong sound lake ice having an assumed flexural strength of 1500 kN/m² range between 30 kN for a circumferential crack 25 cm out from the center of a pile in 25 cm of ice to 300 kN for a 75 cm circumferential crack in 75 cm of ice.

In addition to the circumferential crack near the face of the piling, there are radial cracks. A more severe failure criterion by Nevel [4], with radial cracking and additional circumferential cracking at the ends of the radial cracks, gives a near, if not maximum upper bound to the problem. This cracking pattern forms a series of truncated wedges whose tips are supporting the load and whose bases are failure planes when the circumferential crack develops. The uplift loads computed from the truncated wedges criterion are 3 to 5 times the first crack criterion loads for the range of ice thicknesses and radii of load distributions found in boat harbors.
The above analyses predict a range of ultimate loads on single round piles assumed completely frozen into an infinite, thin, floating, homogeneous ice plate. These assumptions are questionable in light of what is observed in boat harbors. For example, Figure 4 shows two half cross sections of the ice attachment (collar) to a piling and the shapes of the ice sheet. Each cross section is representative of the ice-piling connection around the pilings. It does not matter whether the piling is round or square or tapered. The connection is not complete or for the full depth of the ice, but rather an open joint or plane of weakness exists. The joint freezes closed; and cracks open and is wetted when the ice plate responds to water level fluctuations. The clear ice zones along the plane of weakness indicate the water has refrozen slowly in this zone. Once formed, these joints may "work" all winter.

Methods to Reduce Pile Uplift

The following methods are suggested and recommended for consideration and trial. Some of them have overcome ice uplift, some have not been completely tried or reported on, and some are speculative and not proven or quantified. Nevertheless, they are offered to designers and builders with the hope that they will be helpful in solving ice vs marine pilings problems.

a) Suppress and Weaken Ice Sheet: Compressed air and velocity systems are discussed above. Chemicals, coal dust, waves and mechanical vibrations, and chopping can be used to weaken or eliminate ice.

b) Fail Ice Sheet: Closely spacing piles will tend to perforate an ice sheet causing it to crack and relieve uplift. Piles on extremities of a pier structure see or feel several times more uplift than interior supports. These outside piles should be given extra resistance to uplift and thereby cause the ice sheet to crack about them. Gravity structures not supported on pilings may be appropriate in moderate water depths. Ice will crack rather than lift them.

c) Provide Slip Joint in Pile: Sleeved piles have been used successfully to prevent uplift. A piece of pipe pile with an internal bearing plate
at mid-length is sleeved over a smaller driven pipe pile whose top is below the bottom surface of the ice sheet. The sleeve moves up and down with the ice sheet without pulling on the pile. The docks are framed onto the tops of the sleeves with structural connections and details allowing for irregular vertical displacements throughout the length of the dock.

d) Provide Pliable Material Around Pile: Piles have been surrounded with metal drums and other retainers forming annular spaces backfilled with grease, low shear strength materials, fuel oil (which floats on water but is unacceptable for maintaining water quality), etc. The result is that the surrounded pile experiences little or no uplift force. Piles wrapped with polyethylene sheets have proven unsuccessful as the sheets become ripped and torn.

e) Reduce Pile Surface Area at Ice Line: Since the ice rarely slips directly on the pile material surface, but rather fails along an ice-to-ice plane in the sheet, tapering a pile at the ice line does little to reduce uplift. (An exception would be if the pile were coated with a substance on which ice would slip.) However, if the diameter or area of the pile at the ice line is as small as possible, the uplift force will be smaller. This can be accomplished by variable cross section piles; with the larger sections located where bending stresses are high and the smaller sections where the ice grips the pile.

f) Use Treated Pile: Piles treated with pentachlorophenol and creosote appear to experience less uplift. These treatments will in time lose effectiveness as the pile surface becomes abraded by the ice. Greased piles similarly will not last.

g) Use Insulated or Heat-Pipe Piles: Pipe piles filled with vermiculite or other insulation appear to reduce ice adhesion in pull-out tests of insulated pile pieces extracted from ice sheets. Marine piles using geothermal heat-pipe principles may work but have not been reported.

h) Develop Pile Skin Resistance: If site conditions are appropriate, this method can be very effective. Since analytical methods to determine pile resistance to uplift require complete geotechnical information and are in themselves subject to considerable error, pile extraction tests are recommended. Preliminary estimates of required pile penetration can be made assuming skin friction equal to about one-third of the submerged unit soil weight for granular deposits, and about equal to the undrained shear strength of cohesive deposits. Penetrations less than 6 to 8 m are usually unsuccessful whereas long piles (15 m or so) have not been extracted. Obviously many factors come into play--piles are very site specific and extraction tests are recommended.

Skin friction should be calculated on the pile perimeter which is the core bounding area for non-circular shapes, i.e. for an H-pile, the perimeter is roughly four times the pile size, and not the pile surface area per se. A round shape maximizes effective surface area and is desirable.

Displacement piles driven with conical points will least disturb soil formations and provide greatest skin resistance. If wood piles are used they should be driven butt-end down.
Rock anchor piles socketed into competent formations have successfully resisted ice uplift. Deadmen and earth anchors also may be appropriate at certain sites.

Spiles or barbed piles are successful but methods of design remain to be quantified. The principle involved is to equip the pile shaft with a hook or barb. When the ice pulls, the pile offers more resistance.

Some successful examples are listed below: 25 cm bearing H-pile with two 25 mm plate thru-the-web barbs spaced 1.5 m apart driven 3.5 m into fractured limestone; and 15 cm pipe with four 6 cm angle pieces, 25 cm long, welded 30° off vertical on the lower end of a pipe pile with about 3 m penetration into sand and gravel.

i) Reduce Ice-Pile Adhesion Force: This is one of the most promising methods to prevent damage to pilings from ice. Some methods tried or proposed are listed below. Much of the research on coating methods is being done by the Cold Regions Research and Engineering Laboratories (CRREL) in Hanover, New Hampshire.

H-piles have been jacketed with round PVC split-shells and the annular space backfilled with concrete, in a manner similar to standard methods used to repair damaged marine piles. The H-piles at the test site had not previously been uplifted. The PVC jackets resulted in ice slipping on the jacket surface rather than failing along a plane in the ice sheet. Force measurements were not made but clearly the H-pile felt less uplift with its PVC jacket. Additional work is underway.

Laboratory studies with epoxy treated and untreated model piles have demonstrated five fold decreases in recorded force measurements on treated piles cyclically moved up and down in constraining ice sheets. Some field tests have been performed where pile pieces were frozen into ice sheets and subsequently jacked free. Epoxy coatings were experimented with and gave some indications of reduced ice adhesion.

A mixture of silicone oil and toluene, and a long chain copolymer compound of polycarbonates and polysiloxanes successfully reduced (but not eliminated) ice adhesion between a concrete lock wall at Sault Ste Marie and ice.

Ice breaker hull coatings are being developed and should have application for marine pilings. Some bulkheads in military installations have been coated for corrosion with these ship hull coatings.

Conclusions

Ice suppression systems can be designed to eliminate ice and its forces. Also ice forces on marine structures in protected boat harbors can be estimated. Ice thickness of one meter can be expected in the Great Lakes. Water temperatures in boat harbors are isothermal with depth and are usually 0.2°C or less. Lake seiches cause constant fluctuations of ice covers. The full dead weight of ice can be assumed as an ultimate load condition on pilings. Uplift forces jack pilings from the bottom. Estimated minimum value for uplift forces have been computed from a first crack elastic analysis of a floating ice plate. Some methods to reduce ice forces on pilings have been reviewed. Lateral ice forces on pilings are believed to be less than mooring forces from wind and boat impacts.
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References


OFFSHORE STRUCTURES ON WEAK FOUNDATIONS
EXPOSED TO LARGE ICE FORCES

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ABSTRACT

Almost throughout the length of the St. Lawrence Waterway, the bedrock is covered with soil deposits; some of them like marine clay, also known as Leda clay, or fresh water deposits have an extreme low bearing capacity. At the same time offshore structures in the St. Lawrence Waterway are exposed to very large ice forces. The early construction of offshore lighthouses on such foundation resulted in their tilting, deformation and the numerous structural failures.

Early in the 1960's, new methods of design and construction, using advanced soil mechanics, improved knowledge about ice forces and construction technology. Special elements, such as sloped surfaces, cylindrical and conical shapes have been incorporated in the structures to reduce ice forces. Special pile clusters, floating pile and raft foundation types have been used to overcome stability problems caused by the weak foundation. During the last two decades some 40 lightpiers have been built successfully on weak foundations, 20 of them on very weak marine clay. Furthermore, artificial islands to support light towers have been developed.

INTRODUCTION

General soil conditions

The St. Lawrence Gulf, the St. Lawrence River and the Great Lakes make the 4,300 km (2,700 miles) of the St. Lawrence Waterway. All this area was once covered and depressed by several glaciers. Later a large part of it, almost to Lake Ontario, was inundated by the sea.

Glaciers deposited sands, silts, clays and gravel. A compact mixture of these four is frequently very dense and has a high bearing strength. For the purpose of this paper, glacier deposits are considered to be a good foundation.
In the St. Lawrence Waterway bare rock is exposed only in a few places. When the glaciers receded, sea water invaded the depressed area and deposited clay sediments in lower locations. These are known as marine or Leda clays and have very low shear resistance and are very compressible. These clays have an extreme low bearing capacity and make a very weak foundation for structures.

In the Great Lakes' area, i.e., St. Clair River and St. Clair Lake, soft clays was deposited in fresh water, but their geotechnical properties are similar to those of marine clays.

Ice forces

Because the river and lakes are very large and winters are very cold, all offshore structures in the St. Lawrence Waterway are exposed to large ice forces. These may be caused by thermal expansion of the ice, by wind-induced drag or by the impact of the floating ice.

During the last twenty years the Canadian Coast Guard of Transport Canada has built almost 60 offshore lightpiers. This paper is based on experience in the design and construction of these lightpiers as well as on a study of the behaviour of some 20 old lightpiers and several bridges.

Since 1959 Marine Aids of Canadian Coast Guard has used the following formula to calculate the ice impact force $P$ on the structure [4 and 3]

![FIG. 1. General plan of the St. Lawrence Waterway.](image-url)
\[ P = m \ n \ b \ h \ q \]

where \( m \), shape and contact coefficient, from 0.9 to 04; \( n \), slope coefficient, depending on angle; if friction is neglected, then \( n \) is \( \cos A \) for a force perpendicular to the surface, \( \cos^2 A \) for the horizontal component and \( \cos A \sin A \) for the vertical component of the force; \( A \) is slope angle with the vertical; \( b \), projected width of the structure; \( h \), effective thickness of ice sheet and \( q \), effective compressive strength of ice \([4,2]\).

The design thickness has been assumed to be from 0.6-1.2 m (2-4 ft) as a solid homogeneous ice sheet. Thus for the design ice-crushing strength of 17.6 kg/cm\(^2\) (250 lb/in\(^2\)), the ice force on a vertical face of a structure is from 108-216 t/m (72,000-144,000 lbs/ft).

**OLD LIGHTPIERS**

Already in the 19th century it was very important for fast-growing Montreal that large ships from overseas could navigate to the city. The greatest obstacle was a shallow lake, Lac St. Pierre downstream from Montreal (Fig.1). In order to provide better navigational aids, in 1905 construction of four major lightpiers in the lake begun. However, failures of two of them occurred immediately.

One of these lightpiers was built at Curve No.3, opposite Point du Lac and three others at Curve No.2, opposite Louiseville (Figs. 2 and 3). The lightpier at Curve No.2, on the edge of navigation channel, built in a water depth of 9 m (30 ft) at low water (Fig.3), was tilted about 2.1 m (7 ft) in the downstream direction during the first winter.

The lightpier at Curve No.3, built in a water depth of 6.3 m (21 ft), was almost overturned by ice pressure and had to be abandoned. A new one was built 550 m (1,800 ft) closer to the shore in shallow water, 1.8 m (6 ft) at low water. But design of this one was different from its predecessor. The substructure was built only to the average water level in summer and a tower supporting a housing for navigational aids was made a removable steel tower. In spring, after the ice was gone, the steel tower with housing was placed on the concrete cap of the substructure and removed before beginning of winter (Fig.2).

The badly-tilted lightpier at Curve No.2 was not completely demolished; only the superstructure and part of the substructure were removed. A concrete cap was placed to the summer water level and, as with the lightpier at Curve No.3, a removable steel tower provided. As the concrete cap was placed below winter water levels, ice in spring would float over the pier and no, or very little, ice pressure would be exerted on the pier [2]. These failures discouraged further construction in Lac St. Pierre for the next 50 years.

From the available records, it appears that the first lightpiers in Lac St. Pierre were built on 15 x 15 m (45 x 45 ft) timber cribworks filled with
FIG. 2 (above). Lac St. Pierre, Curve No. 3, Point du Lac Range Front Light Pier built in 1909 to replace one almost overturned by ice.

FIG. 3 (right). Lac St. Pierre, Curve No. 2. Front light pier of a range built in 1905 and rebuilt after extensive damage by ice in 1906.

stones. The cribwork was founded on timber piles, 12-13.5 m (40-45 ft) long. The concrete substructure exposed to ice pressure was relatively large, approx. 12 x 12 m (40 x 40) ft.

The total or partial failures in Lac St. Pierre were caused by a combination of heavy ice forces and the weak marine clay foundation, both of which at that time could not be accurately evaluated. At that time there was a complete lack of knowledge of the geotechnical properties of soft marine clay. The very sparse remaining records of 1905 describe the soil at Curve No. 2 as very soft and weak because a steel rod could be driven by a sledge hammer for some 21 m (70 ft). Furthermore, earlier it had been necessary to build rather large substructures to accommodate bulky equipment. With technological progress, the lights became electric, small in size; much smaller piers exposing small areas for ice thrust could be built.

LIGHTPIERS ON TIMBER CRIBWORK

For many years timber cribworks were extensively used for marine structures, small dams, cofferdams as well as for the substructures of offshore lightpiers [2,5]. Only late in the fifties did steel sheet piling and concrete almost completely replace cribwork.

Many old lightpiers with cribwork as a base were considerably damaged, tilted, deformed and partially displaced but they still could be used to support the light and other navigational aids. Of course, frequent adjustment was required to correct alignment and verticality. From structural point of view, the flexibility of the cribwork allowed it to withstand greater impact forces. A timber cribwork with stone fill can be deformed to a rather large degree without losing its complete structural strength. Among the shortcomings of timber cribworks are susceptibility for decay at the water line, lack of rigidity and limited strength. Their advantages for construction were availability of timber, low cost, no need for skilled labour and floating capability.

Early in sixties a lightpier on a cribwork in Lake St. Clair at the mouth of the St. Clair River was still designed and built (Fig.4) [2, 6].

The lake bottom at the site was covered with a layer of loose sand 2 m (7 ft) thick which was followed by 24 m (80 ft) of very weak clay. A timber cribwork supported by friction timber piles was chose for the lower part of the substructure. A number of innovations were introduced in designing this lightpier: (a) a slender reinforced concrete cone was anchored into the cribwork;
slender cone exposed small area to ice thrust, (b) the top of the timber cribwork was placed well below the low water level in order not to expose it to ice thrust, and protect the timber from decay, (c) timber sheet piling was driven around the crib to protect the foundation from underscouring; but first this sheet piling was driven only a part of the total depth and was used as a cofferdam, inside which the construction could be done in dry.

CONTEMPORARY LIGHTPIERS IN LAC ST. PIERRE ON FRICITION PILES

By the end of the fifties, navigation to Montreal considerably increased and at the same time the ships and lake freighters became much larger. It was very important for Montreal Harbour to extend the navigation season so that minimum time be lost because of winter [7]. The need for fixed reliable navigation aids—lightpiers became very great for ships and even for the icebreakers in Lac St. Pierre, the critical section for winter navigation to Montreal.

As the lightpiers in Lac St. Pierre had to serve range lights indicating the centre line of the navigation channel, the stability requirements were rather strict. Only a very small lateral movement and differential settlement was allowable, and no great general settlement desired. Because of the large ice impact forces and weak foundation it was necessary to develop a structure which would be light and would expose small areas to ice thrust. At the same time the structure had to be massive and sturdy enough to withstand ice battering and remain free of dangerous vibration.

First a timber cribwork on timber piles (a structure similar to St. Clair lightpier LL 721) (Fig. 4) was considered but it was found that a reinforced concrete slab over the piles for the existing conditions was a better solution for overall consideration. Fig. 5 shows the design of one of the modern lightpiers for West Range at Yamachiche Bend in Lac St. Pierre which together with the front lightpier was built in 1972. After the successful construction of the first two lightpiers, later five more lightpiers of the same design was built in the same lake. The substructures of some of these have 45° cones. The lightpier shown in Fig. 5 has a large concrete slab over 208 timber piles 19 m (63.5 ft) long and 25 cm (10 in) in diameter. The large concrete slab was required to distribute the load over the piles; as these are friction piles, the adhesion force between clay and a pile dictated the allowable bearing capacity of a pile. The pile can take only a limited horizontal force; therefore the number of piles was also governed by the maximum horizontal ice force. A detailed description of the design and construction of this type of lightpier is in References 7 and 4.

Other problems to be solved included uneven settlement, remoulding of the clay while driving the piles: a loss of shear strength and its recovery, drag force on piles, and prevention of a lateral movement of the structure. So far, none of the 7 lightpiers built between 1966 and 1972 have showed any differential settlement or lateral displacement; first two of them was built in 1966.

LIGHTPIERS ON PILES TO FIRM FOUNDATION

A deep pile foundation is preferable to a floating foundation and is always used if firm soil or bedrock can be economically reached. Fig. 6 shows a lightpier at Curve No. 3, Pointe du Lac in Lac St. Pierre built on steel-pipe piles filled with concrete; battered piles were used to take the large horizontal ice force. Furthermore, to reduce the ice force, the substructure is a cone with 45° sides. This lightpier was built in 1973 at the site where three lightpiers previously had complete or partial failures [2].

In Lake St. Francis (Fig. 1) 8 lightpiers have been built on steel "H" or steel pipe piles since 1966; piles up to 40 m (130 ft) were driven through marine clay to the bedrock or the firm glacial till [2].
In Lake St. Francis and Lake St. Louis several lightpiers have gravity-type foundations as they could be founded on very firm till. Their cost was about half the cost of similar lightpiers in Lac St. Pierre, built on friction piles in soft marine clay.

**LIGHTPIERS ON ARTIFICIAL ISLANDS**

Four artificial islands were built in 1967-68 in Lac St. Pierre to control ice movement. and on one of them Island No. 3 near Yamachiche Bend, a light tower was placed (Fig. 7). At that time this was the first attempt to build such islands.
a structure on very soft deep marine clay. Its design was quite elaborate because of a very weak foundation; the island was designed with two berms and varying slopes. Such shape is good not only to spread the load but also is favourable to reduce ice pressures against the island.

If the foundation is very poor, artificial islands are economical only in shallow water. Island no. 3 at Yamachiche in Lac St. Pierre was built only at half of the cost of a concrete lightpier on piles. Such islands require maintenance work because of foundation settlement and erosion by ice, waves and swells. A detailed description of the design and construction may be found in Reference 1.

Another island to support a light tower was built at Lancaster Bar in Lake St. Francis.

LIGHTPIERS WITH PROTECTIVE STONE BERMS

Fig. 8 shows a small lightpier on a weak foundation in very shallow water. The concrete substructure supporting a tower is surrounded by stone berms which would ground the ice and protect the tower from ice impact. Nine such structures were built along the New Southeast Navigation Channel at the outlet of St. Clair Lake. Such structures require continuous observation and maintenance of the stone berms. At the two of these structures the berms were eroded and the ice thrust tilted the towers.

GENERAL COMMENTS

To overcome the twofold problem of weak foundations and large ice forces it is necessary, first, to design a structure as light as is practical and with the least amount of area exposed to ice thrust. Shapes that reduce ice forces such as cylinders, cones and inclined surfaces should be incorporated, if possible, to cause ice failure by bending instead of by crushing or even in the case of failure by "crushing", to reduce the horizontal force and to increase the stabilizing vertical force.

But the structure and its elements should be of a massive type. If the structure or its members are too slender or too light, a dangerous vibration may
Ice crushing against the structure develops a cyclic load and may induce oscillation of the structure. Ice force per linear unit on a very narrow structure is much larger than on a wide structure, because of local concentration or, as it is sometimes called, because of the "indentation" factor.

Ice force is only one element in the design of these structures; sometimes a conical shape, although reducing the ice force, may increase the total cost of the structure. During the design stage of an offshore structure, great attention should be given to construction procedures. All construction operations and expensive floating equipment are exposed to inclement weather, currents, winds, waves and storms. Thus the actual amount of work at the site should be kept to the very minimum. The design should make possible a great degree of prefabrication of the components of the structure.

The choice of the type of foundation, deep piles, friction piles, raft foundation or open caisson, - depends very much on the expected behaviour of the structure. For a single lightpier a substantial settlement and even some differential settlement may be allowable because the light supporting tower can be adjusted for verticality. Also, a small lateral movement might be tolerated. But requirements for a lightpier supporting range light would be much stricter, and even more for wharves and docks.

Pile testing at an offshore site before construction often is impractical. The designer has to rely on laboratory and field geotechnical tests and may have to assume a somewhat higher safety factor because of many unknowns [7]. But the present status of soil mechanics can provide satisfactory design data.

REFERENCES


ICE RESISTANCE EQUATION FOR FIXED CONICAL STRUCTURES

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Abstract

A general form of ice resistance equation for an icebreaking ship is adapted to fixed, upbreaking, conical structures, such as those proposed for use in offshore Arctic petroleum exploration. This adapted general equation has three terms: (1) a term to account for breaking of the ice sheet into pieces; (2) a term to account for lifting of the broken pieces as they clear around the structure; and (3) a term to account for the inertial effects of the moving ice sheet. This equation encompasses most of the specific expressions used by other investigators.

Dimensional analysis yields relations between the exponents in each term of the equation. For a given set of exponents, the three coefficients of the adapted equation are evaluated using model test data and a least-squares-error criterion. The exponents are incremented through 13,671 possible combinations, and coefficients are calculated for each combination. The result is a three-dimensional array of least-errors with the values for one set of exponents along each axis. The set of exponents, and therefore coefficients, with the minimum least-error is selected as the "best-fit" solution for this data base.

The model test data are from 1:50 scale tests of a 45° conical structure, and the test variables include waterline diameter, ice flexural strength, ice thickness, and impingement velocity.
Introduction

A large conical-shaped structure is one alternative for bottom-founded, shallow-water installations in ice-infested waters. As sea ice impinges on such a structure, it will tend to slide up the conical sides. The lifted ice sheet then fails in flexure, and the broken pieces either fall or are pushed off the cone. The conical shape has received much attention because flexural failure occurs at a lower load than would crushing failure against a corresponding vertical cylinder.

To appreciate more fully the problem of estimating the load from ice impinging on a conical structure, the problem of estimating the resistance encountered by an icebreaking ship is introduced. This resistance is generally assumed to consist of several terms:

(a) the resistance due to breaking the ice
(b) the resistance due to submerging the broken ice
(c) the resistance due to the inertia of the ice
(d) the resistance due to friction between the ship's hull and the ice

Vance [1] presents a dimensionally-consistent equation that explicitly accounts for the first three of the above factors and implicitly accounts for the fourth:

\[ R = C_B \sigma_f h^{X_1} B^{Y_1} + C_S \rho_s g B h^{X_2} T^{Y_2} + C_V \rho_i V^2 L h^{X_3} B^{Y_3} \]  

(1)

where:
- \( C_B, C_S, \text{ and } C_V \) are dimensionless coefficients
- \( \sigma_f \) is ice flexural strength
- \( h \) is ice thickness
- \( B \) is ship's beam
- \( \rho_s \) is submerged unit mass of ice \((\rho_w - \rho_i)\)
- \( \rho_w \) is unit mass of water
- \( \rho_i \) is unit mass of ice
- \( g \) is gravitational acceleration
- \( T \) is depth to which the broken ice is submerged
- \( V \) is ship's speed
- \( L \) is ship's length

and \( X_1+Y_1=2; X_2+Y_2=2; X_3+Y_3=1 \) (from dimensional analysis)
There is by no means a universally accepted form of the resistance equation. Each investigator assigns different values to the exponents and coefficients, and some assume additional or fewer terms. Vance has shown that these three terms (breaking, submergence, and inertia, respectively) are the primary contributors to the resistance of icebreaking ships.

This paper will adapt Vance's general expression to an upbreaking, fixed, conical structure, and then determine a set of exponents and coefficients by comparison with model tests.

**Adaptation of General Equation**

**Breaking Term**

In the breaking term, the major adaptation is the use of cone waterline diameter in lieu of ship's beam. The analogy is obvious, and this adaptation is common to most cone resistance equations. A more subtle adaptation is the use of flexural strength from a tension failure in the bottom surface of the ice (upbreaking strength). For an icebreaking ship, the proper flexural strength is that from failure in the top surface of the ice (downbreaking strength). This adaptation is most critical for sea ice, where the bottom surface is generally the weakest part of the ice.

**Submergence Term**

The submergence term is renamed the removal term here, because with an upbreaking cone, the ice is forced up over the cone and is not submerged. This basic difference leads to the first adaptation; the use of the unit mass of ice in lieu of the submerged unit mass of ice. Again the waterline diameter replaces ship's beam, and the last adaptation to this term involves \( T \), the depth to which the broken ice is submerged by the icebreaking ship. As stated above, the ice is not submerged by a cone but slides up the cone and possibly the vertical cylindrical part of the structure (Figure 1). The removal term is therefore a function of the broken ice piece size and the average height to which the pieces ride up (friction will be discussed later).

The piece size is determined by the distance between successive circumferential cracks in the ice failure sequence, or is a function of the characteristic
Figure 1

1/50 Scale Model Conical Structure with Ice Impingement
length of the ice. Assume an elastic, infinite ice sheet floating on water:

\[ \ell = \left[ \frac{E h^3}{12 (1-\nu^2) \rho_w g} \right]^{1/4} \]  

(2)

where: \( \ell \) = characteristic length  
\( E \) = elastic modulus  
\( \nu \) = Poisson's ratio  

From Equation (2) and for a given ice sheet, piece size is a function of ice thickness.

For a given structure, the height to which pieces can slide up the cone is a function of waterline diameter (i.e., with a large waterline diameter, pieces have a greater distance, thus height, to slide up). With an eye on the practicality of expressing the resistance in terms of readily measurable parameters, \( T \) is therefore assumed to be a function of waterline diameter.

**Inertia Term**

The inertia or velocity term is adapted by equating ship's length to waterline diameter, and again equating ship's beam to waterline diameter.

**Adapted Expression**

The general, dimensionally consistent, resistance equation, as adapted for conical structures is:

\[ R = C_B \sigma_f h^{X_1} D^{Y_1} + C_R \rho_i g h^{X_2} D^{Y_2} + C_V \rho_i v^2 h^{X_3} D^{Y_3} \]  

(3)

where: \( D \) = cone waterline diameter  
\( X_1 + Y_1 = 2; X_2 + Y_2 = 3; X_3 + Y_3 = 2 \) (from dimensional analysis)

**Friction Effects**

Friction between the cone and the ice will add to the resistance. The magnitude of the friction force will be a function of the coefficient of friction.
between the cone surface and ice, and broken ice piece weight, which is dependent on piece size and unit weight of ice ($\rho_i g$). The removal term incorporates both ice piece size and the unit weight of ice. The frictional resistance could be approximated explicitly by adding a new factor to the removal term, $(1+\mu)$, where $\mu$ is the coefficient of friction. This was not done in this work because the coefficient of friction was not varied during the model tests. It is doubtful that friction can be accounted for this simply; in fact, friction probably contributes to all three terms with the greatest effect on the removal term and the least effect on the breaking term. If the coefficient of friction is assumed constant, the effects of friction are implicitly accounted for in the three coefficients.

Model Test Data

A 1:50 scale model of a 45° conical structure (Figure 1) has been tested in model ice in a test basin. The basin is 16 feet (4.87 m) long, 6 feet (1.83 m) wide, and 2 feet (0.61 m) deep. The model ice is grown from an aqueous solution, seeded with fine ice particles to initiate columnar ice with a small crystal size. The strength of the model ice is controlled by the concentration of the dopant in the solution, and thickness is controlled by the air temperature and duration of the growth. The model cone is drawn through 11 feet (3.35 m) of the basin length by an instrumented carriage. This instrumentation consists of load cells to measure horizontal and vertical force, and overturning moment. The measurements are recorded on a multichannel strip chart recorder. The carriage speed can be varied from zero up to about one inch (2.54 cm) per second. The cone waterline diameter is varied by changing the water depth in the basin.

As the ice impinges on the cone, the horizontal load increases from zero to a peak value. The first peak is associated with the formation of the first circumferential crack, and the load abruptly falls off after the crack forms. The load does not return to zero force, because the newly broken ice piece is being pushed up the cone. This sequence continues as successive circumferential cracks form and more broken pieces of ice are pushed up the cone. A "steady-state" condition is reached after about the fourth peak. This is when the first broken ice piece has fallen or has been pushed off the cone, and there is now a relatively constant number of ice pieces being pushed up the cone.
In this "steady state" condition, several values of peak force are obtained during each run, before tank-end effects are noticeable. The arithmetic mean of these several values is assumed to be the maximum horizontal force (resistance) for the particular ice thickness, ice flexural strength, carriage speed, and cone waterline diameter during that run. Ice thickness is assumed to be the arithmetic mean of ten physical measurements of ice thickness. Ice flexural strength is assumed to be the arithmetic mean of ten, simply-supported, downbreaking, submerged beam failure tests (the beams had a width-to-thickness ratio of about two and a length (between supports) to thickness ratio of about five. A line load was applied at mid-span, and the force necessary to cause flexural failure was measured. Using elastic beam theory and accounting for the submerged weight of the beam, the flexural strength was calculated). The average carriage speed is determined from the time necessary for the carriage to traverse the known length of the run.

This paper uses measured data from 47 runs at various ice thicknesses, ice flexural strengths, cone waterline diameters, and carriage speeds. The ranges of values determined from these 47 model test runs and used to calculate the coefficients and exponents of the adapted resistance equation are tabulated below:

<table>
<thead>
<tr>
<th></th>
<th>min.</th>
<th>ave.</th>
<th>max.</th>
</tr>
</thead>
<tbody>
<tr>
<td>D (in)</td>
<td>25.0 (63.5 cm)</td>
<td>30.4 (77.2 cm)</td>
<td>40.4 (102.6 cm)</td>
</tr>
<tr>
<td>$\sigma_f$ (psi)</td>
<td>1.49 (10.3 Kpa)</td>
<td>2.22 (15.3 Kpa)</td>
<td>3.23 (22.3 Kpa)</td>
</tr>
<tr>
<td>h (in)</td>
<td>1.40 (3.56 cm)</td>
<td>1.78 (4.52 cm)</td>
<td>2.33 (5.92 cm)</td>
</tr>
<tr>
<td>V (in/sec)</td>
<td>0.07 (0.18 cm/s)</td>
<td>0.32 (0.81 cm/s)</td>
<td>0.43 (1.09 cm/s)</td>
</tr>
</tbody>
</table>

The value for unit mass of ice ($\rho_i$) was assumed to be 0.0000862 lb-sec$^2$/in$^4$ (0.923 g/cm$^3$). The ratio of the elastic modulus to the flexural strength ($E/\sigma$) was about 500 for this data base.

**Data Analysis**

The first step in the data analysis is to determine limits on the coefficients and exponents. The only limit on coefficients used in this analysis was that they must not be less than zero, because negative coefficients were assumed physically unrealistic. For the exponents, the lower limit was that they not be less than zero for the same reason as negative coefficients (e.g., $x_l > 2.0$).
would require \( Y_l < 0.0 \), from dimensional analysis, and result in breaking resistance decreasing with increasing waterline diameter). The upper limits for the exponents are calculated from the lower limits and the exponent relationships presented with Equation (3).

Within the selected exponent limits, initial exponent values were selected (i.e., \( X_1 = 0 \), \( Y_1 = 2.0 \); \( X_2 = 0 \), \( Y_2 = 3.0 \); \( X_3 = 0 \), \( Y_3 = 2.0 \)), and the coefficients were determined based on a least-squares fitting of the predicted to the measured resistance. When these coefficients had been determined, one set of exponents was incremented by 0.1 (e.g., in the second step \( X_1 = 0.1 \), \( Y_1 = 1.9 \), and the other exponents were unchanged), and another set of coefficients was determined. This procedure was continued through all possible combinations of exponents, resulting in a three-dimensional array of results. The selected values of \( X_1 \), \( X_2 \), and \( X_3 \) describe the three axes of this array (21x31x21 = 13,671 elements). The element of this array with the minimum value of error from the least-squares criterion is assumed to have the best set of coefficients and exponents.

The best set of coefficients and exponents yields the following resistance equation:

\[
R = 0.284 \sigma_f h^2 + 5.47 \rho_i g h^{1.7} D^{1.3} + 797. \rho_i V^2 D^2
\]  

(4)

It is difficult to explain in physical terms why cone diameter does not appear in the breaking term and why ice thickness does not appear in the inertia term. The gradients of the errors in the vicinity of the minimum are very low, and inclusion of these two parameters does not significantly increase the error (e.g., with \( X_1 = 1.0 \) and \( Y_1 = 1.0 \), the error increases from 13.28% to 13.30%). Obviously different coefficients would be used with these different exponents.

The two-dimensional, sub-array with \( X_3 = 0.0 \) and which includes the above best result is shown in Figure 2. The values of the elements are the root-mean-square error in the predicted resistance expressed in parts per ten thousand. The elements for negative values of coefficients, that were disregarded, are indicated by 9999. Therefore, the best-fit resistance equation (Equation 4) has an RMS error of 13.28% in predicting the resistance of the 47 model runs. The correlation coefficient is 0.923.
Figure 2

Sub-Array for $X_3=0.0$ (Elements Are RMS Error in Parts Per Ten Thousand)
The relative contributions by the three terms to total resistance are 4%, 83%, and 13%, respectively. The contribution by the inertia term is larger than expected and may be a consequence of the data base. Thirty-four of the forty-seven velocity values were very similar, and this lack of distribution may be the cause of this unexpected result.

Also, the surface described in four-space by \( C_B, C_R, C_V, \) and \( R \) may be very nonlinear and may have negative slopes in the \( C_B, C_R, \) and/or \( C_V \) directions. It is possible that without the limits on the coefficient and exponents the "best-fit" plane of tangency, in the limited area of the surface described by this data base, may be a better fit than that determined with the limits.

**Conclusion**

This paper has presented one method for determining an ice resistance prediction equation. Many previous investigators have used the least-squares method to determine the coefficients, but only for a given set of exponents. The best-fit resistance equation with the particular coefficients and exponents (Equation 4) was determined from a limited data set derived from 1/50 scale model testing and is presented for illustrative purposes only. It should not be used for extrapolation beyond the ranges of the input data.

However, the basic resistance equation (Equation 3) and the method of determining coefficients and exponents can be used with any scale data. The limits, if any, to be put on the coefficients and exponents must be carefully reviewed in light of the data base available, the physical problem being described, and application of the results.

**Reference**

ON THE STATE OF COMMERCIAL
ARCTIC MARINE TRANSPORTATION

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ABSTRACT

This thematic paper on the subject of "Navigation in Cold Regions" gives the author's personal opinion as to why there are no significant arctic marine transportation systems in existence today, and suggests what might be done to change this state of affairs. Cargo transport demands which could have been satisfied by arctic marine transportation systems existed in the United States throughout the seventies and were eventually satisfied by pipelines. Similar future transport demands are imminent. Decision makers in the United States do not believe arctic marine transportation is either technically or economically feasible. The marine industry in the United States and probably throughout the world have not provided decision makers with any proof to the contrary. To prove that arctic marine transportation is technically and economically feasible, it is recommended that a 100,000 to 200,000 DWT, Class 10, arctic icebreaking tanker be constructed and operated year-round from Valdez, Alaska through the Northwest Passage to ports on the North American east coast.

INTRODUCTION

It has now been twelve years since the voyage of the SS MANHATTAN through the Northwest Passage. Since then, a large amount of research has been conducted on the subject of ship-ice interaction. Numerous cold regions laboratories and organizations have sprung up throughout the world and many people are now involved with this science. Still, in spite of all these efforts, it appears that they have had little impact in making commercial Arctic marine transportation a reality.

It would be easy enough to pass this situation off on misfortune if there were no cargo around which might be hauled by commercial arctic ships. But this is not the case. In the United States alone, oil and gas already discovered and likely to be
discovered in and around Alaska present a pressing cargo transportation demand which could be met by arctic marine transportation systems. Billions of dollars have already been spent on transporting crude oil from Alaska and billions more are currently being committed to transporting the gas. All of this cargo is presently being transported or planned to be transported by pipelines. In the case of the crude oil transportation, an arctic marine transportation system was seriously considered in the MANHATTAN project. In the case of the recent gas transportation problem, it is questionable whether arctic marine transportation was very seriously considered.

Those of us involved with marine transportation know that carrying cargo by ships almost always results in the least unit cargo transportation costs when a water route is available. Why is it then that there are no significant arctic marine transportation systems in the world today? Is it because we have not really done our homework? Is it because commercial arctic ships are really uneconomic? Or is it because we have not convinced decision makers that commercial arctic shipping is technically feasible? Let me provide some background information which might provide some answers to these questions.

BACKGROUND

In early 1969, a demand arose in the United States to transport raw energy (crude oil) from Prudhoe Bay, Alaska to the contiguous 48 states. A number of alternative transportation systems were considered and studied in some detail. Seriously considered alternatives included a pipeline from Prudhoe through Canada to the United States; a combination pipeline from Prudhoe to an ice-free port in Alaska, and thence via conventional tankers to the United States; and, a completely marine system utilizing icebreaking tankers. Other alternatives considered included submarine tankers, trucks, rail, airplanes, and even conversion of the crude to electrical energy at Prudhoe and then transmission of the electrical energy to the United States.

Numerous factors were considered in determining the risks and costs of each alternative transportation system. These factors included environmental impacts, political and social impacts, existence of partial transportation system components, experience with components of each alternative system, and many more. After considerable analyses involving very large expenditures of funds, the decision seemed to boil down to a choice between an all marine system employing arctic icebreaking tankers and the combination pipeline-tanker system [1]. The combination pipeline-tanker system was finally selected. The basis for this decision to my knowledge has never been published, but I believe it had much to do with the risks associated with arctic ice-breaking tankers which, at that time, were believed far greater than those associated with the combination system. Having been personally involved with this project from the arctic marine transportation system side, I think it is fair to say the marine
transportation industry was ill prepared at the time and consequently could not convince decision makers that the risks associated with an arctic marine transportation system were acceptable.

The Prudhoe Bay to Valdez pipeline met the crude oil transportation demand existing at that time but not the gas transportation demand. An act was passed by the U.S. Congress in the mid-1970's that required the Federal Energy Commission to evaluate alternatives for transporting Alaskan natural gas to the contiguous 48 states and to recommend a system to the President by May 1, 1977 [2]. The Commission recommended that the President adopt a proposal from El Paso Natural Gas Company which involved a pipeline running through Canada to the continental United States. On September 8, 1977, President Carter and Prime Minister Trudeau jointly announced that they had selected an overland gas pipeline route proposed by ALCAN and on November 8, 1977, the U.S. Congress passed Public Law 95-158 approving the presidential decision. That pipeline will cost in excess of 20 billion dollars according to current estimates.

How seriously was the arctic marine transportation alternative considered? Why was the pipeline alternative selected over the arctic marine alternative? The following quotes from a statement made by a U.S. Department of Energy spokesman [3] in May, 1980, to the U.S. House of Representatives' Committee on Merchant Marine and Fisheries may provide the answers (sentences in italics are mine):

"The options for delivering arctic gas supplies to the United States via marine systems can be described as (1) those transportation systems using surface tankers with icebreaking capabilities, and (2) those transportation systems using submersible tankers. The possibility of navigating the Arctic Ocean year-round with icebreaking tankers is extrapolated from the field performance of the SS MANHATTAN. The possibility of routing submarine tankers under the polar icecap is predicated on the preformance of U.S. Navy submarines in the arctic. However, at this time, both of these options are considered advanced concepts in need of additional development to determine technical and economic feasibilities."

This conclusion appears to have been based on proposals submitted to the Department of Energy by Dome Petroleum Ltd., Globtik Corporation, and Seatrian Lines, Inc. The Department of Energy spokesman went on to say:

"There are several unresolved issues common to the above discussed transportation systems using icebreaking tankers. The major technical question is whether any of the proposed systems can indeed navigate the arctic year-round. At present, there is no surface ship which can operate under all arctic ice conditions. The issue
revolves around not only the icebreaking capability of the tankers, but also the variability of arctic ice conditions. The major economic question is whether the high capital costs of these systems are justified given the uncertainties of cargo levels (that is, of arctic oil and gas reserve levels) and of the navigability of the Northwest Passage. The safety and environmental impacts of such systems are at issue, as well as the effects of such high-technology development on the native Inuit Indians. An additional question of international scope concerns territorial claims to the arctic waterways that would be used by the icebreaking tankers in the proposed systems.

And later:

"Considerable development work on alternative marine systems has been performed by industry and government agencies such as MARAD.* However, most of this work has not been demonstrated or developed to levels whereby the required technical and economic comparisons can be made. Moreover, long lead times are required to achieve the necessary development.

Development needs include gaining a better understanding of the arctic environment and the technical requirements of vehicles such as icebreaking tankers and underwater pipelines for minimizing ecological and environmental impacts. The potential involvement of other nations should be evaluated for each alternative concerning territorial rights and regulations.

Marine transportation alternatives could offer the potential advantage of being highly flexible, and consequently economically attractive, in that the level of transportation capability can be closely coupled with the rate of expansion of proved oil and gas reserves. On the other hand, such systems have not been demonstrated for natural gas transportation and require facilities for conversion to LNG, PNG, or other liquid forms such as methanol."

Perhaps many of us in the marine transportation industry would disagree with some or all of the statements made by this U.S. Department of Energy spokesman. Yet, it is very clear from his statements that the decision makers involved with the Alaskan natural gas transportation decision believe that:

* U.S. Maritime Administration.
(1) Arctic marine transportation systems are advanced concepts in need of additional development to determine their technical and economic feasibility; and

(2) Arctic marine transportation systems have not been demonstrated or developed whereby the required technical and economic comparisons can be made.

Whether we like it or not, I believe that the above examples clearly show that the marine industry has not done its homework and that decision makers currently believe that commercial Arctic marine transportation systems are technically and economically infeasible. What, then, can we do to change this state of affairs? Let's first attempt to analyze our problem.

ANALYSIS OF PROBLEM

The problems facing the segment of the marine industry involved with Arctic transportation are complex, but I can't believe they are unsolvable. I do not pretend to know all the answers to these problems. Nor do I pretend to even be capable of listing them or organizing them into a coherent program for solution. However, I'm going to try.

I think the biggest problem we face is that the decision makers (i.e., those involved with deciding to commit or go after 20 billion dollars to build a transportation system) have no proof that Arctic marine transportation systems are technically or economically feasible. The only way we can prove technical and economic feasibility is with real, full-scale operational experience of an actual Arctic marine transportation system. No amount of model testing, paper studies, design studies, or full-scale experiments with specialized icebreakers will ever be sufficient proof. This is not to imply that these efforts are unnecessary or useless. Every engineer knows they are not. It's just that such efforts are all fruitless until they are demonstrated in an actual marine transportation system.

Another significant problem we face is our lack of organization. As an example of what I mean, let's look again at the Alaskan natural gas transportation problem. Was an organized, hard-hitting, Arctic Marine Transportation System proposal made to the decision makers which included such things as: (1) detailed designs of Arctic LNG ships and terminals prepared by experienced engineers and designers and backed by complete model studies; (2) detailed routing analyses prepared using the most up-to-date ice information and full-scale ship performance data; (3) a thorough economic analysis including identification of sources of construction funds; and (4) a thorough investigation of the environmental impact? Was the proposal marketed in a timely manner to every conceivable person, organization, or group that might
influence the decision of the Federal Energy Commission? Was the effort organized by the marine industry commensurate with the potential for the influx of 20 billion dollars to the industry? I suspect that the answer to all of these questions is no. The fact is that the marine industry, at least in the U.S., is splintered and fragmented. Even a large shipping company does not appear to have the organizational structure to pull together a team capable of competing with pipeline companies.

Lack of long range planning appears to be another problem with our industry. The U.S. Department of Energy spokesman quoted earlier stated that, "Moreover, long lead times are required to achieve the necessary development." It does, in fact, take years of planning, designing, and construction to implement major energy transportation systems. U.S. and Canadian oil companies are vigorously exploring arctic offshore waters for new oil and gas sources. Significant discoveries have already been made which we know will create a demand for transportation in the near future. Are we doing the necessary planning and are we implementing plans which will ensure that marine modes of transportation will be considered as a viable alternative? I think not. In fact, I have served (as I'm sure many of you have) on several government and professional society panels over the past several years whose purposes were to develop plans for establishing arctic marine transportation systems. Very few, if any, of the elements in these plans have, to date, been implemented.

I'm sure that I have only hit on a few of the problems facing our industry today. Hopefully, the ones I've listed are the major ones. But the really important question is what can we do to help solve our industry-wide problems?

RECOMMENDATIONS FOR ACTION

The most important recommendation I can make to our industry is to immediately build and then operate on a year-round basis, a large arctic icebreaking tanker. I envision the construction of a 100,000 to 200,000 DWT, Class 10 (Canadian Arctic Pollution and Prevention Regulations Classification) ship based on the best design data we currently have available. I recommend that this ship operate on a year-round basis from the Port of Valdez, Alaska, through the Northwest Passage to ports along the east coast of North America. This route would eliminate the need for an expensive arctic terminal while providing the basis for technical and economic evaluation. There are tankers currently transporting crude from Valdez to U.S. east coast ports via the Panama Canal so these could provide a basis for economic comparison.

There are numerous options for financing and owning the ship. A consortium of the U.S. Government, Canadian Government, and shipping companies from both countries is one. Such a consortium would have the advantages of spreading risks and involving federal agencies which could resolve questions related to territorial claims to arctic
waterways, pollution prevention, and environmental and social impacts.

The tanker would be equipped with the most advanced navigation equipment available and would be heavily instrumented. On each voyage, a small team of engineers and scientists would be on board to collect ice and ship performance data for the purpose of verifying design calculations and improving future designs. The economics associated with the operations would be continuously compiled and made available to consortium members.

The benefits which would accrue to the marine industry throughout the world, especially in the United States and Canada, I think are tremendous. A true alternative mode for transporting oil and gas resources from arctic waters would be proven. And, if my estimates are correct, the cost of transporting future energy would be considerably reduced, thus benefiting the citizens of both the United States and Canada.

Perhaps there are other arctic marine transportation projects more worthy of consideration than the ones I have considered in this paper. If you have any such ideas, I hope you will express these to responsible persons so that we can do something to change the current state of the arctic marine industry.

REFERENCES


EXTENSION OF THE NAVIGATION SEASON
ON THE
GREAT LAKES AND ST. LAWRENCE SEAWAY SYSTEM

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ABSTRACT

The Great Lakes within the United States and Canada is the world's largest body of fresh water and embodies the Great Lakes - St. Lawrence Seaway System. This system provides a low cost, energy efficient, deep draft waterway to the U.S. and Canadian heartland. Historically, the system closes down from about mid-December to around 1 April due to weather and ice conditions.

In 1970, the United States Congress authorized a Winter Navigation Program consisting of an action-oriented demonstration program, and a detailed investigation (survey study) to determine the feasibility and the U.S. Federal interest in extending the navigation season to as much as year-round. The two related but independent activities were completed and reports were prepared by the Detroit District, Corps of Engineers, in 1979.

The overall finding during the eight years of the demonstration program is that the traditional navigation season on the Great Lakes - St. Lawrence Seaway System has been successfully extended. The Demonstration Report has been provided to the Governors and Members of Congress of the Great Lakes States for their information.

The Detroit District Engineer, in his final survey report summarizing the feasibility study, concluded an extension of the navigation season is considered engineeringly and economically feasible and recommends an extension on the upper three Great Lakes, the St. Clair River - Lake St. Clair - Detroit River System and Lake Erie to year-round, and up to ten months on Lake Ontario and the International Section of the St. Lawrence River. He further recommends that the work be accomplished in geographically and time-based incremental phases. It is recognized that formal agreement with the Government of Canada is required for an extension on the system beyond the upper three Great Lakes. To assure and to confirm environmental and social feasibility of the program, an Environmental Plan of Action is also recommended to be implemented.
concurrently with post-authorization planning, engineering studies, project construction, and system operations. This would involve a program of baseline data collection and assessment prior to construction, and monitoring during construction and operations, to provide information for identifying system changes and adjusting policy and management actions. Additionally, it provides a validation process to confirm, if justified, continuation of the project.

INTRODUCTION

The Great Lakes Basin economy is basically industrial with a major recreation and tourist industry. In addition, there is significant agricultural, mining, and forestry production. The Basin's importance is underscored by the fact that the United States' portion of the Basin contains one-seventh of the national population and produces one-sixth of the national income--on less than five percent of the Nation's land area.

The Great Lakes - St. Lawrence Seaway System, including Lakes Superior, Michigan, Huron, Erie, and Ontario, the connecting channels, and the St. Lawrence River, form a water highway 2,342 miles long, from Duluth, Minnesota, at the head of Lake Superior, to the Strait of Belle Isle at the mouth of the Gulf of St. Lawrence on the Atlantic Ocean. There are over 60 ports serving domestic and international traffic. Existing Federal projects in the Great Lakes and their connecting channels, and most primary commercial harbors provide for a controlling navigation depth of 27 feet (25.5 feet safe draft) at Low Water Datum (LWD). Four U.S. navigation locks on the St. Marys River allow commercial navigation between Lake Superior and the lower lakes, the largest of which is the Poe Lock (1,200 feet long by 110 feet wide by 32 feet depth over the sills). The Canadian - Welland Canal, with a minimum depth of 27 feet, is the navigation link between Lakes Erie and Ontario. The canal has a series of eight locks permitting transit of ships 730 feet long and 76 feet wide. The St. Lawrence River from Lake Ontario to Montreal has a maximum permissible vessel draft of 25-3/4 feet. The main river channels have a controlling depth of 27 feet. There are seven locks in the system, two of which are operated by the U.S. and five by Canada. The locks are all 800 feet long and 80 feet wide, permitting transit of ships 730 feet long and 75.5 feet wide. Below Montreal to the Atlantic Ocean there is a 35 foot deep navigation channel.

The development of the St. Lawrence River to allow deep-water ocean-going vessels to reach Great Lakes ports and the development of the low moisture taconite pellets, created an economic demand for a more intensive use of the System. An extension of the navigation season is one method of meeting that demand. In response, the U.S. Congress authorized the Winter Navigation Program and provided initial funding in 1971. Program activities have continued for eight years at a total cost of $21,027,000. The program was conducted under the auspices of a
Winter Navigation Board comprised of senior representatives of eight Federal agencies and State, business and labor interests.

The purpose of the navigation season extension program is twofold: (1) to demonstrate the practicability of means of extending the season, and (2) to determine the feasibility of means of extending the navigation season and the extent of Federal participation. Practicability is determined by actually demonstrating in the field the means for extending the navigation season, such as air bubblers, ship voyages, and icebreaking. Feasibility is determined by evaluating the engineering, economic, environmental and social aspects and impacts collectively of a project and making a judgment as to whether the project is justified and is in the interest of the Federal Government to fund.

DEMONSTRATION PROGRAM

Commercial navigation has successfully been extended beyond the historic closing date of 16 December on the upper four Great Lakes and connecting channels during every year of the demonstration program. Year-round shipping was achieved during the last four of five years. On the St. Lawrence River, where historically the season extended from mid-April to early December, the longest commercial season in history was recorded in 1975 with a 25 March opening and a 20 December closing. Specific findings resulted from activities in the areas of ship movement through ice, navigation aids, ice and weather information, crew safety and survival, ice control, and island transportation. In addition, under an ongoing program, year-round lock operation was demonstrated.

. The use of preventative icebreaking and the use of ship convoys were useful tools in moving vessels through ice. The use of air bubbler systems were both effective in melting or reducing ice cover and easing vessel movement through areas of stable ice cover. Bubbler systems at the Lime Island Turn in the St. Marys River showed the practicability of this type of system in reducing ice thicknesses in a river.

. Specially designed ice buoys for use under ice conditions showed some success, but due to their limited utility, emphasis was placed on developing various electronic navigation systems. A mini-Loran C navigation system was used on the St. Marys River, but its accuracy within narrow channels was not yet demonstrated. A Precise All Weather Navigation System (PAWNS) was not fully demonstrated during the program. Radar transponder beacons (RACONS) successfully extended the range and utility of shipboard units.

. Weather and ice information was disseminated by a special Ice Navigation Center at the Ninth Coast Guard District Headquarters in Cleveland, Ohio, in coordination with the National Weather Service. Aerial reconnaissance and Side-Looking Airborne Radar (SLAR) were successfully used as inputs to the ice information portion of the program. The aerial reconnaissance and SLAR provided real-time information on the extent of ice cover. Methods of providing both long and short range ice forecasts
for all areas of the Great Lakes were developed by the National Oceanic and Atmospheric Administration. An ice forecast model was developed to predict the ice breakup period in the St. Lawrence River. The ice breakup forecast technique is used to allow advance scheduling of ocean trade vessels into the system.

- Crew safety and survival in an extended season were given considerable attention. The Coast Guard field tested and evaluated a variety of personnel exposure suits and survival equipment. Emergency Position Indicating Radio Beacons (EPIRBs) and handheld radar transponders have shown effectiveness in pinpointing the location of both ships and personnel, enhancing the efficiency of search and rescue operations.

- Ice jams in constricted portions of the system, especially the St. Marys, St. Lawrence, and St. Clair Rivers, can prevent the passage of all but a few vessels with substantial ice operating capabilities. The jams also cause flooding problems and in some cases reduce the flow of water to power plants and municipal intakes. The annual installation of a navigation ice boom and other structures at the head of Little Rapids Cut in the St. Marys River allowed vessel movement to continue year-round with no major ice problems occurring. Similar, but limited, demonstrations were conducted at Copeland Cut in the St. Lawrence River with the same results. Model studies were conducted to determine the type and effectiveness of ice control structures needed at the head of the St. Clair River by CRREL.

- Systems for winterizing lock operating machinery were successfully demonstrated. The use of co-polymer coatings and steamlines was effective in removing ice from lock walls. A bubbler system and air curtain were effective in keeping floating ice out of gate recesses and limiting the amount of ice entering locks. Protective housing and the use of heated cable helped prevent ice buildup on lock gate machinery.

- Winter navigation has the potential to contribute to increased shore erosion and damage to docks in limited areas of the connecting channels. Further investigation is required to distinguish between that damage caused by ship movement in ice and that caused by natural ice movement.

- Transportation for island residents can be maintained while permitting navigation. To test means of improving the ice operating capabilities of the Sugar Island ferry, several modifications were made to its hull and power components. These modifications enabled the ferry to operate in moving ice floes. The installation of the St. Marys River ice boom above Little Rapids Cut reduced the amount of ice moving down the Cut, further increasing the ferry's capabilities. Additionally, the installation of an air bubbler-flusher unit at the Sugar Island ferry mainland dock, to create a surface current to physically flush ice away from the dock, enabled the ferry to land more easily. An airboat was utilized at Lime Island to provide transportation for the residents of that Island. During the program, several improvements were made to the airboat but the residents have expressed dissatisfaction with this form of transportation.
The upper four Great Lakes and their connecting channels are significantly different, both physically and administratively, from the lower portion of the system. On the upper four Great Lakes, the program was carried out essentially in the United States' waters and did not require co-participating involvement of Canada. In the St. Lawrence Seaway, activities were in the waters of both Canada and the United States and could not be implemented to the same extent, that is including full vessel tests and similar winter operations, without substantial improvements in the all-Canadian portion of the St. Lawrence River. In addition, concerns by power entities initially delayed demonstration program execution on the St. Lawrence Seaway portion of the system.

Under these circumstances, operation on the upper four Great Lakes was stressed in order to obtain prototype information and delayed such tests in the St. Lawrence River toward the end of the program in the belief that sufficient technical data would be acquired in the course of the program to allow resolution of the questions raised by local interests and New York State. Strong opposition by local interests, including the State of New York, finally precluded full demonstration program execution on the St. Lawrence Seaway portion of the system. In 1976, the Winter Navigation Board requested that in view of these circumstances on the St. Lawrence Seaway, Congress provide two additional years and appropriate funding to carry out the St. Lawrence Program including vessel tests. This funding and authority were provided, but the issues could not be resolved and comprehensive vessel tests were never carried out.

Sufficient data on the effects of extended season navigation on the Great Lakes environment are not currently available. A limited number of environmental studies have been conducted on some of the demonstration program activities. While this does not comprise a complete analysis, adverse impacts to the environment have not been documented in the areas that have been investigated. Several baseline studies have been accomplished but they are not sufficient to make judgments as to the long range effects of the program.

The eight years of the demonstration program have shown that technically an extension of the traditional navigation season is practicable. Several issues still need to be resolved before a permanent season extension could be implemented.

1. Before the practicability of winter navigation in the St. Lawrence River can be determined, it must be demonstrated that certain existing ice control structures and related ice fields can be safely transited without disrupting the integrity of the ice fields and adversely affecting regulated water levels and flows of the river.

2. Significant amounts of environmental baseline data need to be collected to establish parameters against which to evaluate the extended season activities.
and to form part of the basis for measures and practices that may be necessary to protect the natural resources of the system.

3. The United States Government needs to seek appropriate Canadian participation in future extended season activities. Their participation is essential before overseas commercial shipping in the system can be extended.

The demonstration program final report does not contain recommendations concerning implementation of a navigation season extension. However, the results and conclusions obtained have been used for formulating future plans, programs, and recommendations to Congress under the survey study authority.

SURVEY STUDY

The Survey Study addressed all significant engineering, economic, environmental, social, and institutional aspects of navigation season extension on a system-wide basis.

An Interim Survey Report was prepared in 1976 and submitted to higher authority for review by the Detroit District Engineer. The District Engineer recommended an extended navigation season to 31 January ± 2 weeks on the upper four Great Lakes between non-ice restricted harbors using existing operational measures. The recommendations were primarily based upon the experiences gained under the Demonstration Program. Following intensive formal Washington review of the report, it was determined that authority exists for Federal agencies to implement most features of the plan.

This guidance formed the base conditions for the recommendations in the Final Survey Report which considers further navigation season extension on the entire System.

Specific conclusions of the Survey Study are:

1. Studies and the Demonstration Program have shown that season extension up to year-round on the upper four Great Lakes (Superior, Michigan, Huron, and Erie) and the connecting channels (St. Marys, St. Clair, and Detroit Rivers) is engineeringly and economically feasible.

2. Studies have further shown that season extension on Lake Ontario and the International Section of the St. Lawrence River is engineeringly and economically feasible up to an 11-month season.

3. Should a navigation season extension be implemented, it would be phased, both geographically and over time. Evidence, to date, indicates that season extension can be accomplished in an environmentally acceptable manner. However, it would be necessary to utilize a concept (referred to as an Adaptive Method) to insure environmental investigations and considerations are appropriately integrated with engineering planning, design, and construction activities and to provide necessary checks and balances to assure protection of the environment. This would include the implementation of an Environmental Plan of Action to gather data, analyze, suggest or recommend
engineering changes, mitigation, etc., then to confirm assumptions, observations and conclusions reached with respect to environmental feasibility.

Mitigative measures have been identified to preclude disruption of island transportation in the St. Marys and St. Lawrence Rivers, however, to assure that impacts of extended season navigation are minimal on Drummond Island on the St. Marys River, and Harsens Island on the St. Clair River, further monitoring of these areas would be carried on and mitigative action taken, if needed.

Existing ice booms in the St. Lawrence would need to be modified to permit winter vessel transits. Vessel transit tests would also be required to fully evaluate alternatives. These measures would need to be coordinated with the Government of Canada.

System benefits to costs are favorable to the Nation. Benefits would be realized in the areas of transportation efficiency, stockpiling savings, better fleet utilization, and more effective use of the Great Lakes Waterway. As the Great Lakes fleet has been substantially upgraded in the last ten years (with this trend continuing), the more complete utilization of recent fleet improvements is of even greater necessity.

Studies have indicated that energy consumption from an extended navigation season compares favorably to energy consumption of alternate transportation modes.

Studies to date have indicated that, should an extended navigation season be implemented, the hydraulic regime of the system could be maintained or improved through mitigative action.

Studies to date have indicated that navigation season extension would result in an increase in regional income and employment for ports and their surrounding areas within the Great Lakes Region.

The necessary steps to achieve formal Canadian coordination should begin upon authorization of this project by the Congress.

The recommendations of the District Engineer are for 12-month navigation on the upper three Great Lakes and their connecting channels, up to 12-month navigation on the St. Clair River-Lake St. Clair-Detroit River System and Lake Erie, and up to 10-month navigation on Lake Ontario and the International Section of the St. Lawrence River to be accomplished in separable phased increments, concurrently with an Environmental Plan of Action to be authorized for construction as a Federal project for navigation purposes. The first cost to the United States is presently estimated at $447,380,000 with an annual operation, maintenance and replacement costs presently estimated at $54,044,000. The locations of the measures in the recommended plan are shown in Figure 1, Pages 9 and 10. Those measures directly related to the responsibility of the U.S. Army Corps of Engineers: Ice Control Structures in Great Lakes Harbors and Connecting Channels; Air Bubbler Systems in Great Lakes Harbors and Connecting Channels; Lock Modifications at Sault Ste. Marie, Michigan; Dredging in
Location Map
for
Measures in Recommended Plan
NOTE: SEE PAGE 9 FOR MEASURES KEYED TO LOCATION MAP
SURVEY STUDY FOR
GREAT LAKES AND ST. LAWRENCE SEAWAY NAVIGATION SEASON EXTENSION

LOCATION MAP FOR MEASURES IN RECOMMENDED PLAN
NOTE: SEE PAGE 9 FOR MEASURES KEYED TO LOCATION MAP
SURVEY STUDY FOR
GREAT LAKES AND ST. LAWRENCE SEAWAY NAVIGATION SEASON EXTENSION

FIGURE 1
Survey Study for
Great Lakes and St. Lawrence Seaway
Navigation Season Extension
MEASURES IN RECOMMENDED PLAN

SYSTEM WIDE MEASURES

ICEBREAKERS TYPE B (4)
ICEBREAKERS TYPE C (20)
AIDS TO NAVIGATION
ENVIRONMENTAL PLAN OF ACTION

UPPER FOUR LAKES-ONLY
VESSEL TRAFFIC CONTROL
ICE DATA COLLECTION/DISSEMINATION
WATER LEVEL MONITORING

UPPER THREE LAKES-ONLY
ICE AND WEATHER FORECAST

CHANNELS AND LAKES

A. AIR BUBBLER SYSTEM
B. SHORELINE PROTECTION AND SHORE STRUCTURE COMPENSATION
C. LOCK MODIFICATIONS U.S. LOCKS
ICE BOOM
ICE STABILIZATION ISLANDS (2)
ICEBREAKER MOORING TYPE B
ICEBREAKER MOORINGS TYPE C (5)
D. AIR BUBBLER SYSTEMS (5)
DREDGING ISLAND TRANSPORTATION
E. PILOT ACCESS
F. ICEBREAKER MOORING TYPE C
G. ICEBREAKER MOORING TYPE C
H. ICE BOOM (4,600')
COMPENSATION WORKS
SHORELINE PROTECTION AND SHORE STRUCTURE COMPENSATION
I. PILOT ACCESS
ICEBREAKER MOORING TYPE B
ICEBREAKER MOORINGS TYPE C (2)
J. ICE BOOM (6,000')
COMPENSATION WORKS
SHORELINE PROTECTION
K. PILOT ACCESS
ICEBREAKER MOORING TYPE C
L. ISLAND TRANSPORTATION
M. ICEBREAKER MOORING TYPE C
N. ICEBREAKER MOORING TYPE C
O. ICE BOOMS (11 EACH)
LOCK MODIFICATIONS
FIREHOSE AND SNELL
SHORELINE PROTECTION AND SHORE STRUCTURE COMPENSATION

HARBOR MEASURES

1. ICEBREAKING TUG
2. AIDS TO NAVIGATION
BUBBLERS (23,000')
HIGH POWER ICEBREAKING TUG
CHANNEL CLEARING CRAFT
ICEBREAKER MOORING TYPE B
3. BUBBLES (1,000')
HIGH POWER ICEBREAKING TUG
4. BUBBLES (1,000')
ICEBREAKING TUG
5. BUBBLES (17,000')
ICEBREAKING TUG
ICEBREAKER MOORING TYPE C
6. NAVIGATION LIGHTS (4)
ICEBREAKING TUG
7. BUBBLES (8,000')
8. ICE BOOM (4,000')
9. ICE BOOM (7,600')
ICEBREAKING TUG
10. ICE BOOM (8,000')
ICEBREAKING TUG
11. NAVIGATION LIGHT
BUBBLER (2,000')
12. NAVIGATION LIGHTS (2)
ICE BOOM (10,000')
ICEBREAKING TUG
13. BUBBLES (4,000')
ICEBREAKING TUG
14. AIDS TO NAVIGATION
ICEBREAKING TUG
ICEBREAKER MOORING TYPE C (2)
15. BUBBLES (1,000')
HIGH POWER ICEBREAKING TUG
16. BUBBLES (1,000')
ICE BOOM (1,600')
ICEBREAKING TUG
17. ICE BOOM (6,800')
18. ICE BOOM (4,800')
ICEBREAKER MOORING TYPE C
ICEBREAKER MOORING TYPE B
19. ICE BOOM (7,200')
20. ICE BOOM (7,600')
21. HIGH POWER ICEBREAKING TUG
ICEBREAKER MOORING TYPE C
22. ICEBREAKER MOORING TYPE C

FIGURE 1
CONTINUED
St. Marys River; Compensating Works in St. Clair River; Shoreline and Shore Structure Protection in the St. Marys, St. Clair and Detroit Rivers; Island Transportation Assistance in the St. Marys River; Water Level Monitoring in the St. Marys, St. Clair, and Detroit Rivers; Great Lakes Connecting Channels Operational Plans; and Environmental Plan of Action (joint responsibility with U.S. Fish and Wildlife Service).

In addition, to provide the institutional framework to oversee the implementation of these recommendations and to provide a final validation report, the U.S. Section of an eventual Joint Board should be created. The agencies would include Department of the Army, Department of the Interior, Department of Transportation and a State Representative. This U.S. Section would be disestablished once the objectives of the Season Extension Program have been realized, the operations satisfactorily tested and implemented, and the validation report submitted to Congress.

The above stated recommendations by the District Engineer, Detroit, and Division Engineer, North Central, have been forwarded to higher authority to undergo the formal Washington review process. They have undergone review by the Board of Engineers for Rivers and Harbors (BERH). The BERH is an independent Review Board of the Corps of Engineers. The Board acted on the report 19 February 1980 and forwarded it to the Chief of Engineers, U.S. Army Corps of Engineers, for his consideration prior to forwarding it for Congressional consideration by the Secretary of Army and the Office of Management and Budget.

The findings by BERH are as follows:

a. The report should be transmitted to Congress for information;

b. Further environmental and other analyses of the 31 January + 2 weeks season on upper Great Lakes (base condition) should be continued under present Corps' operational programs;

c. Season extension in the United States is primarily an operational matter for which there is adequate authority, but for which specific measures may require additional authority;

d. The Adaptive Assessment Technique and Environmental Plan of Action, as proposed by the reporting officers, are not acceptable for this particular study, and that a conventional predictive-type EIS, to assess environmental problems and solutions thereto, should be prepared prior to implementation;

e. Navigation season extension of up to ten months on the St. Lawrence Seaway-Great Lakes system and extending to about ten and three-fourths of a month on the upper four Great Lakes is economically justified, contingent upon appropriate Canadian cooperation and support. Additional extension to a year-round season on the upper Great Lakes appears to be marginally feasible based on existing information.

f. Canadian coordination and cooperation in navigation season extension on the Great Lakes-St. Lawrence Seaway System should be pursued by the United States.

This paper can only touch on conclusions and recommendations because of the multi-faceted aspects of the program and the vastness of the system. More technical data and reports can be obtained by writing the authors.
MARINE TRANSPORTATION IN ARCTIC WATERS

Stig Skarborn, P. Eng. Albery, Pullerits, Dickson & Associates, Don Mills, Ontario, Canada

A detailed study on the marine transportation of oil and LNG from the Arctic Islands to southern Canadian markets was carried out in 1977. Key elements in the study were to evaluate the required horsepower of the ships, the rate of progress through various ice conditions, the ships' fuel consumption, and other related costs.

Ice conditions were evaluated for segments of the route with similar conditions on a monthly basis, including probabilities of a given multi-year ice concentration occurring. Multi-year ice avoidance criteria were established and the forward rate of progress calculated on the basis of a formula relating ice and ship parameters. Allowances were made for ridging, ramming in unavoidable concentrations of multi-year ice, speed restrictions due to environmental factors and other delays. It was concluded that it is technically and economically feasible to transport oil and gas from Arctic to southern markets.
1.0 INTRODUCTION

As conventional sources of oil and gas in Southern Canada are becoming depleted, exploration in the Canadian Arctic has gathered momentum. Oil has been discovered, and gas has been proven in quantities which should be marketable, if an economically viable and environmentally acceptable transportation system can be implemented.

Both pipeline and marine systems have been studied by private industry. In order to obtain an independent appraisal of the viability of a predominately marine transportation system, Transport Canada, Strategic Studies Branch, initiated a study in 1977 which was carried out by Albery, Pullerits, Dickson and Associates. [Ref. No. 1]

The supply source for oil was assumed to be Cameron Island, and for natural gas, King Christian Island and Melville Island (Fig. 1). The southern delivery area was assumed to be the Montreal area. Trans-shipment, at various locations, and both arctic and southern pipelines were considered as alternatives to an all marine route.

In undertaking the study, as much use as possible was made of existing information. However, it was found that there was comparatively little existing information available that was reliable. It was therefore necessary to undertake considerable new research and analysis, particularly with regard to ice conditions and the performance of large vessels in ice.

2.0 ICE CONDITIONS

General descriptions of the sea ice have been given in a number of previous publications, notably "Arctic Resources by Sea" [Ref. No. 2] and the "Sea Ice Atlas of Arctic Canada 1961-1968" [Ref. No. 3], where the descriptions were prepared from data largely based on information gathered from June through September. These publications do not take into account recent
satellite imagery, or ice thickness measurements made during seismic studies carried out by the Arctic Petroleum Operators Association.

In order to update the available information and present it in a form useful for assessing the feasibility of year-round commercial shipping, extensive studies were carried out by Norcor Engineering and Research Ltd. [Ref. No. 1]. These studies involved subdividing the various potential routes into 49 legs in order to isolate areas with similar ice conditions, as shown in Figure 1 and Figure 2. For each leg, the mean concentrations and the mean thickness of first-year ice and multi-year ice were derived for each month of the year, as shown in Figure 3 for a typical leg. Also shown in Figure 3 is the root mean square (RMS) ice thickness (a composite value initially intended to be used for calculating the ships' resistance to motion in ice), and the probability of a given multi-year ice concentration being exceeded.

On a typical voyage, five main zones of different ice conditions would be encountered:
- Sverdrup Basin
- Parry Channel and Jones Sound
- Baffin Bay and Davis Strait
- Labrador Sea
- St. Lawrence River and Gulf.

The Sverdrup Basin has a very short open water season and a high concentration of multi-year ice which increases rapidly to the west of Cornwall Island. Once freeze-up has effectively taken place, the ice within the basin becomes fast and unmoving and the multi-year ice is locked in position for 10 to 11 months until the following break-up. This means in effect, that if an open-water route to King Christian Island can be found just before freeze-up, the same route should be available through first-year ice for the rest of the winter.
Parry Channel is a transition zone between the dynamic first-year ice in Baffin Bay and the relatively stationary mixture of first-year and multi-year ice in the Sverdrup Basin. Conditions become progressively more severe moving westward through the channel.

Jones Sound is predominantly first-year ice, but especially at the west end in Cardigan Strait, there is some multi-year ice which drifts in from Norwegian Bay.

Baffin Bay and Davis Strait have ice conditions which are probably the most dynamic of the regions considered in this study. Changes in wind direction can cause major ice movements at almost any time of the year, with the creation of leads or serious pressure situations.

The Labrador Sea has significant ice only adjacent to the Labrador coast, and only a route into Groswater Bay would encounter land fast and consolidated pack ice. Icebergs would, however, be a serious hazard which would tend to reduce the safe speed in these waters under conditions of poor visibility.

In the St. Lawrence River and Gulf, the ice conditions would present no problem to the large Arctic Class ice-breaking bulk carriers.

3.0 MULTI-YEAR ICE AVOIDANCE
The multi-year ice is obviously a major obstacle to the progress of large ships unless excessively large propulsion systems are installed. It will, therefore, be essential to select routes which avoid multi-year ice as far as possible, for any commercial marine system in the High Arctic. In those areas of the Sverdrup Basin and Viscount Melville Sound where multi-year ice generally exists in considerable concentrations, the ice becomes fast and essentially unmoving after freeze-up. Therefore, a route reconnoitred and planned at freeze-up should be available throughout the winter until break-up.
There is, unfortunately, insufficient detailed information over a long enough period to assess, with any reasonable degree of certainty, the extent to which multi-year ice can be avoided when it exists in fairly large concentrations. For the purpose of comparative system studies, it was assumed that in an average year all multi-year ice up to a maximum concentration of 7/10 can be avoided if necessary, at the expense of substantial detours. The assumed relationship between percentage of multi-year ice and the distance multiplier is shown in Figure 4. In an exceptionally bad year, a channel may have to be broken through multi-year ice over considerable distances. The broken channel may be available for a number of transits until the depth of broken ice increases to such an extent that the channel becomes clogged and another channel has to be forced through the multi-year ice.

While it is the apparent opinion of some investigators, that a ship could beneficially use the same track throughout the entire winter, others [Ref. Nos. 4, 5, 6 and 7] are less optimistic. Cases have been presented where the broken ice has frozen into a mass which ice-breakers no longer could break. Measurements in a northern Baltic port, and its approach channel, indicated that a total broken ice matrix of up to 5 m thick (original ice 60-70 cm thick) after some 47 ships had used the channel in a three month period. In addition, a broken channel may present a weakness plane in the ice which either closes up or becomes the location of a ridge. While the re-use of the same track for a number of transits is partially supported by recent Baltic experience, there must be some doubt regarding the extent to which this practice would be equally successful in the more severe Arctic environment. The generation of ice in the channel primarily depends on the number of vessel transits and the degree-days of freezing, and it would be reasonable to assume that each new track through multi-year ice could continue to be used with advantage until the product of the number of transits and the degree-days of frost exceeds a certain limit. A benefit will also arise from using the same track through fast first-year ice, but, although
this will obviously be done to a considerable extent in practice, no allowance was made in the study for increased speed or reduced horsepower.

4.0 ICE RESISTANCE AND RATE OF PROGRESS

In spite of a fairly long history of attempts to explain the performance of vessels in ice, the science of predicting performance is still in its infancy. A number of investigators, mainly in Canada, the U.S.A., Russia, Finland and Germany, have studied the question in some detail and, in recent years, numerous model and full-scale tests have been carried out. However, most of the research has related to the resistance of continuous uniform ice and comparatively little has been done on the resistance of pack ice of different floe sizes and varying concentrations. There has also been little research on the effect of the size, composition and spacing or pressure ridges. Even in the case of uniform ice, there is considerable disagreement among the experts regarding the resistance of ice to the motion of a ship, especially at higher velocities and for ice thicknesses beyond existing experience.

4.1 Performance in Continuous Uniform First-Year Ice

While a number of workers in the field from about 1890 onward have presented equations for the resistance of uniform ice to the motion of a ship, it was V. I. Kashteljan et al [Ref. No. 8] who wrote the first comprehensive and scientific treatise on the subject. All subsequent investigations have followed the same general line of reasoning, and there is general agreement that the resistance to motion in continuous solid ice consists of the following main components:

- the resistance due to the actual breaking of the ice;
- gravity forces due to the turning and submerging of the broken ice; and
- inertia forces dependent on the velocity of the ship.

In each component, the parameters and indices selected are based on each author's appreciation of the physical forces at
work and the principles of dimensional similarity which must be adhered to, if model test results are to be scaled up to full size. Within this process of formulating an equation, there are many combinations of parameters and indices which will satisfy dimensional similarity with the result that no two authors have derived identical equations. There is, however, almost unanimous agreement on three points:
- the overall resistance varies linearly with the beam of the ship;
- the ice-breaking component is a relatively small part of the total resistance; and
- the strength of the ice is not of major importance.

The disagreements concern mainly the indices to be applied to "h", the ice thickness, and to "v", the speed of the ship. Some researchers find a predominantly linear relationship between resistance and ice thickness, while others find a mainly $h^2$ relationship. Similarly, some find resistance varying with "v", while others find a variation with $v^2$ or something in between. Clearly, these variations in indices can make a big difference when considering large ice thicknesses and moderately high velocities.

In addition to the above disagreements, very few equations developed to date use the draft of the vessel as a parameter or take friction into account, except as a form-dependent coefficient. There is growing evidence that resistance varies with draft, and it is now recognized that friction plays a very important part in the total resistance to motion [Ref. No. 9].

In view of the wide range of results given by existing equations, a middle-of-the-road equation was developed, which takes draft into account, and for which coefficients were calibrated against confidential data made available from the Manhattan tests. It was found, however, that such an equation gave results close to the equation of Enkvist [Ref. No. 10]
and, since this also took draft and friction into account, it was finally decided to use Enkvist's equation for predicting vessel performance.

While it is necessary to use a predictor equation to assess the horsepower requirements and the speed of advance through the ice, the uncertainties involved in the present state of knowledge should be fully appreciated. In the Enkvist equation, as with most other equations, the values of the various coefficients have been assessed from either model tests or a few full scale tests with relatively small ice-breaking vessels. Regression analysis of the data has often been employed, and the results apply only to the vessel or vessels tested and the range of variation of the parameters for which the data was obtained. Even if it could be shown that the equations were fundamentally sound and incorporated all parameters affecting ice resistance, the degree of extrapolation would still be very great, as shown in Figure 5. In fact, it is clear that most equations developed to date do not include all necessary parameters: the missing ones being covered by coefficients which are obtained empirically and which apply only to the vessel in question. As a result, the various equations give widely different results, if used to predict the performance of large bulk carriers. This is illustrated in Figure 6, which shows the resistance at various velocities given by a selection of the equations for a VLCC of 300,000 tonnes deadweight in 2.5 m thick ice.

4.2 Ramming Through Multi-Year Ice
A fundamental assumption used in the analysis was that, in an average year, multi-year ice can largely be avoided. However, in those years when the concentration of multi-year ice is greater than average, it may be necessary to resort to ramming over a considerable distance. This mode of operation was therefore studied. However, the space restraints of this paper only allow a typical result to be presented.
On the basis of a 10 knot ramming speed, Table 1 summarizes the results of the calculations for a 125,000 m³ LNG carrier having 200,000 shaft horsepower.

**TABLE I**

**RAMMING PROCEDURE SUMMARY**

125,000 m³ LNGC, 200,000 H. P.

<table>
<thead>
<tr>
<th>Procedure</th>
<th>Time</th>
</tr>
</thead>
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<tr>
<td>Ramming</td>
<td>66 s</td>
</tr>
<tr>
<td>Delay in maneuvering</td>
<td>60 s</td>
</tr>
<tr>
<td>Backing</td>
<td>110 s</td>
</tr>
<tr>
<td>Delay in maneuvering</td>
<td>60 s</td>
</tr>
<tr>
<td>Charge through broken ice</td>
<td>71 s</td>
</tr>
<tr>
<td></td>
<td>367 s = 6.12 minutes</td>
</tr>
<tr>
<td>Rams/nautical mile</td>
<td>12.9</td>
</tr>
<tr>
<td>Time for nautical mile</td>
<td>78.9 minutes</td>
</tr>
<tr>
<td>Average transit speed</td>
<td>0.75 knots</td>
</tr>
<tr>
<td>Distance travelled/day</td>
<td>17.97 nautical miles</td>
</tr>
</tbody>
</table>

Assumptions: Ice thickness 4 m (13 feet)

Ice bending strength 1,034 KPa (150 psi)

If it is assumed that unavoidable multi-year ice exists all the way over legs 25, 26 and 27 to King Christian Island, the calculations indicate that it would take about 9.1 and 7.4 days respectively to ram through concentrations of multi-year ice having 10 and 20 percent probability of occurrence. If it is further assumed that a channel, once formed, can be re-used for a total of six times with gradually increasing difficulty, the average round trip times from Hell's Gate to King Christian Island are 3.9 and 3.3 days, respectively, for the 10 percent and 20 percent probabilities, compared with about 1.5 days for travelling through first-year ice only.

In general, it may be said that the preliminary indications are that the need to ram through heavy concentrations of multi-year ice in one or two years out of 10 would not rule
out an all-marine route to and from King Christian Island or Melville Island. It is essential, however, to bear in mind the assumptions on which this tentative conclusion is based. These are:
- heavy multi-year ridges of ice islands can always be avoided;
- a rammed channel can be re-used subsequently for an average of about six times;
- the Arctic Class 10 LNGCs and their containment systems can be designed to withstand the repeated inertial forces imposed by ramming; and
- the assumptions adopted for the ramming calculations will be proven reasonable.

4.3 Performance in Broken Ice

There is insufficient data to predict the true performance of a vessel in various concentrations of broken ice and much will depend on the size of the floes predominating in the ice pack. It is known that the resistance to motion of a ship decreases rapidly as the ice concentration decreases, and becomes quite small for concentrations less than six tenths when most of the floes can be pushed aside rather than be broken.

In the ice conditions summaries, Figure 3, ice concentrations are given month by month for each leg of the journey on the assumption that the ship follows a straight course. In those months when the concentration of ice is less than about eight tenths, it will nearly always be possible to make minor detours and avoid some of the ice pack that would otherwise be encountered. On the assumption that forward reconnaissance and route selection will be an essential part of any commercial marine system in the Arctic, the following assumptions were made in order to simplify the calculation of the speed of advance in broken ice:

a) - the open water traversed will generally be 25 percent greater than that indicated in the ice condition summary;
b) - over the portion of the route leg equal to the adjusted percentage of open water, there will thus be open water resistance only; and
c) - over the remaining portion of the route leg, resistance will be as for continuous ice of the thickness indicated, plus a 15 percent allowance for ridging. Assumption (c) above is somewhat conservative, but is partly compensated for by assumption (a).

In weak and broken ice and in open water between ice floes, the very large horsepower installed in ice-breaking bulk carriers will allow high speeds to be maintained which may not be safe in certain areas, depending on the prevailing conditions.

Speed restrictions were established by selecting maximum allowable speeds, based on a seasoned captain's actual experience and foresight, for both good and poor visibility. The values given in Table 2 are weighed averages of these two speeds and have been derived by proportioning the speeds according to visibility and other relevant factors.

5.0 CONCLUSIONS
Using the previous criteria, it was found that a predominantly marine transportation system for oil and LNG from the Arctic Islands is technically feasible. The transportation of oil is commercially feasible, once reserves equivalent to at least 50,000 barrels per day have been proven. The marine transport of LNG from the Arctic Islands would be economically dependent on the escalation of the price of competing fuel sources over the next five to 10 years.

6.0 ACKNOWLEDGEMENT
While implementing the Canadian government assignment, Albery, Pullerits, Dickson and Associates were assisted in the study of ice conditions and ship resistance by the following firms and individuals who were part of the Study Team:

a) Norcor Engineering & Research Ltd. of Yellowknife, NWT, who were responsible for the analysis of ice conditions along the various shipping routes.
<table>
<thead>
<tr>
<th>AREA</th>
<th>MONTH</th>
<th>SPEED KNOTS</th>
<th>SPEED m/s</th>
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<td>1, 2, 3 Lancaster Sound</td>
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<td>10</td>
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</tr>
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<td>3A</td>
<td>Jun</td>
<td>14</td>
<td>7.2</td>
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<tr>
<td></td>
<td>Jul-Oct</td>
<td>18</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>Nov-Dec</td>
<td>13</td>
<td>6.7</td>
</tr>
<tr>
<td>4-7, 7A, 8, Viscount Melville</td>
<td>Dec-Jun</td>
<td>10</td>
<td>5.2</td>
</tr>
<tr>
<td>9, 16, 17, Sound, Austin Ch.</td>
<td>Jul</td>
<td>12</td>
<td>6.2</td>
</tr>
<tr>
<td>17A, 19-21 Wellington Ch.</td>
<td>Aug-Oct</td>
<td>14</td>
<td>7.2</td>
</tr>
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<td></td>
<td>Nov</td>
<td>12</td>
<td>6.2</td>
</tr>
<tr>
<td>10-15, 18, Sverdrup Basin &amp; Approaches</td>
<td>Dec-May</td>
<td>10</td>
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</tr>
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<td></td>
<td>Jun-Jul</td>
<td>11</td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td>Aug</td>
<td>13</td>
<td>6.7</td>
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<tr>
<td></td>
<td>Sep-Nov</td>
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</tr>
<tr>
<td>28-32 Baffin Bay N of 69°</td>
<td>Dec-Feb</td>
<td>10</td>
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<tr>
<td></td>
<td>Mar-May</td>
<td>11</td>
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</tr>
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<td>Jun</td>
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<td></td>
<td>May-Sep</td>
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<td>Oct-Dec</td>
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<td>35, 40, 45, Strait of Belle</td>
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<tr>
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<td>16</td>
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<td></td>
<td>Oct-Dec</td>
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<td>34, 36-39, Labrador Sea,</td>
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<td>9.3</td>
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<tr>
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<td>Jun-Sep</td>
<td>20</td>
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<tr>
<td></td>
<td>Oct-Dec</td>
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<tr>
<td>42-43 St. Lawrence River</td>
<td>Feb-Apr</td>
<td>16</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td>May-Jan</td>
<td>18</td>
<td>9.3</td>
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<tr>
<td>44 Quebec to Montreal</td>
<td>All year</td>
<td>12</td>
<td>6.2</td>
</tr>
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</table>
b) Captain T. C. Pullen, who provided expert advice on bathymetry, ship handling and possible delays to ships.

c) Mr. E. E. Bustard, Consulting Naval Architect, who checked Albery, Pullerits, Dickson and Associate's equation for the performance of vessels in ice and gave valuable advice on the powering of large ice-breaking bulk carriers and the selection of propeller sizes.

Other valuable ice-related information was obtained through:

a) National Swedish Administration of Shipping and Navigation;

b) N. E. Markham, Chief, Ice Forecasting Central, Environment Canada; and

c) Mr. D. Lindsay, Northice Consultants.

Transport Canada made it possible for the Author to participate in the Arctic probe of the Louis S. St. Laurent in May, June, 1977, in order to study navigation and arctic conditions.

7.0 References


[2] NORTHERN ASSOCIATES LTD. - "Arctic Resources by Sea".


[6] SWEDISH-FINNISH BOARD OF WINTER NAVIGATION - "Vintersjofart med Stova Partyg i Bottenviken" (Translation: "Winter Navigation With Large Ships in the Gulf of Bothnia").


FIGURE 2
TRANSPORTATION ROUTES

SEE ENLARGED MAP
FIGURE 1

132
FIGURE 3: ICE CONDITIONS, LEG NUMBER 7

LOCATION: VISCOUNT MELVILLE SOUND, CAPE COCKBURN TO RAE POINT

MEAN FREEZE-UP DATE: SEPT 26 (269)  STD DEV. ± 10 DAYS
MEAN BREAK-UP DATE: AUG 7 (219)  STD DEV. ± 14 DAYS

LENGTH 140 km

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<tr>
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<th>Mean Conc First 10ths</th>
<th>Mean Conc Multi 10ths</th>
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<td>DEC</td>
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<tr>
<td>JAN</td>
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<td>3.7</td>
<td></td>
</tr>
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<td>3.7</td>
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<tr>
<td>APR</td>
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<td>6.3</td>
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<table>
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<th>Mean Thickness Multi m</th>
<th>RMS Thickness m</th>
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<td>2.9</td>
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</tr>
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<td>NOV</td>
<td>0.6</td>
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</tr>
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<td>DEC</td>
<td>0.9</td>
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<tr>
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<tr>
<td>FEB</td>
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<td>1.5</td>
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<td>APR</td>
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<td>JULY</td>
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<tr>
<td>AUG</td>
<td>0.7</td>
<td>3.0</td>
<td>1.4</td>
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</tbody>
</table>

MULTI CONCENTRATION (10ths)

PROB = GIVEN MULTI CONC.

0  2  4  6  8  10
MULTI CONCENTRATION (10ths)

MEAN ICE THICKNESS (m)

0  1  2  3  4
MONTH

S  O  N  D  J  F  M  A  M  J  J  A
FIGURE 4:
MULTI-YEAR ICE AVOIDANCE CRITERIA

FIGURE 5:
POWER VS DISPLACEMENT & ICE THICKNESS

SOURCE BUSTARD - IMPORTANCE OF SIZE IN AN ARCTIC SHIP AND AUTHOR
FIGURE 6: ICE RESISTANCE FORMULAE - 300,000 DWT. VLCC

ICE THICKNESS - 2.5m
ICE FLEXURAL STRENGTH - 518 KPa (75 psi)

1 KASHTELJAN
2 KASHTELJAN as revised by ASSUR
3 LEWIS - EDWARDS based on full scale test on LOUIS ST. LAURENT
4 ENKVIST (Ψ = 63°, µ = 0.13)
5 MATHEWS
6 CANADIAN COASTGUARD
7 APD
8 KLOPPENBURG
9 VANGE
Abstract

Transit performance of large barges in arctic waters from Lancaster Sound to Bridport Inlet has been examined, based on the survey of environmental data, model experiments in ice and some experiences with marine transportation in ice. Six barge bow types were designed and investigated through model experiments in both broken and level ice. Comparative studies on the transit performance of six candidate towing systems were made from the results of the model experiments and environmental data, taking into account the capabilities of an existing icebreaker and ice worthy tugs. As a result two towing systems are considered feasible for summer season transit.

1. Introduction

For marine transportation in ice covered waterways, various kinds of systems are operated such as icebreakers, ice worthy vessels and convoys. With the recent development of energy and mineral resources in the Arctic Regions, extensive studies have been made to find the system most appropriate to the materials and quantities to be transported, and the route, season, etc. to be used. As a result, some barge systems available in arctic waters are considered to be effective means for the transportation of large amounts of materials or products for a short period during the summer season, when ice conditions are moderate.

Mitsubishi Heavy Industries Ltd. conducted a comparative examination on the feasibility of some candidate barge systems for transportation of product plants in the Canadian Arctic. The route from Lancaster Sound to Bridport in Melville Island was defined for the study and environmental data were accumulated over several years by satellite imagery and normalized for the following investigations.

Six kinds of towing barge systems were designed on the basis of present knowledge on marine transportation in ice. The six bow types installed on the barge were designed and investigated by model experiments in broken ice. The best of these was chosen from the point of view of resistance performance in broken ice and was tested in level ice and broken channel ice.

Transit performance of the six candidate systems, composed of a barge, an icebreaker
and ice worthy tugs, was evaluated by using the ice data, the results of the model experiments and present knowledge of transportation in sea ice. As a result, two of the candidate systems are considered feasible, being capable of traversing the defined route within approximately 2-4 days during the period from the beginning of August to the end of September.

2. Design of ice worthy towing system

A barge was designed to carry some liquid gas product plants and to have capability as a storage barge in the arctic region. Several barges will have to be transported to the area through ice covered waters within a rather short period during the summer season.

2-1 Environmental data

The environmental data for the route from Lancaster Sound to Bridport was studied through the use of several years of satellite imagery (Fig. 1). Photomosaics of the ice conditions were assembled and optimum routes for selected time periods were chosen. Ice conditions along these routes were classified according to the following categories:

(a) Open water
(b) Small floes; Agglomeration of ice floes ≤ 500 m
(c) Large floes; Agglomeration of ice floes ≥ 500 m
(d) Large and small floes; Large floes with small floes filling the channel between them.
(e) Level ice; This category includes both newly formed grey ice at the end of open water season and white ice at the beginning of open water season.

Ice thickness was calculated using Bilello's ice degradation formula [1].

2-2 Candidate towing systems

The towing system is composed of a barge, ice worthy tugs and an icebreaker for tow or escort purposes. The capabilities and availabilities of current powerful icebreakers and ice worthy tugs were also taken into consideration, together with the Canadian Arctic Waters Pollution Prevention Act.

(1) Single tug tow, without icebreaker escort.
(2) Twin tug tow, without icebreaker escort.
(3) Twin tug tow, with icebreaker escort.
(4) Close-coupled tow, using icebreaker.
(5) Twin tug push, without icebreaker escort.
(6) Twin tug push, with icebreaker escort. (Fig. 2)

<table>
<thead>
<tr>
<th>Table-1 Principal Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Icebreaker</td>
</tr>
<tr>
<td>Lpp (m)</td>
</tr>
<tr>
<td>Bm (m)</td>
</tr>
<tr>
<td>dm (m)</td>
</tr>
<tr>
<td>Displacement (t)</td>
</tr>
</tbody>
</table>

2-3 Design of bows

Six bow types were designed and installed on the front of the barge for this study. The capability of a bow to override the ice and to push aside the ice was considered in the design.

Bow(0); No bow on the rectangular front of the barge.
Bow(1); Relies only on the barge weight overriding the ice.
Bow(2); Modification of Bow(1) in that the ends are somewhat tapered.
Bow(3); Further modification of Bow(1) with smooth curved geometry.
Bow(4); Vertical sides and a pointed bow.
Bow(5); Attempt to combine the best ice-overriding and ice moving capability. (Fig. 2)

Fig. 1 Transit Route and Ice Condition
2-4 Resistance characteristics

The following resistance characteristics are necessary for evaluation of the towing systems.

(1) Open water resistance; for Barge, Tug and Icebreaker
(2) Level ice resistance; for Barge, Tug and Icebreaker
(3) Level ice resistance with partial beam encounter; for Barge behind icebreaker in close-coupled tow
(4) Broken ice resistance; for Barge, Tug, Icebreaker
(5) Broken ice resistance with partial beam encounter; for Barge behind icebreaker in close-coupled tow

Formulae for ice resistance of barge were determined by resistance test in ice, and those for icebreaker and tugs were obtained from present knowledge on ice resistance.

3. Model experiments

Resistance tests were carried out on the barge with various bow types, as follows:

<table>
<thead>
<tr>
<th>Model (scale 1/48)</th>
<th>Wooden model finished by vinyl paint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test in broken ice</td>
<td>Synthetic model ice pieces (thickness 45 mm, $\mu = 0.12 - 0.13$)</td>
</tr>
<tr>
<td></td>
<td>Concentration; 50% and 90%</td>
</tr>
<tr>
<td>Test in level ice and broken channel</td>
<td>Saline ice (thickness 8.5 mm, strength 12 - 13 kPa, $\mu = 0.11$)</td>
</tr>
</tbody>
</table>

3-1 Testing in broken ice

Resistance tests on the barge alone and the barge towed by an icebreaker in close-coupled towing were conducted in both 50% and 90% ice concentrations. The environmental data and Kashteljan's empirical results [3] [4] were taken into account in the simulation of ice conditions.

The results of the tests are illustrated in Fig. 3. The ice resistance of the barge fitted with Bow (5) was found to be smaller than that of the barge fitted with the other bow types in the towing condition.
In the tests of the barge behind an icebreaker in close-coupled tow, two wing propellers were operated to keep actual thrust equivalent to that of a twin screw tug with approximately 15000 HP. The variation in resistance of the barge with these bows are small. However, Bow (5) is considered to be the best because of the observed behaviour of ice pieces around the bow.

Fig. 3 Resistance Tests in Broken Ice

Fig. 4 Model Tests in Broken Ice
3-2 Testing in level ice

Resistance tests on the barge with Bow (5) were carried out in level ice and in narrow channel ice. The narrow channel was adjusted to the same breadth as that of the icebreaker.

The results are shown in Fig. 5. In this figure it is noted that the shoulder part of bow makes a major contribution to the resistance of the barge.

![Fig. 5 Resistance Tests of Bow (5) in Level Ice](image)

4. Transit analysis

4-1 Resistance formulae

Resistance formulae were derived on the basis of data obtained from the model experiments and published data as shown in Table-2. Kashteljan's formula and Ryblin's formula were adopted as the fundamental form in the regression of model test results.

4-2 Transit performance

Transit performance (day time for traversing the subject sea route) was compared under the following assumptions.

1. Total resistance is the algebraic sum of the resistances of each element.
2. The ice concentration behind an icebreaker or ice worthy tug is 20% less than that ahead of it.
3. Large floe fragments or small floes extending several kilometers are either avoided by maneuvering or penetrated as level ice. The avoidance of large floes results in increasing the distance travelled by an amount proportional to the concentration\(^6\). Towing system speed is kept at 6 knots for impact load considerations.
4. Simulation models for penetration in broken ice.

<table>
<thead>
<tr>
<th>Large floes &gt; 500 m</th>
<th>Small floes 50 ~ 500 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concentration &lt; 70%</td>
<td>Concentration 80%-100%</td>
</tr>
<tr>
<td>Avoidance</td>
<td>Level ice model</td>
</tr>
<tr>
<td>Vs = 6 Kn.</td>
<td></td>
</tr>
<tr>
<td>Avoidance</td>
<td>Level ice model</td>
</tr>
<tr>
<td>Vs = 6 Kn.</td>
<td></td>
</tr>
</tbody>
</table>
(5) Simulation models for transit in large and small floes which characterize a wide track filled with a mixture of large and small floes.

<table>
<thead>
<tr>
<th></th>
<th>Large floes</th>
<th>Small floes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Concentration &lt; 70%</td>
<td>Concentration 80%-100%</td>
</tr>
<tr>
<td>Broken ice model</td>
<td>Broken ice model</td>
<td>Level ice model</td>
</tr>
<tr>
<td></td>
<td>Concentration &lt; 70%</td>
<td>Concentration 80%-100%</td>
</tr>
<tr>
<td></td>
<td>Finely broken ice model</td>
<td>Coarse floe model</td>
</tr>
</tbody>
</table>

(6) Available thrust

<table>
<thead>
<tr>
<th></th>
<th>Icebreaker</th>
<th>Ice worthy tug</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vs = 0 - 5 knots</td>
<td>Vs = 0 - 5 knots</td>
<td>Vs &gt; 5 knots</td>
</tr>
<tr>
<td>200 (t)</td>
<td>300 - 20 x Vs(t)</td>
<td>100 (t)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150 - 10 x Vs(t)</td>
</tr>
</tbody>
</table>

Total available thrust for any system is the sum of the elements involved in towing excluding escorting icebreaker.

A prediction of the time for traversing the sea route was made for each candidate system on the basis of the assumptions above by use of the environmental data collected during the July to October period for the years 1973-1977 and resistance characteristics mentioned in the previous section. The results are illustrated in Fig. 6.
In evaluating each system, the reliability in operational aspects has to be taken into account, i.e. maneuverability in ice, the risk of getting trapped in changing ice conditions, recent experiences with barges in sea ice, and so on.

Summarizing the above, it may be concluded that

(1) Systems 1, 4 and 5 are risky if there are changes in the ice conditions, in spite of feasibility in moderate ice conditions,
(2) System 2 is still risky in severe ice conditions, in spite of better transit capability than that of systems 1, 4 and 5,
(3) Systems 3 and 6 are feasible through the period from the beginning of July to the end of September.

These systems can traverse the defined sea route within about 2-4 days during the period from the beginning of August to the end of September, and within about 6-8 days in July. Thus, it is considered that system 3 (twin tug tow with icebreaker escort) and system 6 (twin tug push with icebreaker escort) are feasible and valid alternatives for this purpose.

Table-2 Resistance Formulae

<table>
<thead>
<tr>
<th></th>
<th>Icebreaker</th>
<th>Ice worthy tug</th>
<th>Barge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open water</td>
<td>1.06V^{1.62}</td>
<td>0.30V^{2}</td>
<td>1.4V^{2}</td>
</tr>
<tr>
<td>Level ice</td>
<td>152.2h^{2} + 33.88h^{0.5}V</td>
<td>81.38h^{2} + 38.72h^{0.5}V</td>
<td>133.4h + 194.5h^{2} + 133.2hV</td>
</tr>
<tr>
<td>Narrow channel behind icebreaker</td>
<td></td>
<td></td>
<td>133.4h(1 - B/Bo) + 194h^{2} + 133.2hV + 0.573</td>
</tr>
<tr>
<td>Finely broken ice (very fine floe)</td>
<td>307.9K_1\sqrt{rh} + 0.582K_2 \times (rh)V + 0.046K_3 \times (rh)V^{2}</td>
<td>97.3K_1\sqrt{rh} + 0.416K_2 \times (rh)V + 0.05K_3 \times (rh)V^{2}</td>
<td>103.1(S - 0.27)\sqrt{rh} + 23.26(1 + 0.191S) \times (rh)V</td>
</tr>
<tr>
<td>Coarsely broken ice</td>
<td>(C_K + 9.0V^{1.3})h^{1.2} \times (B/23.5)</td>
<td>(C_K + 9.0V^{1.3})h^{1.2} \times (B/23.5)</td>
<td>7.158(C_K + 9.0V^{1.3}) \times h^{1.2}</td>
</tr>
<tr>
<td>Broken ice with partial beam encounter</td>
<td></td>
<td></td>
<td>14.79(S - 0.27)\sqrt{rh} + 21.13(1 + 0.191S) \times (rh)V</td>
</tr>
</tbody>
</table>

V ; Ship speed (m/s)  h ; ice thickness (m)  B ; Breadth of icebreaker and tug (m)  Bo ; Breadth of barge (m)  K_1,K_2,K_3 ; Coefficient in Ryblin's formula  \sqrt{rh} ; Mean size of ice floe in Ryblin's formula  S ; Concentration/100  C_K ; Coefficient in Kashteljan's formula
5. Conclusions

Based on the investigations above, the following conclusions were obtained.

(1) The barge transfer from Lancaster Sound to Bridport is feasible during the period from the beginning of July to the end of September.

(2) Two kinds of barge systems i.e. (a) twin tug tow with icebreaker escort (b) twin tug push with icebreaker escort are valid alternatives for this purpose.

(3) These systems can traverse the defined sea route within 2-4 days during the period from the beginning of August to the end of September.

6. Acknowledgement

The authors wish to express their gratitude to Mr. R.Y. Edwards Jr., President of Offshore Technology Corporation and the staff of ARCTEC CANADA Ltd. for their cooperation.

References


PERFORMANCE OF ICEBREAKER YMER
ON THE SWEDISH ARCTIC EXPEDITION
"YMER 80"

Göran Liljeström, Arctic Project Co-ordinator
Kaj Lindberg, Naval Architect
Götaverken Arendal Sweden

ABSTRACT

In summer 1980 the Swedish icebreaker Ymer spent 100 days in the Arctic to commemorate the centenary of the first North-East Passage transition. The expedition known as "Ymer -80", carried out an extensive scientific program, a part being the ship technology program by Götaverken Arendal.

Ymer travelled in summer ice conditions from Spitzbergen to north-eastern tip of Greenland, 82° 24' N, 16° 26' W, back to Spitzbergen and north-east to Franz Josef Land, 82° 25' N, 45° 05' E.

The ice conditions were recorded by a television camera on videotape. The average ice conditions are described.

In addition, shaft torque, thrust and rpm were recorded on a continuous basis. The instrumentation for these measurements described together with some machinery performance data.

In the forebody, at 27 locations strain gauges were installed for measuring stresses in the shell plating, frames, stringers and web frames. Representative stress levels for various ice conditions and speeds were recorded and are presented as a separate appendix to the paper.

The performance of the Ymer was above expectations and the ship technology programs was very successful resulting in valuable information that undoubtly will be useful for designing new ships and offshore systems for the Arctic.
INTRODUCTION

This paper covers the Ship Technology Program of the Swedish Arctic Expedition - "YMÉR 80". This scientific expedition was launched to commemorate the centenary of the first transition of the North-East Passage by A.E. Nordenskiöld. The expedition covered almost all scientific disciplines for providing an integrated research program for a most interesting part of the Arctic. The expedition was carried on board the Swedish icebreaker "YMÉR" and supplemented by a number of shore parties on Spitzbergen.

The voyage started June 24, 1980, when Ymer took her departure from Stockholm, and ended when she entered Stockholm harbour October 1, 1980, one hundred days later. During this period she had steamed as far west as off the north-west tip of Greenland, to the east, north off Franz Josef Land and several voyages around Spitzbergen. In the process she had travelled nearly 11000 miles of which abt 5350 miles were in icy waters.

The Ship Technology Program was carried out by Götaverken Arendal on the second and third legs of the expedition, i.e. from August 22 to September 24, 1980.

THE SHIP - ICEBREAKER YMÉR

The icebreaker "YMÉR" is one of a series of 5 icebreakers for Sweden and Finland. The vessels are built acc. to DnV class 1A1 and Finnish/Swedish ice class IA Super with strengthening of hull and appendages which by experience are required for icebreakers.

The vessel has following main particulars:

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length over all (excl towing notch)</td>
<td>104.6 m</td>
</tr>
<tr>
<td>Length between pp</td>
<td>96.0 m</td>
</tr>
<tr>
<td>Breadth (max.)</td>
<td>23.8 m</td>
</tr>
<tr>
<td>Breadth (at CWL)</td>
<td>22.5 m</td>
</tr>
<tr>
<td>Draught (max.)</td>
<td>8.3 m</td>
</tr>
<tr>
<td>Draught (at CWL)</td>
<td>7.3 m</td>
</tr>
<tr>
<td>Displacement at CWL</td>
<td>7900 tonnes</td>
</tr>
</tbody>
</table>

As can be seen from the arrangement (fig. 1) the vessel is divided by 9 watertight bulkheads and 3 coastal decks, of which the upper deck is the strength deck. The hull is transversely framed with frame spacing 800 mm and webs at every third frame. Frames, stringers and webs in the ice belt are E-grade with minimum yield strength of 33 kp/mm². Hull plating is also of E-grade and designed acc. to plastic deformation theory. The hull plating in the ice belt has the following minimum thickness: bow 32 mm, midship 28 mm and stern 30 mm.
Propulsion machinery is like in previous Baltic icebreakers diesel-electric and consists of 5 diesels coupled to five DC generators, which supply 4 DC propeller motors, 2 for stern propellers and 2 for bow propellers. The main diesels are Wärtsilä built S.E.M.T. - Pielstick 12 PC 2.2, each of which has an output of 5 000 Bhp at 485 rpm.

To lower the friction between hull and ice and to minimize the risk of getting stuck, the bow propellers are so designed that their slipstream lubricates the hull. "YMER" has also after delivery been painted with INERTA 160, the low friction coating specially developed to withstand the wear of ice. Under heavy ice conditions when these precautions are not sufficient, the vessel has to be heeled continuously.

THE SHIP TECHNOLOGY PROGRAM

The aim of this program was exclusively to determine the ability of the ship to proceed through ice, that is, the ship's effect on ice and on ship operations under Arctic conditions.

The program consisted mainly of following subsections:

1. Obtain data on ship speed, ice conditions, analyze the data and correlate the speed to ice characteristics and thrust.
2. Obtain thrust, torque, rpm on the propulsion system during ice transit, analyze the data and relate it to data under item 1.
3. Measure strains in the hull during ice transit and relate this to the information obtained under item 1.
4. Testing of night vision equipment as navigational aids.

This program was successfully carried out by Götaverken Arendal, resulting in about 600 hours of ice transition on video tape and strain measurement recordings under a number of different ice conditions.

INSTRUMENTATION

Part of the instrumentation such as TV equipment was tested in the Bay of Bothnia during the winter before. The final mounting of the camera and cables was done at shipyard prior to expedition. The strain gauges and cables were installed on same occasion.
- **TV equipment**

For recording ice conditions and the performance of "YMER" a low light level TV camera, make RCA TC 1006, in an environmental resistant sealed housing, was installed at the top of the wheelhouse. The camera was fitted with a 12.5 mm lens, giving a good over all view of the ice field ahead of the stem. A RCA TC 3450X Recorder with time/date generator and time-lapse capability together with a RCA TC 1209 video monitor, were placed in the wheelhouse. On the monitor a special calibrated grid was fitted, giving distances forward of the bow. This facility provided the possibility of determining the ship's speed using the timing on the screen. Use of the time-lapse video recorder allowed up to 24 hours of video information to be stored on a 120 minutes tape.

An additional monitor was also provided in the instrumentation container on 3rd Bridge Deck.

During the latter part of the voyage an additional TV-camera was tested for the purpose of detecting ice in open water.

- **Machinery**

Measurements on the propulsion machinery were performed on forward port shaft and aft port shaft.

The measurements can be divided in two categories:

- power elements as integrated values for 10 second intervals
- dynamic loadings measurements of torque and thrust.

For measuring torque fore and aft, strain gauges together with telemetric transmitters were installed on the shaft. Revolutions were recorded by means of photo-electric equipment on each shaft.

In the container the results could be read on a "powerlog" and they were also continuously printed every 10 seconds. Printout is shown in a later section. Thrust and torque could not be recorded during astern manoeuvring.

Dynamic variations in torque and thrust were measured by means of frequency-current converter and same strain gauge and telemetry equipment already described. Signals were connected to a multi-channel UV-recorder.
Hull

The stresses in plating, framing, stringers and webs were measured by means of strain gauges. A total of 27 gauges, type Hottinger LP 21, located according to following figure 1, were used. The distribution of gauges on the structural members was:

- plating: 8 gauges
- framing: 13 gauges
- stringers: 3 gauges
- webs: 3 gauges

The 7 gauges installed in the forward ballast tank were all protected by a special plastic coating. Three multi-conductor cables connected the gauges with the recording equipment in the instrumentation container. The gauges were divided in 8 groups, 6 groups for measuring the character of impacts and 2 groups for the random nature of impacts. The first 6 groups were plotted on a 6 channel UV-recorder. The latter 2 groups were recorded on a 4-channel FM instrumentation tape recorder, type Bruel & Kjaer 7003. For simultaneous viewing of a signal from a gauge, a dual trace oscilloscope, Tektronix Model 326 was used. Unfortunately the unusual large number of radio transmitters on board for this very expedition caused some disturbances.

NOTES ON VIDEO AND LIGHT INTENSIFYING EQUIPMENT

Beside the fixed TV camera installation for the recording of the progress of Ymer through the polar ice, a very low light level camera was tested aiming at early detection of ice in darkness. The camera used was a RCA TC 1030/H10 with zoom lens 16-160 mm (10X) f/1.8 with a TC 1430 controller. This camera gives useful pictures with as little light as quarter moonlight. The camera was mounted on a tripod on the bridge.

Unfortunately the light conditions did not allow us to test the camera for more than a couple of nights before leaving the ice.

The recordings obtained very excellent and floating ice, varying from floes about 5-10 m to 50 m were easily spotted at distances 1200-2000 m in cloudy night conditions for that time of the year. These tests confirm the potential of TV equipment of this kind in aiding ice navigation.

A medium range night observation device was also tested along the TV equipment. This handheld device, make Varo Inc, Model 9875, weighing 3.2 kg was used from the bridge surveilling the surrounding seas. Small ice floes were easily spotted at distances up to 1600 metres. These floes were not spotted on the radar. Earlier tests in Greenland waters have also proved the value of this device. Growlers were detected and safe manoeuvering was made easier.

The conclusions from Ymer are, there is optronic equipment available today, that can give valuable support in arctic shipping operations. Next generation of equipment will even to greater extent increase this potential.
ICE CONDITIONS ENCOUNTERED

- The westbound route (figure 2)

From Longyearbyen Ymer steamed in open water to 79° 08' N and 02° 57' E where Ymer carried out scientific work in the marginal ice zone during August 23-26. The following days on route to Greenland Ymer encountered both first and multi-year ice varying between 1-2 metres, occasionally even thicker ice. Multi-year flows 4.2 m thick were measured. New ice had formed in between old floes, abt 10 cm thick. Ymer made very good progress.

On August 29-30, Ymer proceeded along the Greenland coast in open water. Very heavy landfast ice was observed along with a number of grounded tabular icebergs. The height was estimated to 30-50 metres and depth varied from 100-150 m on the echo-sounder.

After the short stop near "Station Nord" Ymer proceeded north on August 30, to 82° 24' N and 16° 22' N. The ice consisted of old ice, ridged and hummocked. Multiyear ice in excess of 6 m was measured. A few leads in north-southerly direction were found. Helicopter reconnaissance showed that the ice was too difficult to break and it was decided to return same path back southwards. The progress of Ymer was in this type of ice totally dependent on existing leads.

Ice reconnaissance by a Danish Hercules aircraft on afternoon August 30, confirmed the severe ice conditions futher north but ice conditions in a easterly directions were reported 7/10 FY/OLD. Ymer continued on its southeasterly course during September 1-5. The ice was mainly heavy multi-year ice, cracked with a few leads, covered by new ice 10-15 cm. New snow, abt 5 cm, made it difficult to locate navigable tracks. The ice in this area must be considered as the heaviest ice navigable by Ymer.

On September 6-7 the ice got lighter when Ymer approached the ice edge again.

- The eastbound route (figure 3)

West and north of Spitzbergen Ymer only ran into patches of very open pack. From abt 81° 30' N and 23° E Ymer came into close pack ice, 1-2 m thick. On September 12-14 on the easterly course Ymer had great difficulty breaking the ice. Southerly winds pressed the ice which was abt 1-2 m thick. Even using the heeling system, the average speed of Ymer dropped to 2 knots.

When heading south again the ice pressure released and Ymer got out in open water on September 15.
Ymer continued eastwards and on morning September 16, she got into very open pack. By midnight September 17, the ice became heavier. Southeasterly winds had pressed the ice for over a week but Ymer still made 2-4 knots in the close pack. On September 18, the wind shifted to east, later to northeast which opened leads in east-westerly direction. Ymer made an average 8 knots on the westerly course, a very unexpected progress at this latitude. Ice was both first and multi-year varying between 0.8 - 2 m. The progress south on course abt 40°E went through close pack 7-9/10. Ymer made good use of leads and cracks and the pack ice got lighter as Ymer was reaching the ice edge. Floe sizes were quite large causing no problems for Ymer to make passage around and still maintain main course.

Summarizing the ice conditions in summer 1980 the judgement must be a somewhat more difficult season for the particular areas where Ymer was operating. This appears to be the situation further east too, where the Soviet shipping has had greater difficulty than normally.

POWER MEASUREMENTS

During icebreaking a number of elements were measured on the machinery. These were power, thrust and rpm. From these elements torque may be calculated.

Measurements were made during different ice conditions and the recorded elements cover a representative range of Ymer's "power levels". The total time of recordings is abt 11 hours.

An analysis of the elements is presented in a separate appendix.

STRAIN MEASUREMENTS

The aim of these measurements, was to study the character of the ice impact on the shell and to study the character of the structural response in the hull structure. Further the random nature of the impact was studied.

The character of the impact can be described by the magnitude of the ice pressure and by the size of the area of impact. The random nature of the impacts and the response can be described by statistical methods.

As previously described the gauges were located in three areas. Enclosed table shows the detailed grouping.

Measurements in the rear area were made on shell plating and framing. By gauges Nos. 4, 5, 7 and 8 the sheer forces on frame 87 1/2 were measured.
In the mid area the horizontal length of impacts was studied by gauges Nos. 9-14 located on frames. The depth of impacts was studied by gauges Nos. 16-18 (11 and 15) located on the shell plating.

Responses in a stringer and a web were measured in the forward area. The depth of the impacts was studied by gauges Nos. 20 and 21.

Ice conditions and speed of the vessel were noted for each measurement.

Figure No. 1 shows a typical recording of group No. 3. The ice condition was close pack 7/10 - 8/10 FY abt 1.7 m thick. Speed of Ymer 2.5-3 knots.

Peak value measured was 2600 kp/cm² or abt 80% of the yield stress. This stress occurred in shell plating fore (group No. 5). Ice condition on this occasion was 7/10-9/10 FY close pack abt 1.5 m thick with new ice abt 10 cm in between. The speed of Ymer was 3.6 knots.

Measurements were successful except for the fact of some disturbances. However a lot of representative and good recordings were made. A detailed analysis with results is presented as a separate appendix.

SUMMARY

From scientific point of view the expedition was very successful. The scientists highly appreciated Ymer as research platform. There was plenty of space and Ymer could weightwise carry more than needed for the scientific work.

Ymer performed above expectations in the polar ice. The icebreaking capacity was better than expected. The overall judgement is that Ymer is an excellent icebreaker, in hands of a skillful crew with vast icebreaking experience.

The research program by Götaverken Arendal added a huge amount of information on a ship under arctic conditions to the knowledge already within the organization from previous years of R&D work in this field.
FIGURE 1.
ARRANGEMENT OF YMER AND THE INSTRUMENTATION BY GOTAVERKEN ARENDAL.
FIGURE 2. THE WESTBOUND ROUTE
FIGURE 3. THE EASTBOUND ROUTE
CORRELATION OF UNDER-ICE ROUGHNESS
WITH SATELLITE AND AIRBORNE THERMAL
INFRARED DATA

Leonard A. LeSchack, President
LeSchack Associates, LTD
U.S.A.

This report, based on empirical data, concludes that a correlation has been found between easily obtainable sea ice surface temperature and under-ice roughness data which are obtainable only at great expense. Under-ice roughness is valuable in evaluating acoustic attenuation beneath the Arctic ice and is expressed in terms of either root-mean-square (RMS) ice depth or standard deviation about the mean ice depth, both of which are closely correlated. In previous work, a comparison of the skewness of the temperature distribution derived from NOAA VHRR thermal infrared satellite data of the Beaufort Sea taken in April 1974 was made with the RMS under-ice roughness derived from under-ice profile data recorded by the SSN SARGO in February 1960 taken at essentially the same locations. Five data sets were chosen ranging from the highly deformed and hence thicker ice area near the Canadian Archipelago to thinner ice off the coast of Alaska near Prudhoe Bay. A very high negative correlation was observed. In a second study, under-ice data recorded in April 1976 by the SSN GURNARD was correlated with the skewness of temperature distributions derived from NOAA VHRR TIR data recorded in March 1976 over nominally the same area of the Beaufort Sea. Because of the relatively high resolution, i.e., 10 N. miles, some data set misregistration was anticipated, but by moving the satellite data matrix about 10 NM to the west, northwest or southwest, a good registration was found and a good statistical correlation was obtained. This slight matrix shifting appears significant because it is consistent in both direction and magnitude with the actual measured ice drift between March and April 1976. Also, shifting in any of the other three quadrants shows no correlation. In the current study it is shown, perhaps more graphically than the others, the correlation of under-ice data recorded by the British nuclear submarine HMS SOVEREIGN between 18-21 October 1976 with airborne IR data recorded during the same period over the submarine track by a Canadian Forces Argus aircraft. The airborne TIR data are recorded in the form of imagery on a scale large enough to see the individual features of the ice surface. A scanning microdensitometer generated histograms of film density values that are proportional to ice surface temperatures. The skewness of the histograms were correlated with the standard deviations about the mean ice depth for the associated submarine track. In all three examples, when the RMS ice depth range was between 4 and 8 m, corresponding to a standard deviation of ice depth ranging between 2 and 6 m, there is a strong, negative linear correlation between the skewness of the temperature distributions, whether measured from satellites or aircraft, and the under-ice roughness measured by submarine upward-looking sonar.
1. Background

Since 1973 the author has been examining the statistical properties of Arctic sea ice thermal infrared (TIR) temperature measurements obtained with VHRR satellites [1,2,3]. The thrust of that research was to determine whether it is feasible to objectively (i.e., numerically) identify different sea ice types and develop a system for automated mapping of sea ice. At POAC-75 where the results of the above studies, along with those of submarine under-ice analyses were presented [4], Buck [5] made the observation that it would be valuable to his under-ice acoustic work if some correlation could be found between the satellite and submarine data sets, both recorded in the eastern Beaufort Sea. Although the idea of making a large area correlation between submarine and airborne or satellite data of sea ice was not new [6], this was the first time that two data sets usable for this task became available. Accordingly, with both the availability of the necessary data and the impetus provided by a concrete use for any correlations developed, work was begun in 1976 to investigate the relationship between satellite and submarine under-ice roughness data.

2. Under-Ice Roughness

The variable that we wish to ultimately predict is under-ice roughness, since this variable strongly influences acoustic propagation beneath the ice. Several mathematical definitions of under ice roughness can be formulated; however, only two forms appear significant to our problem. These are the root-mean-square (RMS) under-ice depth and the standard deviation, $\sigma$, of the under-ice depth, respectively defined as:

\[
\text{RMS} = \left[ \frac{\sum_{l=1}^{N} x_{l}^2}{N} \right]^{1/2}
\]

and

\[
\sigma = \left[ \frac{\sum_{l=1}^{N} (x_{l} - \bar{x})^2}{N} \right]^{1/2}
\]

where

- $x$ = value of under-ice depth in m (depth of ice from water line)
- $N$ = number of depth values along a profile

At the outset of this work, RMS under-ice roughness was the parameter of choice. This was because not only is it a measure of roughness, but it appears to be a good indicator of overall ice deformation for a given ice surface area, since any significant departure from the underformed equilibrium ice depth over the Arctic Ocean (about 3 m) can occur only through building ice ridges and keels or opening of leads and polynyas.

As the work progressed, however, the standard deviation, $\sigma$, of the ice depth was suggested by Buck [7] as being more meaningful to the acoustic attenuation problem than was the RMS ice depth; therefore, in subsequent work, $\sigma$, was used. There is a close correlation between these measures of under-ice roughness, as well as the "ice roughness factor" suggested by Buck [8],[9]. Both RMS and $\sigma$ are used in this study.
3. Preliminary Correlation of RMS Ice Depth with Satellite TIR Temperature Data

Evidence from previous research [1,2,3] revealed that when a statistically sufficient number of surface temperature measurements were made of the undeformed multi-year ice surface in the Arctic, using airborne or satellite TIR scanners, the measurements would fall into a normal or Gaussian distribution. As the ice deformed, the surface temperature distribution skewed toward the side of the curve corresponding to the lower temperatures. The temperature distribution appeared to skew more with the greater amounts of deformation.

The physical basis for this relationship appears to relate to surface ice roughness only indirectly. The reasons for an increased number of lower infrared temperature measurements with increased ice deformation appear to relate to how the TIR scanner perceives the roughened surface, rather than to any direct sensing of lessened heat flow due to thicker ice in deformed areas. From an analysis of the sea ice surface, the following factors seem to be of greatest importance. With increasing ice deformation:

- there is an overall increase in snow catchment, thus reducing the surface temperature directly and because snow's emissivity is less than that of sea ice, reducing also the TIR scanner's perception of the surface temperature.
- there is an increase of shadowing of the sun's radiation during daylight hours, relative to an undeformed surface. These shadow areas will naturally be cooler than irradiated areas.
- there is an increase in aerodynamic surface roughness. This effect, depending on conditions, may result in either slightly increased or decreased surface temperature.

It has been shown by statistical analysis of radiant temperature data distributions for given areas of the ice surface, as obtained by the NOAA VHRR satellite, that an objective identification can be made of the various surface types, i.e., first-year ice, multi-year ice, water, landfast ice, land, and clouds [1,2]. LeSchack [5] notes, for example, that an area of all multi-year ice produces a normal distribution of temperature data points with a mean value close to that predicted by the model of Maykut and Untersteiner [10], with a skewness of zero and a kurtosis (peakedness) of about 3. If increasing areas of warmer first-year ice are included in the area surveyed, the temperature distribution skews positively. If the area includes colder clouds, the distribution skews strongly in the negative direction (≤ -0.20). If there is a lot of ridging of the multi-year ice, the temperature appears colder than the mean multi-year ice, thus producing a modest negative skewness. Therefore, if by the selection techniques discussed in [2], a large expanse of ice surface can be identified without cloud cover, it should be possible to predict the general extent of surface ridging by analyzing the skewness of the temperature distribution.

In a recent report by the author [9], a comparison of the skewness of the temperature distribution derived from NOAA VHRR TIR satellite data of the Beaufort Sea taken in April 1974 was made with the RMS under-ice roughness derived from under-ice profile data recorded by the SSN SARCO in February 1960 taken at essentially the same locations. Five data sets were chosen ranging from the highly deformed and hence thicker ice area near the Canadian Archipelago to thinner ice off the coast of Alaska near Prudhoe Bay. A very high negative correlation was observed.

In a second study discussed in [9], under-ice data recorded in April 1976 by the SSN GURNARD were correlated with the skewness of temperature distributions derived from NOAA VHRR TIR data recorded in March 1976 over nominally the same area of the Beaufort Sea. The SSN GURNARD cruise track of some 780 N. miles was divided into 63, 10 N.
mile (16 km) sections. The NOAA VHRR TIR data were similarly grouped into 10 N. mile squares that nominally corresponded to the positions from where RMS under-ice data were gathered. Twenty useable skewness values were obtained with the RMS under-ice data. Because of the relatively high resolution, i.e., 10 N. miles, some data set misregistration was anticipated, but by moving the satellite data matrix about 10 NM to the west, northwest or southwest, a good registration was found and a good statistical correlation was obtained. This slight matrix shifting appears significant because it is consistent in both direction and magnitude with the actual measured ice drift between March and April 1976. Also, shifting in any of the other three quadrants shows no correlation.

4. Correlation of Simultaneous Airborne TIR Data with Under-Ice Data by HMS SOVEREIGN

4.1 Introduction

A third study, presented here, was then conducted that shows, perhaps more graphically than the others, the correlation of under-ice data recorded by the British nuclear submarine HMS SOVEREIGN between 18-21 October 1976 with airborne TIR data recorded during the same period over the submarine track by a Canadian Forces Argus aircraft. In many ways, this example is better than the previous two. The airborne TIR data are recorded in the form of imagery on a scale large enough to see the individual features of the ice surface. Every attempt was made to cover the exact track of the submarine within hours of its passing beneath the ice.

Figure 1 shows the submarine track. The data discussed in this example were collected along Segment AB. Wadhams [11] divided Segment AB in 10 sections, each approximately 97 km in length. They are numbered as in Figure 1. The airborne TIR data used in this correlation were recorded from an altitude of 4000 ft (1200 m) between 1910 hrs and 2109Z hrs on 19 October 1976 along Segment AB. The exercise was referred to as "Brisk Laser #1" by the Canadian Defense Research Establishment.

4.2 Data Analysis

4.2.1 Under-Ice Data

As in the previous Beaufort Sea examples cited, the object of this work is to derive an under-ice roughness parameter from the profile data, and to correlate it with a parameter easily derived from the airborne TIR data that physically relates to the topside roughness of the ice. Although in the previous examples the RMS ice depth was used as the under-ice roughness parameter, recent work by Buck [8] indicates standard deviation, \( \sigma \), from the mean under-ice depth is more meaningful for Arctic acoustics. Therefore, \( \sigma \) will be the under-ice roughness parameter used in this example.
Wadhams [11] prepared histograms (probability density functions) of the under-ice depths recorded in each of the 10, 97 km zones along Segment AB. These are shown in Figure 2. For each of the 10 zones P(H), the frequency of occurrence, is plotted against the ice depth. In this example, the standard deviation of the mean ice depth was computed from the Wadhams' histograms using the following relationship:

\[
\sigma^2 = \frac{f_1 (d_1 - \bar{d})^2 + f_2 (d_2 - \bar{d})^2 + \ldots + f_n (d_n - \bar{d})^2}{(f_1 + \ldots + f_n - 1)}
\]

where

\( \sigma \) = the standard deviation
\( f_i \) = the frequency of occurrence for each 1 m depth cell
\( d_i \) = the ice depth at the center point of each 1 m cell

(i.e., \( d_1 = 0.5 \text{ m} \), \( d_2 = 1.5 \text{ m} \), \( d_3 = 2.5 \text{ m} \), etc.)

The computed values for \( \sigma \) are listed in Table 1.

4.2.2 Airborne TIR Data

As previously shown [1,2,3], when many TIR temperature measurements are made over a sea ice surface, each ice type, i.e., multi-year ice, first-year ice and thin ice, has associated with it a normal or Gaussian distribution of temperatures. For example, if a given ice surface is comprised wholly of multi-year ice, an adequate series of surface temperature measurements produce the familiar "bell-shaped" distribution curve. If there is multi-year and first-year ice in the surface area measured, two slightly overlapping normal distribution curves are produced. When thin ice is also included, three overlapping curves can be seen. The area under each curve is proportional to the amount of surface represented by each type.

FIGURE 2: Probability density functions of draft. Plots are numbered according to divisions seen along Segment AB in Figure 1. (after Wadhams, 1978)
In the present study the airborne TIR data were recorded over the submarine track in
the form of imagery on 70mm film. This section of imagery had been carefully
annotated by Dr. Bev Young of the Canadian Department of National Defense and the
geographical position of the imagery can be easily ascertained. Between 1912 and 2040
hrs, aircraft altitude was constant, there was no cloud cover, and the IR instrument
operator left the gain control constant so that the photographic gray scale, although
not temperature calibrated, maintained the same temperature/gray scale relationship.
During this period, imagery covering sections 4-8 of the submarine track was obtained.

Five sections of imagery, representing the ice surfaces close to the center of each
segment, were scanned with a Photometric Data Systems Inc. Series 1000 Microdensito-
meter\(^1\) to obtain quantitative values proportional to the ice surface temperature.
Each area scanned was chosen so that a majority, if not all, of the surface was multi-
year ice. The decision was made by photo-interpretive means. A straight scan line
19 cm long through the center of the image, parallel to the line of flight, was made.
A circular aperture 86 um in diameter was used to measure density. A density value
was recorded every 86 um along the scan for a total of 2210 samples per scan.

In the imagery used, the lower the film density, the warmer the surface; hence, the
thinner the ice. No functional relationship between the film density and the actual
surface temperature is known, since the IR scanner was not calibrated. All that can
be said is that the range of thin ice to multi-year ice results in a monotonic
increase in film density from approximately 1.0 to 2.0 density units for all five
scans. Since this range is typically in the linear portion of the D-log E or Hurter-
Driffield curve (for example, see Thompson [12] p. 248), it can be assumed that the
density/ice thickness relationship is at least constant among the five scans, and
this is sufficient to evaluate variations of skewness of the density (or ice thick-
ness) distribution for these five samples.

4.3 Correlation of Under-Ice Roughness with Skewness of Surface Temperature
Distribution

Figures 3 and 4 show positive images of two of the five negative imagery sections
actually scanned for the present analysis. Beneath each image is a histogram of the
film density values derived from the scan. Where there is multi-year ice only in the
imagery, the histogram of density values suggests a single normal distribution. Over-
lapping normal distributions can be seen when first-year or thin ice is observed in
the imagery.

For this work, only the statistics of the multi-year ice distribution are used. Where
there is an overlapping first year distribution, the technique of subtracting the
estimated first-year ice histogram from the multi-year histogram is used (see for
example, Mack [14], p. 140). From the histogram of the multi-year ice, the skewness,
s, or the amount that the "bell-shaped" curve leans to the left or the right, was
determined as follows:

\[
S = \left( \frac{M_3}{M_2^3} \right)^{1/2} \quad (4)
\]

and

\[
M_r = \frac{f_1 (x_1 - \bar{x})^r + f_2 (x_2 - \bar{x})^r + \ldots + f_n (x_n - \bar{x})^r}{f_1 + f_2 + \ldots + f_n} \quad (5)
\]

\(^1\) Courtesy Research Institute, U.S. Army Engineer Topographic Laboratories, Fort
Belvoir, Virginia.
where

\[ M_r = \text{the } r \text{th moment about the mean} \]

\[ S = \text{skewness} \]

\[ x_i = \text{the film density values (the center of each class interval is chosen)} \]

\[ f_i = \text{the frequency of occurrence of } x_i \]

The values of the skewness as computed from (4) and (5) and from the histograms shown in Figures 3 and 4, as well three others not reproduced here, are listed in Table 1. To be consistent with the two examples cited earlier [9] we are defining positive skewness as a skewing of the temperature distribution toward higher temperatures, and negative skewness as a skewing toward lower temperatures.

When the values of skewness of the distribution of ice surface temperatures are compared with the measured under-ice roughness, i.e., standard deviation of ice depth, as in the previous studies, a clear functional relationship exists, as shown in Figure 5. The distribution of surface temperature measurements of sea ice skews toward the colder, or thicker ice as deformation increases. In the range of standard deviation of ice depth covered in this example, i.e., between 4.3 and 4.8 m, the relationship is linear and negative.

FIGURE 3: TIR image (above) of central portion of region 4 in Segment AB. Approximately 10 km have been covered by the 19 cm microdensitometer scan indicated between the arrows. The histogram of film density values is displayed below. The two minor distributions to the left are associated with thin ice and first-year ice. The major distribution is associated with multi-year ice and has a skewness of -0.475.
In all three examples, when the RMS ice depth range was between 4 and 8 m, corresponding to a standard deviation of ice depth ranging between 2 and 6 m, there is a strong, negative linear correlation between the skewness of the temperature distributions, whether measured from satellites or aircraft, and the under-ice roughness measured by submarine upward-looking sonar.

TABLE 1: Comparison of standard deviation about the mean ice depths for the 10, 97-km segments shown in Figure 1 with the skewness of the film density distributions derived from 5 sets of imagery such as illustrated by Figures 3 and 4.

<table>
<thead>
<tr>
<th>AREA (from Figure 1)</th>
<th>STANDARD DEVIATION (ice, in m)</th>
<th>SKEWNESS (TIR histograms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.995</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>4.266</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>4.557</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>4.345</td>
<td>-0.475</td>
</tr>
<tr>
<td>5</td>
<td>4.783</td>
<td>-0.983</td>
</tr>
<tr>
<td>6</td>
<td>4.677</td>
<td>-0.830</td>
</tr>
<tr>
<td>7</td>
<td>4.617</td>
<td>-0.713</td>
</tr>
<tr>
<td>8</td>
<td>4.554</td>
<td>-0.580</td>
</tr>
<tr>
<td>9</td>
<td>4.057</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>4.735</td>
<td></td>
</tr>
</tbody>
</table>

FIGURE 4: TIR image (above) of central portion of region 5 in Segment AB. Approximately 10 km have been covered by the 19 cm microdensitometer scan indicated between the arrows. The histogram of film density values is displayed below. The scan covers all multi-year ice, hence there are no minor distributions. The skewness of the distribution is -0.983.
5. Conclusions

Although different units of measurement, as dictated by the three available data sets, were used in the correlations to date, one thing is clear from the data presented here. For under-ice roughness measured in terms of RMS ice depth between the ranges of 4 m and 8 m, or in terms of standard deviation about the mean in ranges of 2 m and 6 m, there is a good negative linear correlation with the skewness of the temperature distribution of the corresponding ice surface, where increasing negative skewness is defined as the temperature measurements skewing toward lower temperatures.

Experimental evidence reveals a definite relationship. To define the relationship numerically, the parameters being correlated must be expressed consistently. This is easily done with the under-ice parameter which is measured in a straightforward manner, and can be expressed as standard deviation or RMS ice depth, and appears conservative over time [14].

On the other hand, the skewness of the surface temperature distribution, which appears to be the most sensitive measurement of variations in ice surface roughness found, has a numerical range that varies with the number of measurements used in its determination and most probably, the time of year the measurements were made. As a result, it is likely that a given under-ice roughness predictive equation must be developed from TIR data recorded at essentially the same time.

It is concluded that the under-ice roughness of multi-year ice may be predicted from airborne or satellite TIR measurements. An universal predictive equation has yet to be developed, however, since sufficient data are not presently available.

![Figure 5](image)

**FIGURE 5:** Correlation of the skewness of the TIR temperature distributions with the standard deviations of ice depths for the same regions. The specific regions are noted on the plot.
6. **Acknowledgement**

The airborne TIR data were provided by Dr. Bev Young of the Canadian Defense Research Establishment in Ottawa, and the HMS SOVEREIGN under-ice data were provided by Dr. Peter Wadhams of the Scott Polar Research Institute, Cambridge, England. This work was conducted under U.S. Office of Naval Research Contract N00014-76-C-0757, NR 307-374.

7. **References**


COMPARISON OF SEA ICE FEATURES IN THE BEAUFORT AND BERING SEAS USING SLAR AND LANDSAT DATA

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ABSTRACT

As off-shore activities in the Arctic increase, remote sensing will become increasingly important for operational monitoring of sea ice conditions. Radar systems, both Side-Looking Airborne Radar (SLAR) and Synthetic Aperture Radar (SAR), will be used extensively for this purpose because of their all-weather observing capabilities. With sea ice data being collected by these radar systems, as well as satellite visible, thermal infrared, and passive microwave sensors, a need exists for the user to understand and be able to interpret the various types of remote sensing data. Now, through efforts such as the NASA Sea Ice Radar Experiment (SIRE), data to enable comparative studies of different observing methods are being accumulated.

This paper describes an investigation to conduct comparative analyses of sea ice features using SLAR and Landsat imagery, as well as aerial photography. The SLAR data used in the study were collected as part of the SIRE project in March 1979 over portions of the southern Beaufort and eastern Bering Seas. Landsat scenes covering the same areas were also acquired for dates near to those of the SLAR. A number of sea ice features could be identified in both types of imagery, including zones of fast ice, shear zones, fracture zones, refrozen leads, first-year ice fields, and multi-year ice floes (including the T-3 ice island). Several of the sea ice features displayed in the SLAR data products were verified through analysis of correlative low level aerial photography. A final product of the investigation was an interpretive chart showing the comparative signatures of the ice features that can be identified in SLAR and Landsat imagery.
1. Introduction
Remote sensing is an indispensable part of sea ice data collection in the Arctic [1]. Now, as oil and gas exploration and production moves to off-shore areas, increased monitoring of sea ice will be needed. Although extensive data bases have been accumulated using high resolution data from Landsat and operational meteorological satellites [2], cloud cover, especially during the summer months, is a serious deterrent to the use of visible and infrared sensor systems. Microwave sensors have, therefore, great potential for collecting arctic data because of their capability for observing ice through clouds. In addition to passive microwave sensors, active radar systems, including both Side-Looking Airborne Radar (SLAR) and Synthetic Aperture Radar (SAR), will be used extensively for operational ice monitoring.

The use of SLAR data for identifying ice features was first reported in 1971 [3]. Subsequently, an all-weather ice information system for the Great Lakes, for which SLAR was the primary observing system, was developed at NASA Lewis Research Center (LeRC) as part of the Great Lakes Winter Navigation Program [4]. The Great Lakes ice information system was extended to the Arctic and demonstrated in August and September 1976 under a cooperative NASA, NOAA, Coast Guard, and Navy program [5]. Results from the 1976 arctic ice information demonstration indicated the capability of this system to provide timely ice information on a routine basis. The use of SLAR is also an important part of the Canadian ice reconnaissance program [6].

As part of the continuing program to develop improved remote sensing systems, the Sea Ice Radar Experiment (SIRE) was carried out by NASA during March 1979. Included in the experiment plan was the collection of SLAR data over portions of the southern Beaufort Sea and the Bering Sea. The purpose of the investigation described in this paper [7], which was conducted in association with the Sea Ice Radar Experiment, was to compare the ice features mapped from the SLAR imagery with features mapped from correlative Landsat imagery and from aerial photography to determine what the relative scales are and to determine what ice features not detectable in Landsat may be detectable in the SLAR imagery. A product of the study was the development of an ice analysis scheme for SLAR data similar to the analysis methods developed earlier for Landsat imagery.

2. SLAR and Landsat Data Sample
Three geographic areas were designated for the 1979 experiment: (1) the southern Beaufort Sea off the Mackenzie Delta; (2) the Beaufort Sea off the northern Alaska coast from Prudhoe Bay to Barrow; and (3) the Bering Sea, including the Norton Sound-Bering Strait area and southward to the ice edge. During the March experiment period, the NASA Lewis Research Center C-131 aircraft flew SLAR missions over all three test sites.
The SLAR flown on the C-131 was an AN/APS-94D system with a frequency in the X-Band (9.10 - 9.40 GHz). This system has H-H polarization, 30 meter range resolution, and a mapping swath of 25, 50 or 100 km. The data can be output in real-time as a preliminary film product. However, since the photographic film used to record the backscattered signal returns has only a limited dynamic range and cannot portray the full dynamic range of signals processed by the radar receiver, the final products were processed from magnetic tape on which the full dynamic range of the receiver output signal was recorded.

The SLAR images used in the analysis were at a scale of 1:250,000 and were processed in the same manner as were earlier SIAR data discussed in reference [5]. Thus, the shades of gray correspond to the intensity of backscattered microwave radiation from within the antenna's field of view. Light toned, white areas indicate high intensity backscattered radiation as might be expected from ice features exhibiting a high degree of surface roughness, such as rafted and ridged areas as well as from features with multiple edges such as brash ice. Dark toned, black areas represent areas of minimal backscattered radiation such as open water or smooth surface ice sheets from which the incident microwave pulse is specularly reflected away from the receiving antenna.

The Landsat data were acquired in two spectral bands, the MSS-5 (visible) and MSS-7 (near-infrared). The initial identification of ice features was made using these 1:1 million scale images. Subsequently, enlarged prints at a 1:250,000 scale were produced and also used in the data analysis. Because some ice features are more evident in the visible band, and others display greater detail in the near-infrared band, both the MSS-5 and MSS-7 were analyzed for comparison with the SLAR. The SLAR data set and corresponding Landsat scenes used in the comparative analysis are listed in Table 1.

<table>
<thead>
<tr>
<th>Test Site</th>
<th>SLAR DATA Date</th>
<th>LANDSAT DATA Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beaufort Sea: North of Mackenzie Delta</td>
<td>12 March</td>
<td>11 March</td>
</tr>
<tr>
<td>Beaufort Sea: Prudhoe Bay to Barrow</td>
<td>16 March</td>
<td>8, 12, 13, 21 &amp; 22 March</td>
</tr>
<tr>
<td>Beaufort Sea: Prudhoe Bay area</td>
<td>20 March</td>
<td>12, 13, 21 &amp; 22 March</td>
</tr>
<tr>
<td>Bering Sea: Norton Sound</td>
<td>26 March</td>
<td>24 March</td>
</tr>
<tr>
<td>Bering Sea: Norton Sound</td>
<td>27 March</td>
<td>24 March</td>
</tr>
</tbody>
</table>
3. SLAR-Landsat Comparative Analysis

3.1 Beaufort Sea, Harrison Bay Region (20 March)

A SLAR image over a portion of the southern Beaufort Sea on 20 March, shown in Figure 1, displays the variety of ice conditions just north of Harrison Bay near 71°N latitude and 151°W longitude. A number of the features displayed in this SLAR image were also clearly visible in a correlative Landsat-3 image of 21 March. An enlargement of that portion of the Landsat scene (MSS-7) viewing these features is shown in Figure 2.

The distinctly darker tone in the SLAR, and high reflectance in the Landsat, of the ice features labeled 1, 2 and 3 suggests older, thicker ice floes, comprised of either thick first-year ice or possibly multi-year ice. The overall backscatter of these floes is generally low in the SLAR, indicating mostly level (smooth) ice. The bright linear features observed crossing floes 1 and 3 are narrow ridges; numerous ridges and hummocks are also observed across floe 2. The higher return in the SLAR and lower reflectance in the Landsat of the sea ice surrounding these floes, indicates that they are embedded in hummocked and ridged zones of younger ice.

The features labeled 4 and 5, which appear relatively narrow and with a high reflectance in the SLAR data, display a considerably lower reflectance in the Landsat image. These features are broader portions of the outer shear zone boundary. The lower reflectance in the Landsat is indicative of generally young ice in the refrozen flaw lead, while the high backscatter in the SLAR image indicates that considerable ridging has occurred.

Features 6 and 7, which display a low backscatter in the SLAR and very high reflectance in the Landsat, are narrow zones of level ice located adjacent to another shear zone boundary. Feature 8 is similar to features 4 and 5, and indicates a broad zone of ridging in a refrozen lead comprised of young ice.

3.2 Beaufort Sea, Mackenzie Delta Region (12 March)

On 12 March 1979, SLAR data were acquired from the Mackenzie Delta region northward to approximately 73°N in the eastern Beaufort Sea. Correlative Landsat-3 coverage for this region was available for the preceding day, providing an excellent opportunity to compare ice features in the two types of data. The SLAR image, shown in Figure 3, views the region near 71°N and 132°W; the portion of an enlarged Landsat image viewing this same region is shown in Figure 4.

A number of sea ice features detectable in both images have been identified. Features 1, 2 and 3 are older, thicker ice floes comprised of generally level ice with numerous ridges present. These floes are typical of the overall ice conditions which exist in the surrounding area. The overall low backscatter in the SLAR is indicative of level
ice, whereas the bright linear features indicate considerable ridging. This SLAR signature along with the high reflectance in the Landsat suggest that the floes are likely thick first-year or second-year ice.

The features labeled 4 and 6 are refrozen leads comprised of young ice (grey and grey-white) as indicated by the low reflectance in the Landsat image; however, the high backscatter within these refrozen leads in the SLAR images indicates that numerous ridges are present. Feature 5, which displays a low reflectance in the Landsat image and high backscatter in the SLAR, is an example of a fracture. Some finger rafting is indicated to have occurred along this fracture in the SLAR image.

4. Ice Analysis Scheme for SLAR Data
The method of interpreting and analyzing sea ice features that can be reliably determined from SLAR data (at small depression angles) is based almost entirely on the amount of observed backscatter. The sizes and shapes of the features must, of course, also be considered. As previously stated, the highest backscatter (brighter tones) observed in the SLAR imagery is associated with features such as ridges, rubble ice or hummock fields, and rafting; the lowest backscatter (darker tones) is related to features such as smooth or level ice, multi-year floes, ice islands, and open water or ice-free conditions. Examples of some of these features have been identified in the figures discussed in the previous section.

An interpretive chart showing the ice features that can be determined from SLAR is presented in Table 2. For comparative purposes, the corresponding Landsat signature is also given in the chart. This interpretive chart, which is based on the analysis of the data sample available for this investigation, shows that SLAR imagery has distinct advantages over Landsat for identifying certain ice features. In particular, ice features resulting from pressure processes, such as hummocks, ridges, and even finger rafting, are readily apparent in the SLAR imagery because of their high backscatter. For the most part, these types of features are not discernable in Landsat imagery, usually being below the sensor resolution. In several instances, ridges and rubble piles seen in correlative aerial photographs can be identified in the SLAR imagery, but were completely masked out in Landsat. It should also be noted, however, that when several ridges or hummocked areas exist across a zone of generally level ice, as seen distinctly in aerial photography, the entire area may become saturated in the SLAR.

The identification of certain ice types is also possible in SLAR, but not in Landsat. For example, because of weathering processes, which result in the smoothing of ridges and hummocks on the ice surface, and changes in the internal characteristics of the ice, multi-year floes appear with a considerably lower return in SLAR imagery, and
TABLE 2
INTERPRETIVE CHART OF ICE FEATURES IN SLAR IMAGERY

<table>
<thead>
<tr>
<th>Ice Feature</th>
<th>SLAR Signature</th>
<th>Landsat Signature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Older Ridges</td>
<td>bright linear features across floe</td>
<td>not observed</td>
</tr>
<tr>
<td>Young Ridges</td>
<td>high backscatter linear features within refrozen lead</td>
<td>not observed; variations in thickness of young ice are displayed</td>
</tr>
<tr>
<td>Hummock Fields</td>
<td>high backscatter within Area A, and bright spot in center of large dark area</td>
<td>not observed; hummocks and ridges not detectable in Landsat</td>
</tr>
<tr>
<td>Ice Island (T-3)</td>
<td>low backscatter (dark)</td>
<td>not observed</td>
</tr>
<tr>
<td>Multi-year Floes</td>
<td>area of low backscatter with some isolated high backscatter</td>
<td>not observed; uniform high reflectance across floe</td>
</tr>
<tr>
<td>Probable Second Year Floes</td>
<td>greyer tone or medium backscatter</td>
<td>not observed; uniform high reflectance across floes</td>
</tr>
<tr>
<td>Probable First Year Floes</td>
<td>generally low backscatter (dark) with numerous bright linear features (ridges)</td>
<td>not observed; uniform high reflectance across floes</td>
</tr>
<tr>
<td>Level Ice</td>
<td>overall low backscatter (dark) with some isolated high backscatter</td>
<td>low reflectance, grey tones or very dark depending on ice age (thickness)</td>
</tr>
<tr>
<td>Refrozen Lead (older ice)</td>
<td>overall low backscatter (dark) with numerous bright linear features (ridges) present</td>
<td>overall greyer reflectance, but considerably brighter than young ice in new leads</td>
</tr>
<tr>
<td>Refrozen Lead (young ice)</td>
<td>low backscatter (dark) bright linear features (ridges) may or may not be present</td>
<td>lower reflectance of young ice very distinct</td>
</tr>
<tr>
<td>Refrozen Polynya</td>
<td>low backscatter (dark)</td>
<td>lower reflectance associated with young ice (grey-white)</td>
</tr>
<tr>
<td>Fracture</td>
<td>high backscatter linear feature</td>
<td>lower reflectance associated with grey or grey-white ice</td>
</tr>
<tr>
<td>Level Fast Ice</td>
<td>low backscatter (dark)</td>
<td>usually very bright due to heavy snow cover</td>
</tr>
<tr>
<td>Shear Zone</td>
<td>high backscatter, broad linear feature due to excessive ridging</td>
<td>low reflectance usually associated with young ice; ridging not detectable</td>
</tr>
<tr>
<td>Refrozen Flaw Lead</td>
<td>combination low and high backscatter associated with young level ice (dark) and ridging and hummocking (bright) and embedded older floes</td>
<td>an overall lower reflectance associated with young ice (grey-white), embedded older floes very bright</td>
</tr>
<tr>
<td>Open Water or Ice Free</td>
<td>low backscatter (dark)</td>
<td>lowest reflectance (black)</td>
</tr>
<tr>
<td>Fast Ice</td>
<td>generally low backscatter (dark) with numerous bright linear features (ridges)</td>
<td>mostly high reflectance except for some grey tone possibly caused by meltwater</td>
</tr>
</tbody>
</table>
thus can be readily distinguished from first-year ice. These older ice forms can only be inferred in the Landsat imagery, as they are usually associated with increased brightness levels within the pack; the higher reflectance in Landsat data can, however, also be associated with increased amounts of snow cover on younger, level ice.

The narrow shear zone boundaries that separate the pack ice from fast ice, as well as boundaries between ice of varying motions within the pack itself, display distinctly lower reflectances in the Landsat data. However, these same boundaries display near maximum brightness levels in the SLAR imagery indicating the presence of severe ridging and/or rubble ice. Other linear features observed in the Landsat imagery include fractures or cracks. In SLAR imagery, these same features usually appear very bright, due in part to backscatter from the sides of the ice floes along the fracture; the high return also suggests that the cracks or fractures may occur along, and indeed follow, ridges.

Zones of level, smooth ice may appear either very bright or with lower reflectances in the Landsat imagery, usually depending upon the amount of snow cover; newly refrozen leads, therefore, have a low reflectance, whereas fast ice, which has built up an accumulation of snow, has a high reflectance. In the SLAR imagery, on the other hand, level (smooth) ice is always displayed as a very dark feature.

Although the SLAR is considered to have a number of specific advantages for ice analysis as compared to the Landsat data, there are also some disadvantages. One of the major disadvantages is the rather limited swath width of the SLAR, compared to the much broader (185 km swath) areal extent of each Landsat image. Also, unless geographical reference points are visible somewhere in the SLAR tracks, it is extremely difficult to obtain precise locations (latitude-longitude coordinates) of specific ice features unless land reference points also occur in the images.

Because of the low backscatter in the SLAR data associated with ice islands, multiyear ice floes, level ice, and even open water or ice-free conditions, these features often appear in similar gray tones. It is, therefore, sometimes impossible to determine from the SLAR data alone, whether you are observing level ice, a multi-year floe, or open water; it may also be difficult to determine if new leads are ice-free or covered with young, level ice. The results of this study have shown that in these instances, a more reliable assessment of the ice type can be made through interpretation of both SLAR and Landsat.

5. Conclusions
The investigation to carry out a comparative analysis of SLAR and Landsat data has shown that SLAR imagery has distinct advantages over Landsat for identifying certain
ice features and ice types, but that in certain instances, a more reliable assessment of the ice type can be made through interpretation of both SLAR and Landsat data. Because both SLAR and Landsat, as well as other remote sensing systems, will be used extensively to monitor sea ice in coming years, a need exists for an interpretive handbook for remote sensing of sea ice.

As a result of the SIRE and other programs, an adequate data set has been accumulated to make an interpretive handbook possible. Several types of aircraft and satellite observations of the same or similar ice conditions can now be presented, including aerial photography, aircraft SLAR and SAR, Landsat visible and near-infrared, meteorological satellite visible and thermal infrared, Seasat SAR, and satellite passive microwave. It would be possible, therefore, to compile a handbook to include at least the following: discussion of the characteristics of each type of sensor system; the types and scales of ice features that can be identified in each type of data; examples of the same or similar ice features as seen by each sensor system; keys to data interpretation; and discussion of the advantages and limitations of each remote sensing system. An ice interpretive handbook would be a valuable document for all potential users of remotely sensed ice data and would be particularly useful to the arctic petroleum industry, which is by far the most important current and future non-government user of sea ice data.

Acknowledgment

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Comparative Analysis of Sea Ice Features Using Side-Looking Airborne Radar (SLAR) and Landsat Imagery  
Figure 1  SLAR image on 20 March 1979 viewing sea ice in southern Beaufort Sea near 71°N and 151°W.
Figure 2  Enlargement of portion of Landsat-3, MSS-7 image (I.D. 30381-21172) of 21 March 1979 viewing same area as covered by SLAR image shown in Figure 1.
Figure 3  Mosaic of SLAR images of 12 March 1979. Area shown is north of Mackenzie Delta region near 71°N and 132°W in the eastern Beaufort Sea.

Figure 4  Enlargement of portion of Landsat-3, MSS-7 image (I.D. 30371-20195) of 11 March 1979 viewing same area as covered by SLAR image shown in Figure 3.
COMPARISON OF PSEUDO-PARALLAX EFFECT AND CROSS-CORRELATION FOR THE COMPUTATION OF ICE SURFACE VELOCITIES IN NORTHERN WATERS

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Abstract

Velocity measurement in northern waters is difficult at any time but during ice break-up flow measurements are almost impossible. MacKay et al. (1974) and Sherstone (1980) showed that stream flow in a river carrying floating ice can be calculated by a method of pseudo-parallax. Overlapping aerial photographs taken at fixed time interval are set up in a stereoplotter. The apparent height of the floating ice above a datum level gives a measure of the surface velocity. The work reported here is based on the pseudo-parallax method but uses cross-correlation instead of photogrammetry to compute and map the surface velocity. Pairs of aerial photographs representing extremes of flow regime during the break-up period were digitized and registered. The velocity profile across the river was then obtained by selecting the shift of the data that maximises the cross-correlation function. Details are given concerning the computational procedure, and optimal choice of parameters. Although the results are compared with stream flow measurements on the Liard River, N.W.T. this technique has potential application to ice and current velocity determination in large channels and estuaries.
Introduction

With man's increasing use of ice-infested northern waters the need to understand and measure ice movement has increased. Many researchers must deal with problems of large ice masses and the broken ice present in near-shore estuaries and harbours are of major concern. There are thus demands to quantify such variables as size, spacing, and velocity of moving ice blocks within these areas.

Where ice or surface water velocities are desired, individual, ground-based measurements can be undertaken where ice densities are low and favorable observational conditions exist. With higher ice concentrations or when large areas must be studied a remote sensing technique for velocity determination is necessary.

Previous Work

Work performed by the National Hydrology Research Institute (N.H.R.I.) of Environment Canada on northern rivers during spring break-up indicated that measurements of surface velocities were desirable to understand many ice related phenomena (MacKay et al. [8]). Using aerial photography, river ice movement, and aircraft displacement between consecutive exposures, velocity can be calculated for individual ice blocks from measurement of the apparent stereo-parallax effect visible in the water (Manual of Photogrammetry, [12], pp. 1109-1111; Cameron [1]). The apparent three-dimensional topography observed in the water is such that those areas exhibiting the greatest vertical displacement relative to the shoreline are zones of maximum velocity. Vertical displacement values can be converted to downstream velocities through a similar triangle solution. Further examination of the conventional photogrammetric approach for large channels has been performed by Cameron [1], [2], and, for estuarine environments, by Duhaut [3], and Keller [5] and [6]. Such velocity determination methods are also supported by Faig [4] and Moffitt [10].

Duhaut [3] used bouys, plywood targets and manufactured foam to determine current velocities in the Thames Estuary. Over several study years he found average velocities derived from a photogrammetric approach were within 0.02 M. sec.\(^{-1}\) of those obtained by current metering. Sherstone [11] found that rough ground based measurement of ice debris velocities in a large channel varied only 4-6% from airborne values. Velocities obtained in this manner for test sites on
the Liard River, N.W.T. were used to calibrate the auto-correlation technique described below.

**Test Site**

The test site was a reach of the lower Liard River, N.W.T. located near the confluence with the Mackenzie and near the settlement of Fort Simpson (Figure 1). This site was selected by the N.H.R.I. for intensive ice jam and ice break-up research in 1976. Consequently there exists a large volume of aerial photographs suitable for velocity measurement.

Photography was selected to represent widely varied ice transport conditions. Photos A24639, Nos. 130-131, May 5, 1977, represent finely broken, concentrated ice debris of reduced contrast due to embedded sediment. Photos A25112, Nos. 119-119, May 11, 1979, represent high contrast, large ice size, with low concentration conditions. In both cases original photography utilized black and white film; the 1977 photography at a scale of 1:15,000 and the 1979 photography at 1:14,000 (Figure 2).

Using conventional photogrammetry the water surface was contoured and from this contoured "map" of apparent vertical displacements the downstream velocities were calculated. At any location on the contoured map a downstream velocity transect can be calculated and used for comparison with values derived from the auto-correlation approach.

**Computational Procedure**

The computational procedure may be divided into three parts. First, a stereographic pair of aerial photographs is placed under a optical digitizer and the photos are digitized by a mini-computer with the output written onto magnetic tape. The computer digitizes the data using a grey scale with 256 levels. The digitized images of the photograph are then input into another manually-controlled computer system. An operator matches two or more fixed land points at water level on the digitized images, and then one of the two data arrays, the source image, is rotated, and stretched until it matches the other data array, the reference image, on all land points. The two new arrays are then written onto magnetic tape in J.S.C. (Johnson Space Centre) format. These two operations were performed by the staff of the Canada Centre for Remote Sensing (C.C.R.S.).
The data on the J.S.C. tapes are then unpacked and rewritten on N.O.S. (Network Operating System) standard format on disc for use on the C.D.C. Cyber 70 computer system of the Department of Energy Mines and Resources. All further data analysis is performed on the Cyber system.

Since the data arrays are too large to be stored in core, the matrices are partitioned into submatrices each of equal size. Within each submatrix, the center pixel is used to represent a submatrix. An option in the procedure allows one to average all values within each submatrix instead. The large arrays are thus transformed into smaller matrices of more manageable size.

Surface river flow is assumed to be laminar and parallel to columns of the matrices. Therefore, by removing a column vector, \( \mathbf{a} \), from the matrix representing the source image, and shifting it with respect to the same column vector, \( \mathbf{b} \), of the reference image, there should be a shift value, \( \mathbf{j} \), where the vectors match in value. Of course, vectors should only be chosen which lie entirely over water. The value, \( \mathbf{j} \), will correspond to the number of submatrices that vertically displace the two images over water. The flow velocity may then be calculated from the submatrix column size, the scale (metres per pixel), and the time between the two photographs.

Five metrics, \( f_i(\mathbf{a}, \mathbf{b}, \mathbf{e}) \), \( i = 1, \ldots, 5 \), are employed to attempt a match between the two vectors and the value \( \mathbf{j} \), which minimizes \( f_i(\mathbf{a}, \mathbf{b}, \mathbf{e}) \) over all shifts \( \mathbf{e} \), is assumed to be the optimal matching shift.

The five functions, \( f_i(\mathbf{a}, \mathbf{b}, \mathbf{e}) \), are defined by:

1. **Cross-correlation function**

\[
f_1(\mathbf{a}, \mathbf{b}, \mathbf{e}) = \frac{-\frac{1}{n} \sum_{k=1}^{n} \left( a_k + e \cdot j - \frac{1}{n} \sum_{j=1}^{n} a_j + e \right) \left( b_k - \frac{1}{n} \sum_{j=1}^{n} b_j \right)}{\left(\frac{1}{n} \sum_{k=1}^{n} \left( a_k + e \cdot j - \frac{1}{n} \sum_{j=1}^{n} a_j + e \right)^2 \right)^{1/2} \left(\frac{1}{n} \sum_{k=1}^{n} \left( b_k - \frac{1}{n} \sum_{j=1}^{n} b_j \right)^2 \right)^{1/2}}
\]

2. **Cross-covariance function**

\[
f_2(\mathbf{a}, \mathbf{b}, \mathbf{e}) = \frac{-\frac{1}{n} \sum_{k=1}^{n} \left( a_k + e \cdot j - \frac{1}{n} \sum_{j=1}^{n} a_j + e \right) \left( b_k - \frac{1}{n} \sum_{j=1}^{n} b_j \right)}{(n-1)}
\]
(3) Euclidean metric

\[ f_3(a, b, e) = \left( \sum_{k=1}^{n} (a_k + e - b_k)^2 \right)^{1/2} \]

(4) Taxi-cab metric

\[ f_4(a, b, e) = \sum_{k=1}^{n} |a_k + e - b_k| \]

(5) Maximum metric

\[ f_5(a, b, e) = \max_{k=1, \ldots, n} |a_k + e - b_k| \]

where \( n \) is the lengths of the vector \( b \).

The matching shift is then refined by quadratic interpolation with the two neighbouring function values, and finally ice floe velocity is determined from the refined shift value.

**DISCUSSION**

A comparison of the photogrammetric and cross-correlation methods was performed for the images corresponding to photographs A24609-130 and A25112-191 (Fig. 2). The results for a cross-section of the river just south of the ferry crossing points in the photos are given in Figure 3. These results show a relatively high scatter for the mid-channel velocities in the image A25112-191, where the ice masses are more variable in size, and lower scatter for the velocities in image A24639-130. There are several reasons for this difference. First, large ice masses can be grounded and hence cause anomalies in the cross-correlated vectors. Also, rotation of large assymetric ice bodies can affect the cross-correlation coefficients. Finally, since the vectors have either very high or very low gray scale levels, the correlation values will be affected by the number of ice-water boundaries in the vectors. For example, a very large ice mass could create an entire vector of almost identical values, and hence no variation in the magnitude of the cross-correlation coefficient would occur when the two vectors
are shifted relative to each other. The high differences in the results near the
dge of the river are due to the presence of shore-fast ice.

The cross-correlation coefficient is shown rather than the other
statistics because it exhibits results closest to the ground-based measurements.
However, when all the five measures of correlation give similar results we can
generally feel more confident in the accuracy of the cross-correlations statistic.

Various other options were also tested. A jackknife statistic [9] was
used to attempt to estimate sampling error in the computed velocity and
cross-correlation coefficients. Unfortunately scatter was too high in this
statistic to enable reasonable conclusions to be drawn about the error in
velocity. Also, increasing the density of vectors along a cross-section did not
seem to appreciably change the results.

Another area of potential work could be to vary the pixel size since there
could be an optimal pixel size depending on the average size and density of the ice
bodies.

The cross-correlation technique has several drawbacks. The data is
processed vertically down an image so laminar flow along an axis of the references
image is required. Also the cross-correlation method requires the ice blocks not
to be too evenly spaced and of too uniform a size to preclude a proper recognition
of the pattern. However, the main sources of error seem to be the presence of
shore-fast ice and very large floating ice bodies.

In spite of these shortcomings it is evident from Figure 3 that the
mid-channels velocities computed using cross-correlation analysis agree to within 3
to 5%, for image A24639-130, with ground-based measurements.

As Langham [7] has pointed out, this method could prove useful for
analyzing ice flow velocities on a different scale using Landsat imagery. Also,
although the analysis was performed only for river channel flow, it could prove
useful for analyzing ice flow in estuaries as well, provided that there exist fixed
reference points in the images.

Finally, we believe that the cross-correlation techniques when applied to
the analysis of large amounts of data, could prove to be more efficient than the
classical photogrammetric approach in the determination of ice flow velocities.
Acknowledgements

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References


Figure 1

LOCATION OF LIARD RIVER
STREAM VELOCITY MEASUREMENT TEST SITE
WITHIN THE NORTHWEST TERRITORIES,
CANADA
Figure 2. Portions of photographs A24639-130 (top) and A25112-191 (bottom) illustrating variation in ice types and concentrations at break-up.
Figure 3. Cross section velocity profiles of the Liard River corresponding to photographs A25112-191 (top) and A24639-130 (bottom). Cross-sections are located just below ferry crossing site.
In this paper we are reviewing recent work in ice mechanics and discussing it in order to get a broader view of the properties of ice that could be simplified for engineering applications. Two topics are of very immediate interest and are treated; the rheological representation of the material under uniaxial loads and the failure criterion under complex loading conditions and more specifically, for floating ice, under conditions of plain stresses. Although ice a complex material, it is shown that it can be simply represented mechanically, in the first approximation, with an elasto-plastic model failing under maximum shear stresses.

I - INTRODUCTION

There has been a very large amount of work done on the mechanical properties of ice, in the last ten years, which has led at this time to a Babel tower of conflicting theories. Because of sometimes, completely different views on the basic behavior of the material, the results, in the applications, are often quite different using one mode of computation or another. In this paper we want to review the more recent research on the mechanical properties of ice with the objective of giving simple guidelines for engineers to represent, in the first approximation only, the mechanical behavior of ice as a material.

II - RHEOLOGICAL REPRESENTATION OF ICE BEHAVIOR

A) Experimental results under uniaxial loading conditions

The deformation behavior of a polycrystalline ice sample under a constant uniaxial load in tension and compression is shown on Fig. 1, the plain lines being in compression and the dashed lines in tension, for identical loads (Michel, 1978). After instantaneous elastic deformation, we see the classical stages of primary creep,
secondary and accelerated creep. Both extreme limits of creep correspond to completely different behaviors. For very low loads or glacier flow, the transitory part of the curve is important and secondary creep occurs after recrystallization of the ice. For high loads applied quickly, the ice fails in a brittle manner like a perfect elastic material.

The behavior of ice under a constant load is not usually representative of engineering problems. In most cases, like that of a creeping glacier, a moving ice floe or a moving vehicle over ice, the most representative condition is that of an ice sample deformed at a constant speed. This condition is represented in Fig. 2, both for tension and compression. Here again we distinguish the stages of primary creep and tertiary creep, but for most engineering applications with rather high loading rates, there is a maximum value of the stress during deformation corresponding to the yield point followed, in many cases, by a stage of
secondary creep at a lower stress. It is noteworthy to observe that in tension ice is a very brittle material that does not sustain microcracking or a typical yield point, as it does under compressive stresses. It fails brittlely under a constant load value, for a large range of loading rates.

B) Recent rheological models for the creep of ice

1 - The standard approach

The linear rheological models of classical rheology with combinations of Maxwell and Kelvin-Voigt elements have been useful in the past to explain certain mechanisms in the deformation of ice but they have been unable, even approximately, to represent the measured creep curves.

Non-linear models have thus been in use to give a better representation to represent the deformation behavior of ice. Let us represent the total accumulated strain $\varepsilon$, of an ice sample loaded axially by:

$$\varepsilon = \varepsilon_c + \varepsilon_p + \dot{\varepsilon}_s \cdot t$$

where $\varepsilon_c$ is the instantaneous strain on loading, $\varepsilon_p$ is the transient creep, $\dot{\varepsilon}_s$ the so-called secondary creep rate at constant speed, and $t$ is the time.

Royen (1922) was the first to use one form of Andrade's empirical relation (1910) to represent the transient creep of ice:

$$\varepsilon_p = B \cdot t^{1/3}$$

where $B$ is a constant. This representation has been found valid for low creep rates of glacier ice by Glen (1955) and Duval (1976).

An expression for the secondary creep rate which has received very extensive confirmation was first proposed for ice by Glen (1955):

$$\dot{\varepsilon}_s = A \cdot \sigma^n \exp \left(-\frac{Q}{R\theta^*}\right)$$

where the point on $\dot{\varepsilon}_s$ is the derivative in respect to time, $A$ a constant, $\sigma$ the uniaxial stress, $n$ the Glen's exponent, $Q$ the activation energy for the creep of ice, $R$ the universal gas constant and $\theta^*$ the absolute temperature in °K.

This law of creep is compatible with the general theory of dislocations movement. Usually the exponent $n$ is close to three and this is considered a classical mode of dislocation climb (Friedel, 1956). The diffusion process is temperature sensitive and this is represented by the Arrhenius rate function, which has been verified for ice.
A variation of the standard method was introduced by Jellinek and Brill (1956) in a form widely used for other materials:

\[ \varepsilon = \frac{\sigma}{E} + Ct^m \sigma^n \]  

[4]

where \( E \) is the Young's modulus, \( C \) a constant and \( m \) another constant.

In this last equation the Andrade type creep is combined with the stress function. For polycrystalline ice the Jellinek and Brill (1956) found best fit values for \( m \) between 0,47 and 0,53. Gold (1965) also finds values between 0,48 and 0,88 for polycrystalline ice in the range of loading between 0,4 and 1,4 MPa. More recently Ladanyi (1979) using Eq. [3] for borehole relaxation tests found a value \( m = 0,31 \), for polycrystalline fresh water ice. This is closer to Andrade's original value.

Many criticisms can be made against the standard representation. The creep function is empirical and it increases indefinitely with time. This can hardly be used to predict the onset of secondary creep (Ting and Martin, 1979).

But the two main faults of the standard method is the neither Eq. [2] nor [4] does predict the important effect of retarded elasticity, which is extremely important in the behavior of ice, nor does it predict the yield strength of ice and the corresponding strain, because the equation under constant strain rate, never shows yield, even for an infinite time.

To introduce the effect of delayed elasticity, Duval (1978) has proposed to introduce separately in Eq. [1]:

\[ \varepsilon = \varepsilon_i + \varepsilon_a \]  

[5]

where \( \varepsilon_i \) is the instantaneous elastic deformation and \( \varepsilon_a \) the inelastic one. He proposes then to use the standard equations [1] and [2] for the loading part of the creep curve:

\[ \varepsilon = \varepsilon_c + Bt^{1/3} + \varepsilon_s t \]  

[6]

but for the creep recovery curve at unloading he proposes another empirical function:

\[ \varepsilon = \varepsilon_{to} - \varepsilon_{\varepsilon} - K \log [1 + \alpha (t - to)] \]  

[7]

where \( \varepsilon_{to} \) is the total strain at time of unloading \( to \). \( K \) and \( \alpha \) being constants.

The last term of the equation then represents the anelastic strain that is recoverable in function of time. This equation fits very well the data and explains the mechanisms of creep. The importance of the anelastic effect can be realized as his tests show that the corresponding anelastic modulus is one order of magnitude less than the dynamic elastic modulus.
Duval's recent analysis throws much light on the anelastic behavior of ice, but it would be very inconvenient to use in applications as the creep laws do not have the same formulation for loading and unloading conditions.

2 - Sinha's model

Sinha (1978) has proposed recently a rheological model for ice that follow the general form of the standard theory Eq. [1], but where the transient Andrade type creep is replaced by a new empirical relation he had used previously for plate glass:

\[ \varepsilon_p = c \left( \frac{\sigma}{E} \right)^s \left( 1 - \exp \left\{ -(at)^b \right\} \right) \]  \hspace{1cm} [8]

where \( b, c, \) and \( s \) are constants and \( a \) is given by:

\[ \ln a = \frac{Q}{kT} + d \]  \hspace{1cm} [9]

\( d \) being another constant.

With [9] the transient creep has also the same activation energy as viscous permanent creep, which Sinha also expressed by Eq. [3]. He calls a material when both transitory and permanent creep depends on the same activation energy, a simple thermorheological material, which ice is. Contrary to Andrade creep, this function of transient creep is entirely recoverable with a reduction of \( \sigma \) to zero, and thus it is the complete portion of delayed elasticity in the deformation process. Furthermore it tends toward a constant with increasing time, representing then, permanent secondary creep at infinity.

Fig. 3 - Adjustment with Sinha's rheological model of Camp and Brill's experimental data for snow ice under a constant load in tension and after load removal. The zero time for curve (c) has been shifted for clarity.
In a recent paper Sinha (1979) finds that the constants in the transient term are not constant but depend on grain diameter. He assumes that this term in the creep process represents grain boundary sliding mechanisms. Fig. 3 shows the adjustment made with the theory for Brill and Camp's (1961) experimental results.

Although Sinha's model is quite an improvement over the Andrade representation, it still fails to represent the behavior of ice under a constant deformation rate. For strain rates of engineering interest, it shows a yield point at infinity, instead of early in the deformation process.

3 - Our model

In 1968 we presented a physical model of creep of ice which is based essentially on the behavior of an agglomerate of ice crystals each of which glides by dislocation climb, but only in the direction of the basal plane of each crystal and which deforms in an elastic manner only, in the normal to the basal plane (Michel, 1968). Because of the random orientation of the c-axis in polycrystalline ice, some crystals badly oriented for glide in each deformation band, are overstressed and the larger elastic deformation they take, accounts for the delayed elasticity in the creep process.

The physical model is shown with a rheological representation in Fig. 4. It is made up of a serie of a spring representing the instantaneous elastic deformation of the sample plus a network of parallel elements each representing a single crystal and combining the spring of the elastic deformation in the flow direction plus the non-linear dashpot whose relative importance is also a function of the orientation of the basal plane of the crystal in the deformation band.

![Fig. 4 - Sketch of rheological model of creep of ice under a tensile stress σ.](image-url)
Although the whole model is developed for shear stresses which control the dislocations glide processes, the model could be written for a constant normal stress in the following form:

\[ \dot{\varepsilon} = A \left [ 1 + \alpha \left ( \frac{\varepsilon}{\varepsilon_r} \right )^m \right ] \left [ 1 + \beta \left ( 1 - \frac{\varepsilon}{\varepsilon_r} \right )^{n} \right ] \left ( \frac{\sigma}{\sigma_0} \right )^n \exp \left ( - \frac{Q}{R T} \right ) \]  

[10]

Each term in this equation has a physical meaning that comes from the model. The instantaneous elastic deformation does not appear and the function combines both delayed elastic effect and viscous flow.

The parameters all have a physical meaning:

- \( n, Q \) and \( R \) - as previously;
- \( A \) - constant related to the initial number of mobile dislocations in the ice;
- \( \alpha \) and \( m \) - coefficient and power of multiplication of mobile dislocations;
- \( \beta \) - relative importance of the anelastic deformations;
- \( \varepsilon_r \) - is the maximum anelastic deformation (for \( \varepsilon \geq \varepsilon_r \), \( \varepsilon / \varepsilon_r \) keeps a value of one);
- \( \varepsilon_t \) - strain for which the number of mobile dislocations becomes a constant (for \( \varepsilon > \varepsilon_t \), the terms \( \varepsilon / \varepsilon_t \) keeps a value of one). Permanent creep is then given by:

\[ \dot{\varepsilon} = A \left [ 1 + \alpha \right ] \left ( \frac{\sigma}{\sigma_0} \right )^n \exp \left ( - \frac{Q}{R T} \right ) \]  

[11]

As for Sinha's model, this is also a thermorheological model. However the last term for viscous flow is not a constant corresponding to permanent creep, so the value of delayed elasticity can be adjusted with measured values.

The model can be used with convenient parameters to adjust quite precisely any creep curve under a constant load. It does represent also very well the strain recovery curves as for example the same Brill and Camp's curves used for the Sinha's model, as shown on Fig. 5. In this case for example the values of parameters for curve A are \( m = 0,5, \alpha = 20, n = 3 \) and for the B curve, \( m = 0,5, \alpha = 40 \) and \( n = 3 \).

The model is the only one that represents the behavior of ice under a constant strain rate. It does show a yield point and also a stage of permanent creep after yield at a lower stress, as actually observed in ice. An example is shown on Fig. 6 for snow ice under compression. The values of the parameters are here \( m = 0,5, \alpha = 4, n = 3, \dot{\varepsilon}_r + \dot{\varepsilon}_o = 1,8 \times 10^{-3} \text{ s}^{-1}, \dot{\varepsilon}_t = 5 \times 10^{-3} \text{ s}^{-1} \).
Fig. 5 - Adjustment with Michel's rheological model of Camp and Brill's experimental data for snow ice under a constant load in tension and after load removal.

Fig. 6 - Adjustment of creep curve for snow ice under a constant compressive strain rate with Michel's rheological model (measurements by Drouin and Michel 1971).
4 - Simplified model for engineers

In many engineering problems the condition of loading is that of constant speed at high rates for the design of structures. Under these conditions the creep curve is very steep at the origin, and the primary creep is not very significant. Ice can then be simulated by a perfect elastic-plastic material. The whole mechanical behavior of ice can then be simplified as shown on Fig. 7 for both simple tension and compression.

For very high loading rates, \( \dot{\varepsilon} > 10^{-1} \, s^{-1} \), the ice behaves like a perfectly elastic material and fails brittlely by crack propagation either in tension or compression. The elastic modulus \( E_0 \) is the dynamic modulus which gives the instantaneous elastic deformation. The fracture strengths \( \sigma_t^* \) and \( \sigma_c^* \) are given by (Michel, 1978):

\[
\sigma_t^* = 2 \sqrt{\frac{6}{\sigma_{cv}} \frac{G_{66}}{d^*} \left(1 - \frac{G_{66}}{E_{33}}\right)} \tag{[12]}
\]

\[
\sigma_c^* = 2 \left[ C(\theta) + \sqrt{\frac{6}{\sigma_{cv}} \frac{G_{66}}{d^*}} \right] \tag{[13]}
\]

where \( \sigma_{cv} \) is the surface energy of ice in contact with its saturated vapor, \( G_{66} \) is the elastic modulus due to a shear stress in a plane perpendicular from the c-axis, \( E_{33} \) is the Young's modulus in a direction along the c-axis, \( d^* \) is the equivalent diameter of the ice crystal and \( C \) is a cohesive value in compression, depending very much on temperature \( \theta \).
For strain rates between $10^{-1} \text{s}^{-1} < \varepsilon < 10^{-5} \text{s}^{-1}$ the ice will fail in a transition zone, sometimes brittle, sometimes as a perfect elasto-plastic body as shown on Fig. 7. The maximum yield strength of ice, in compression, will then be about $1.5 \sigma_c$ but in tension it will keep the same value $\sigma_f = \sigma_c$.

The Young's modulus will then have to take account of the inelastic effect and will decrease with decreasing rates. Typical values of the static modulus are given by Traetteberg et al. (1975) as shown on Fig. 8. These are, however, tangent modulus close to the origin and the overall behavior might be better approximated with secant modulus about 20 to 30% lower.

![Fig. 8 - Traetteberg et al's measurements of strain rate dependence on Young's modulus.](image)

III - A FAILURE CRITERION FOR FLOATING ICE

It has long be assumed for glacier ice at low stresses, with certain qualifications, that hydrostatic pressure had no effect on the yield strength and that the material was essentially failing under maximum shear stress. This implied a yield criterion similar to the Von Mises or Tresca criteria.

Very little work had been done on these questions for the higher stresses of engineering interest, and we will now review the more recent work on this question.
A) **Glacier ice**

The only published results at this time on the triaxial behavior of glacier ice at higher stresses are those of Jones (1978). These results are shown on Fig. 9 for fine grained ice about 1 mm in diameter.

At first sight it can be seen from the figure that this ice is not far from following a shear stress failure criterion as shown by dashed lines. As a second approximation, however, the strength is affected by hydrostatic pressures in two ways as described by Jones. At low but increasing hydrostatic pressures he observed that samples, under a constant strain rate, show less and less cracking activity with increasing pressure, so that the threshold of crack formation being increased, so is the strength.

Furthermore, starting from a pressure of about 30 MPa (300 atmospheres) the effect of pressure on the melting point should be accounted for. This has the effect of increasing the effective pressure in the ice and thus reducing the ice strength. For very high pressure this would ultimately reduces the strength to zero as suggested by Mellor (1979). Both of these effects could be accounted for in the expression for the cohesive strength of ice in Eq. [14]:

\[
C = c_0 (\theta - \theta_m)^{0.78} + c_1 p
\]  

[14]

where \(c_0\) and \(c_1\) are new constants depending on ice types, \(\theta_m\) is the depression of the melting point caused by the hydrostatic pressure \(p\).

**Fig. 9** - Results of triaxial testing by Jones (1978) for fine grained glacier ice at high stresses and hydrostatic pressures. A proposed extrapolation is made in the tensile zone and a constant shear failure envelope is traced in the compressive quadrant.
Another important remark on failure criterion is that ice is very brittle in the tensile quadrant and the first microcrack propagates immediately through the ice. The yield strength then depends very little on temperature, as given by Eq. [12], and differs essentially from the yield strength in compression by the cohesive value $C$. Thus there is an important discontinuity between the compressive and tensile state on the plane of maximum shear stress as indicated on Fig. 9. Reinicke and Remer (1979) have tried to approximate the yield criterion with a three parameters yield criterion. This approximation which gives a continuous curve fails to reproduce both the condition of discontinuity from compression to tension and the elongated failure curve in the high pressure area.

B) Floating ice

In sea ice, arctic and fresh water floating ice, there is more generally the specific conditions of the free floating surfaces which correspond only to biaxial stresses in the plane of the ice sheet. This plane stress condition can be presented more simply in two dimensions when the $\sigma_3$ value is zero.

Tests by Frederking and Gold (1975) helps very much to understand what is happening in $S_2$ ice with horizontal $c$-axis in the case of plane stress. Their results for two conditions of loading are shown on Fig. 10. In the type $B$ tests, the confining plates are perpendicular to the columns axes or to the basal plane of all crystals. In our basic model, the ductile behavior is caused essentially by plastic flow along the basal plane, so the confinement does not change the fundamental creep process and the yield strength is then the same as under uniaxial loading conditions. However, in type $A$ tests, the glide motion is restrained by the confinement in the basal plane and the ice has to fail across the columns in a complex process. This is much more difficult to do and lead to a much higher strength. Thus these results gives both the values of:

$$
\sigma_y^B = 2 \tau_{12y} \quad \text{(in the plane)}
$$

$$
\sigma_y^A = 2 \tau_{13y} = 2 \tau_{23y} \quad \text{(in the plane perpendicular to it)}
$$

On Fig. 11 we have represented the failure curve for $S_2$ ice under plane stress. In the compression quadrant the deviatoric stress is slightly pressure sensitive, but not much for a very limited variation in pressure. The failure curve then follows a law similar to the case of plane strain but only up to the condition of cross failure of the ice sheet to the top, for $\sigma_y^A$. This value can be seen to be between 2 to 5 times the $\sigma_y^B$ value depending on strain rates on Fig. 10.
Fig. 10 - Frederking and Gold's biaxial compressive tests results for type A and type B loading of columnar $S_2$ ice, at variable strain rates.

Fig. 11 - Proposed failure envelope for both columnar and granular ice under conditions of plane stresses.

On the same Fig. 11 we also show the failure envelope for isotropous granular ice. Because the cross failure strength is the same as the strength in the plane, $\tau_{12} = \tau_{13} = \tau_{23}$, the curve is symmetric around the origin. Not much pressure effect can be expected. We use here a Tresca type representation although a more continuous Von Mises type representation could also be used.
In the tensile quadrant we have shown the corresponding yield strength different than the compressive ones. Each failure curve represents only one condition of strain rate and temperature as all strengths depend very much on both.

These failure envelopes for the two types of ice explain why in the field many authors report failure of ice across the sheet to the tip and others failure in the plane of the sheet by plastic flow around an indentor (Fig. 12). It is thus much easier to fail granular ice under conditions of plane stress, than columnar $S_2$ ice.

![Different modes of failure of an ice sheet under plane stresses if the ice is granular or columnar $S_2$ ice.](image)

IV - CONCLUSION

It can be seen that although ice is a very complex rheological material, it can be represented for engineers, in the first approximation only and for high strain rates, by simple rheological models. It is either an elastic or elastic-plastic model and the long accepted practice of taking the maximum shear stress failure criterion can be continued if the values are qualified, and taken differently either for tension or compression or for non-isotropous ice.

V - REFERENCES


PLASTIC LIMIT ANALYSIS OF ICE SPLITTING FAILURE

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Summary

The splitting mode of ice failure is of interest in mechanical property tests and in some types of structure/floe interactions. This failure mode can be analyzed by the methods of plastic limit analysis using yield criteria derived from independent property tests. The data used in the present work corresponded to laboratory-grown freshwater ice with a columnar crystal structure.

An analysis of the Brazil test implies that the strength measured by this test should be relatively independent of the length of the cylindrical test specimen. This is a consequence of the small difference that exists between the plane stress (short specimen) and plane strain (long specimen) yield criteria in the tension/compression quadrant. The analysis also shows that the ductility of the ice has little effect on the strengths measured by this test. The strength that would be implied by an approximate elastic-brittle material model, in which fracture is assumed to occur when the stress state reaches the yield surface, is within the range of values calculated for a perfectly plastic material with the same yield surface. The Brazil strength can be interpreted as a value measured in a combined state of stress in which the compressive stress in one direction is three times as great as the tensile stress in the perpendicular direction. This strength should not be confused with the uniaxial tensile strength, which could be either greater or less than the Brazil strength depending on the ice temperature or other factors that would affect the shape of the yield surface in the tension/compression quadrant.

Useful information on the failure modes and forces for ice floe splitting and ice plate splitting in laboratory indentation tests is also provided by this approach. Failure mechanisms that involve both shear and tensile deformation have been investigated. The forces corresponding to these mechanisms are not substantially different for some floe sizes and ice properties. This suggests that one should expect to see a variety of failure modes when these events occur naturally. When the analysis is applied to laboratory indentation tests on unconfined ice plates, the results imply that the ice plates should be six to eight times as wide as the indenter if the initial failure load is not to be influenced by the size of the ice plate.

Introduction

Visual observations of ice failure events almost always include ice cracking and splitting as dominant or important characteristics; however, there are few publica-
tions available that provide solutions for such boundary value problems. If an isolated floe of sufficiently small diameter impacts a vertical structure, the floe may fail by splitting. The present analysis presents a solution technique that can be used for boundary value problems in which ice failure by splitting is of interest.

Some property measurements also utilize the splitting failure mode. The Brazil test is one example. This test is conducted by compressing a circular cylinder along a diameter. This loading develops a tensile stress perpendicular to the compression direction along the loaded diameter. The sample usually fails by splitting along this diameter. Except near the loading points, the value of the compressive stress in the loading direction is approximately three times as great as the tensile stress acting perpendicular to the loading direction. The value of the tensile stress computed in this 2-dimensional stress state at the time of failure is generally called the Brazil tensile strength, or simply the Brazil strength.

The methods of plastic limit analysis can be applied to the splitting failure mechanism. This approach is particularly useful in that boundary value problems can be readily formulated and solutions can be obtained with a very reasonable level of computational effort. The principal data requirement for this approach is a yield function, which describes the full-scale strength characteristics of the material. The present work uses the upper bound theorem of plastic limit analysis to compute upper limits for the loads in splitting failures that are developed by tensile and tensile/shear deformation. The basic data are derived from laboratory-grown freshwater ice measurements; however, the analysis could be readily repeated with any set of strength data.

The upper bound theorem of plastic limit analysis is a consequence of yield surface convexity and the normality flow rule. Normality means that the yield surface is the potential function for the plastic strain components, and has the geometric interpretation that the components of the plastic strain rate form a vector that is normal to the yield surface. The use of this theorem allows one to construct rigorous upper bounds for the failure load without knowledge of the details of the exact solution.

An important extension of the upper bound theorem states that if two materials have the same yield surface and one material follows the normality flow rule while the other does not, then an upper bound for the collapse load of the normal material is also an upper bound for the collapse load of the material with the non-normality flow rule. In this sense it is safe to estimate ice failure loads in the brittle range of ice deformation rates using a plasticity model. The extent to which such a model over-predicts actual failure loads can be judged from experiments, field observations, or analytical estimates.

Ice Yield Criteria and Dissipation Functions

The yield criterion used in this paper corresponds to laboratory-grown freshwater ice with an S2 crystal structure. This type of ice is isotropic within the plane of the ice sheet. The form of the full 3-dimensional yield expression is given by

\[
f(\Sigma) = a_1[(\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2] + a_3(\sigma_x - \sigma_y)^2 + a_4(\tau_{yz}^2 + \tau_{zx}^2) + a_6\tau_{xy}^2 + a_7(\sigma_x + \sigma_y) + a_9\sigma_z - 1,
\]

where

\[a_6 = 2(a_1 + 2a_3).\]
The z direction is perpendicular to the plane of the ice sheet while the x, y directions are contained in the ice sheet plane. The properties of this function are discussed in greater detail in Reference [1], and are summarized in the following.

We consider plane stress and plane strain analyses referred to the plane of the ice sheet. The analyses use discontinuous velocity fields in which \( \delta v \) and \( \delta u \) denote the normal and tangential velocity jumps at a discontinuity. In plane stress the yield function reduces to

\[
 f(\sigma) = a_1(\sigma_x^2 + \sigma_y^2) + a_3(\sigma_x - \sigma_y)^2 + a_6\tau_{xy}^2 + a_7(\sigma_x + \sigma_y) - 1, \quad (1)
\]

and the rate of energy dissipation per unit area on a velocity discontinuity surface is given by

\[
 D_0(\delta v, \delta u) = \frac{1}{2a_1} \left[ \sqrt{\frac{a_7^2 + 2a_1}{a_6}} \sqrt{4(a_1 + a_3)\delta v^2 + 2a_1\delta u^2 - a_7\delta v} \right]. \quad (1A)
\]

The plane strain yield function is given by

\[
 f(\sigma) = a_6[(\sigma_x - \sigma_y)^2/4 + \tau_{xy}^2] + (2a_7 + a_9)(\sigma_x + \sigma_y)/2 - (1 + a_9^2/8a_1), \quad (2)
\]

and the corresponding dissipation function is

\[
 D_0(\delta v, \delta u) = \frac{2a_1a_6 + 2a_1a_7(a_7 + a_9) + a_9^2(a_1 + a_3)}{2a_1a_6(2a_7 + a_9)} \frac{\delta v}{\delta v} + \frac{2a_7 + a_9}{4a_6} \frac{\delta u^2}{\delta v}. \quad (2A)
\]

where the constraint \( \delta v > 0 \) is required by the normality condition for the construction of plane strain upper bounds.

The four independent constants \( (a_1, a_3, a_7, \text{ and } a_9) \) in equations (1) and (2) can be determined by four independent strength measurements. The four strengths used for the present analysis are: \( T_x \), the in-plane uniaxial tensile strength; \( C_x \), the in-plane unconfined compressive strength; \( \sigma_b \), the in-plane biaxial compressive strength measured with the vertical direction stress free and equal stresses acting in horizontal directions; and \( \sigma_{ps} \), the plane strain compressive strength measured with vertical deformations restricted and one horizontal direction stress free.

We consider two sets of possible strength properties of freshwater ice which could be considered to be representative of two temperatures. At a temperature of \(-10^\circ C\), a reasonable set of strength ratios for the four parameters are given by \( T_x = 0.142C_x \), \( \sigma_b = 3.5C_x \), and \( \sigma_{ps} = C_x \). We refer to this set of values as "cold" ice strengths. Since the temperature dependences of the tensile and compressive strengths are not the same, the value of the strength parameters at a warm temperature would be somewhat different. We use the ratios \( T_x = 0.333C_x \), \( \sigma_b = 3C_x \) and \( \sigma_{ps} = C_x \) to be representative of "warm" ice. This choice of values for \( \sigma_b \) and \( \sigma_{ps} \) is somewhat arbitrary since there is little experimental data on these parameters for warm ice. All of the data that we use for cold and warm ice are not intended to be precise ice strength data since the purpose of this paper is to present general results and illustrate a calculation procedure rather than to present an analysis of the behavior of a particular type of
ice at specified test conditions.

The tension/compression quadrants of the plane stress and plane strain yield criteria are plotted in Figure 1. As described in Reference [1], the plane stress function is an ellipse while the plane strain function is a parabola. Three of the four strength parameters that are used to determine the yield functions are visible in this figure.

The plane stress yield functions for the warm and cold ice strength parameters are plotted in Figure 2. Note that both plots are normalized by their respective unconfined compressive strength. The plots, therefore, illustrate how a temperature change would alter the shape of the yield criterion, but they do not illustrate changes in absolute magnitudes.

Brazil Strength Test

The geometry of a Brazil test is indicated by the sketch in Figure 3. The present analysis applies to a specimen with its circular cross-section located in the x-y (horizontal) plane of the ice sheet. The thickness direction of the specimen corresponds to the vertical direction in the ice sheet. Thus the strength of the sample is isotropic with respect to diametrical loads.

The elastic stress distribution in a Brazil specimen depends on the width, w, of the load platens. For the extreme case of point loads, the stress distribution is presented by Timoshenko and Goodier [2]. A uniform tensile stress of magnitude \( \sigma_x = 2F/ndt \), where \( d \) is the specimen diameter and \( t \) is its thickness, acts perpendicular to the loaded diameter. The value of this stress component at the time of failure is called the Brazil strength, \( \sigma_B \). That is,

\[
\sigma_B = 2F/ndt, \tag{3}
\]

where \( F \) is the failure force. A compressive stress acts parallel to the applied forces along the loaded diameter. At the center of the specimen its magnitude is \( 6F/ndt \), or three times as great as the tensile stress. If the width of the platens is small, the above expressions are a good approximation to the stress state in the central portion of the specimen. Approximate analytical expressions for the complete stress state in a specimen loaded by platens of finite width are given by Chen [3].

If the mechanical behavior of the test specimen is assumed to be brittle, then failure should occur when the stress state reaches the yield surface. An approximation to a brittle failure criterion can be obtained by neglecting local stress concentrations at the load platens and assuming that the stress state in the central portion of the loaded diameter is the most critical within the specimen. As the load is increased, the stress state in this region follows the line \( \sigma_y = -3\sigma_x \) outward from the origin in Figures 1 and 2. Brittle failure should occur when the yield surface is reached. The failure force is then given by Equation (3), i.e.,

\[
F = \frac{ndt}{2} \sigma_x, \tag{3}
\]

where \( \sigma_x \) is the tensile component of the point of yield surface intersection. This stress component is thus the Brazil strength of a brittle material.

The failure mechanism illustrated in Figure 3 can be used to predict the failure load if the specimen is assumed to be a perfectly plastic material. This velocity field has been previously used by Chen and Chang [4] in a plasticity analysis of concrete splitting tests. The present analysis differs from the concrete work in that the yield criterion used to describe ice strength is not the same as that used for the strength of concrete. The velocity field indicated in Figure 3 consists of four rigid regions with velocity discontinuities between the regions. A rigid wedge of material...
Figure 1. Plane stress and plane strain yield criteria for "cold" ice

Figure 2. Plane stress yield criteria for "warm" and "cold" ice
Figure 3. *Brazil test geometry and failure mechanism*

Figure 4. *Mechanisms for ice floe and plate splitting*
is located at each of the load platens. The velocity discontinuity on the sides of these wedges is given by

\[
\delta u = \frac{V \cos \beta}{2 \tan(\beta - \theta)} + \frac{V}{2} \sin \beta,
\]

\[
\delta v = \frac{V \sin \beta}{2 \tan(\beta - \theta)} - \frac{V}{2} \cos \beta,
\]

where the angles \( \beta \) and \( \theta \) are defined in Figure 3 and \( V \) is the relative velocity of the platens.

The splitting action along the diameter of the specimen occurs with a normal velocity discontinuity of

\[
\delta v = \frac{V}{\tan(\beta - \theta)}
\]

and no tangential discontinuity.

An upper bound for the failure force is obtained by equating the rate of work done by the external forces to the rate of internal energy dissipated in the assumed velocity field. This is mathematically expressed by

\[
FV = \sum_{i=1}^{n} D(\delta v_i, \delta u_i) A_i
\]

(4)

where \( D(\delta v_i, \delta u_i) \) is the rate of energy dissipation on the \( i \)-th surface of discontinuity, \( A_i \) is the area of the \( i \)-th surface, and there are a total of \( n \) discontinuity surfaces. The dissipation function \( D(.,.) \) is given by Equation (1A) for plane stress applications and Equation (2A) for plane strain problems.

Since \( \delta v \) and \( \delta u \) depend on \( \beta \) and \( \theta \), Equation (4) will give an upper bound for any particular choice of these parameters. The choice of \( \beta \) and \( \theta \) that gives the least force from Equation (4) is the optimum set of parameters.

The results of the Brazil test calculation are summarized in Table 1 for platen sizes ranging from 1/6 to 1/12 of the sample diameter. The calculated Brazil strengths have each been divided by the uniaxial tensile strength for presentation in a normalized form. As previously discussed, the value of the "brittle" Brazil strength is simply the tensile stress at which the \( \sigma_y = -3\sigma_x \) line crosses the yield surface. The "perfectly plastic" Brazil strength was obtained by first computing the best upper bound for the failure force \( F \), using Equation (4), and then computing the corresponding Brazil strength from Equation (3).

<table>
<thead>
<tr>
<th>Plastic Analysis</th>
<th>Cold Ice</th>
<th>Warm Ice</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative Platen Size, w/d</td>
<td>Plane Strain</td>
<td>Plane Stress</td>
</tr>
<tr>
<td>1/6</td>
<td>1.52</td>
<td>1.35</td>
</tr>
<tr>
<td>1/8</td>
<td>1.38</td>
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<tr>
<td>1/10</td>
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<td>1.06</td>
</tr>
<tr>
<td>1/12</td>
<td>1.20</td>
<td>0.98</td>
</tr>
<tr>
<td>Brittle Failure</td>
<td>1.33</td>
<td>1.23</td>
</tr>
</tbody>
</table>
The Brazil strengths computed by the plasticity method indicated that the measured strengths should decrease if the sample size is tested with smaller width platens. The magnitude of this decrease is about 20% for a 50% reduction in platen width. No platen size effect is indicated for the brittle failure interpretation since the present brittle approximation neglects the stress conditions in the vicinity of the platens. For a platen width of about 1/8 of the specimen diameter, the plastic and brittle descriptions predict the same failure load. In view of the extent of data scatter that is typical of ice property tests, it is unlikely that Brazil strength data could be used to experimentally distinguish between plastic and brittle material descriptions.

The plane strain Brazil strengths are only 10% to 20% greater than the plane stress strengths for the same ice type. The plane stress analysis would apply to a thin specimen while the plane strain analysis applies to a long specimen. Again considering the expected scatter of experimental data, the length of a Brazil specimen should not have a significant effect on the measurement. The practical problems of fabricating and supporting a thin specimen or uniformly distributing the load to a long specimen would have greater effects on the measurement.

The examples from Table 1 illustrate that the Brazil strength should not be directly interpreted as the uniaxial tensile strength. The relationship between these two strengths depends on the shape of the yield criterion. The shape of the function in the tension/compression quadrant is of greatest importance. Since the shape would depend on the type of ice, orientation of samples, test temperature, and rate of deformation, it is possible that the Brazil strength could be either greater or less than the uniaxial tensile strength. It would be more appropriate to interpret the Brazil strength as the point on the yield surface where $\sigma_y = -3\sigma_x$.

The principal disadvantage of using the the Brazil test to measure ice properties arises whenever rate effects are important. If the rates of interest fall in the ductile/brittle transition zone where ice strengths are greatest, then the Brazil strength can be compared with other maximal ice strength data. For rates that are either faster or slower than the transition range, a rate dependent constitutive theory would be needed for direct comparison of Brazil strengths (or other biaxial test data) with uniaxial data.

Analysis of Ice Floe and Plate Splitting

Two mechanisms for ice floe or plate splitting are illustrated in Figure 4. The ice floe or plate is assumed to be a square with side $D_f$; however, the analysis may be readily generalized to other shapes. The structure, or indenter, is flat-sided with width $W$. The far-field stress distribution at the far edge of the floe is shown as uniform; however, its distribution is not important for the analysis with these velocity fields.

The Brazil mechanism is a direct application of the analysis discussed in the previous section. The floe size $D_f$ can be visualized as one-half of the Brazil specimen diameter.

The "shear splitting" mechanism fails the ice floe along a line reaching from one edge of the indenter to the far corner of the floe. This mechanism has only one free parameter, $e$, which describes the velocity discontinuity along the failure line. Generalizations of this mechanism, such as not requiring the failure line to pass through the corner of the floe, are possible but are beyond the scope of this paper.

The results of plane stress analyses of ice splitting failure for several floe sizes are illustrated in normalized form in Figures 5 and 6. These results apply to ice sheets that are thin compared to the indenter width. The failure force $F$ has been
Figure 5. Ice splitting pressure for "warm" ice as a function of relative ice sheet size.

Figure 6. Ice splitting pressure for "cold" ice as a function of relative ice sheet size.
divided by the ice thickness \( t \) and indenter width \( W \) to give the nominal ice pressure \( \sigma \); i.e.,
\[
\sigma = \frac{F}{tW}.
\]
The nominal ice pressure is normalized by the unconfined compressive strength, \( C_x \), of the ice.

The calculated failure forces for each of the two mechanisms are plotted in Figure 5 for warm ice and Figure 6 for cold ice. Since the calculated force for each mechanism is an upper bound, the actual failure force should be less than the smaller of the two calculated forces for each floe size.

There is little difference between the shear and Brazil splitting forces for warm ice (Figure 5) in floes less than four indenter widths in extent. These results suggest that one would expect to observe a variety of actual failure modes for warm ice floes in this range of floe sizes. The calculated Brazil splitting force is less than the shear force for larger floes.

There is a greater difference between the forces for the two mechanisms in cold ice. The Brazil-splitting curve in Figure 6 is consistently less than the shear splitting force for all floe sizes. This suggests that cold ice floes would be more likely to fail by tension-like splitting rather than shearing action.

The ice pressure curves in Figures 5 and 6 are cut off by the value \( \sigma/C_x = 3 \). This is consistent with the experimental data of Michel and Toussaint [5] and the subsequent analysis by Ralston [1]. The cut-off value is achieved by the floe splitting mechanism for ice sheets that are six to eight times as large as the indenter with the larger size corresponding to the colder ice. Ice sheets that are larger would be expected to crush locally at the indenter, and the forces would not be affected by the lateral floe boundaries.

Although the results in Figures 5 and 6 are upper bounds, they provide some guidance for the design of laboratory indentation tests. These data suggest that tests should be conducted with ice plates that are six to eight times larger than the indenter width if the results are intended to represent a large ice floe. These estimates of size requirements can probably be improved for a particular ice type and test condition by repeating the splitting analysis with the most appropriate ice property estimates for the anticipated test conditions. If smaller ice plates are used, some form of lateral restraint would be needed to eliminate the splitting failure mechanism. Some lateral restraint is usually present in experiments in the form of frictional contact between the ice plate and the base supporting structure. If this restraint were included in the present analysis, a somewhat smaller floe size would be calculated; however, it would be more conservative to design experiments based on the frictionless analysis.
REFERENCES


A generalized, constant-stress creep equation incorporating the effect of grain size for polycrystalline materials is presented and used in calculating ice stress-strain diagrams for constant stress rate loading conditions. It is shown that the effective modulus, determined from the initial part of these diagrams, increases with increase in rate of loading, increase in grain size, and decrease in temperature, in agreement with general observations on ice. Measurements on fresh-water and sea ice are presented and compared with the theoretical predictions. It is shown that the lower effective modulus for sea ice in comparison with that for fresh-water lake or river ice under similar conditions of temperature and rate of loading is due mainly to the differences in the microstructure and not necessarily to the salt content of sea ice. Sub-grains are shown to be a significant factor in controlling the deformation behaviour of ice.
The material properties of most concern to engineers are strength and deformation or effective modulus, often called the apparent Young's modulus. It is common engineering practice in evaluating materials to determine the slope of the stress-strain curve; and it is quite natural that this method has been extended to ice engineering. The test performed most often on ice in both the field and the laboratory is the deformation test, under quasi-constant displacement or load rate, for estimating strength and effective modulus. Beam bending, cantilever and the uniaxial compression or tension tests all fall into this category.

The slope of the stress-strain curve gives the elastic modulus of a material for loading conditions where non-elastic deformation does not contribute noticeably to the total strain. This is essentially the situation for most structural materials when the slope is practically independent of rate of loading, for example, steel at low homologous temperatures.

Ice is a non-linear viscoelastic material like some metals and alloys and may exhibit non-elastic deformation. Most operating temperatures in ice engineering are so close to the melting point that non-elastic deformation almost always plays an important role in the deformation process. Consequently, stress-strain diagrams are sensitive to rate of loading. In addition, the deformation modulus obtained from the slope of the stress-strain curves has been found to depend not only on rate of loading and temperature but on type of ice. Fresh-water ice has considerably higher values than sea ice under similar loading conditions. Such observations may appear to be peculiar to ice and difficult to understand, and because of the apparent differences there is a tendency among ice engineers to treat sea ice as separate from fresh-water ice.

This paper briefly describes the micromechanics that control the behaviour of polycrystalline materials including ice at high homologous temperatures. A phenomenological creep equation incorporating grain size is presented; and a numerical integration method is described for predicting the stress-strain diagram, using this creep equation and assuming conditions of constant stress rate. It is shown as well that the observed dependence of the initial slope of the stress-strain curve on stress rate, temperature and type of ice can be predicted. The substantially lower values of the initial effective modulus for sea ice in comparison with that for fresh-water ice under similar conditions of temperature and rate of loading are shown to be due mainly to difference in microstructure.

**Deformation Processes**

A minimum of three macroscopically observed strain components describe the deformation of any material irrespective of operational conditions. These are: a pure elastic and instantaneously recoverable deformation, $\varepsilon_e$, a delayed elastic or time-dependent recoverable strain, $\varepsilon_d$, and a permanent or viscous strain, $\varepsilon_v$. The total strain, $\varepsilon_t$, is given by the sum of $\varepsilon_e$, $\varepsilon_d$ and $\varepsilon_v$. 

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Uniaxial, constant-load creep of polycrystalline ice was used by Sinha [1] to demonstrate a method of examining the three strain components. A phenomenological viscoelastic creep model was proposed. Pure elastic deformation was assumed to be related to lattice deformation and the viscous component was attributed to intragranular deformation processes, particularly to the movement of dislocations. Delayed elasticity was hypothesized to be associated with intergranular sliding phenomena. These physical processes were later considered in the creep model, allowing the incorporation of the effect of grain size [2]. The total strain, $\varepsilon_t$, at time, $t$, in pure randomly oriented, polycrystalline material of grain size, $d$, subjected to a uniaxial stress, $\sigma$, at a temperature, $T$, was given by

$$E_t = \varepsilon_e + \varepsilon_d + \varepsilon_v$$

$$= \frac{\sigma}{E} + c_1 \left( \frac{d_1}{d} \right) \left( \frac{\sigma}{E} \right)^s \left[ 1 - \exp \left\{- (a_T t)^b \right\} \right] + \dot{\varepsilon}_v \left[ \frac{\sigma}{\sigma_1} \right]^n$$

(1)

where $E$ is Young's modulus; $\dot{\varepsilon}_v$ is the viscous strain rate for unit or reference stress $\sigma^1$; $c_1$ is a constant corresponding to the unit or reference grain size, $d_1$; $b$, $n$ and $s$ are constants; and $a_T$ is the inverse relaxation time. Both $\dot{\varepsilon}_v$ and $a_T$ vary with temperature and were shown to have the same value for the activation energy.

$$\dot{\varepsilon}_v (T_2) = \dot{\varepsilon}_v (T_1) \ S_{1,2}$$

and

$$a_T (T_2) = a_T (T_1) \ S_{1,2}$$

(2)

where $T_1$ and $T_2$ are two temperatures in Kelvin and $S_{1,2}$ is a shift function [1].

**Engineering Test**

Common engineering tests involve deforming a specimen until it fails. Load rates are usually monitored, particularly in the field, because it is difficult to measure strain and even more difficult to control it. It is relatively straightforward, however, to present results as a function of load rate or, if possible, stress rate.

Presentation of strength results on the basis of stress rate during loading, or average stress rate to failure, was recommended by Sinha [3] after he pointed out that conventional laboratory tests conducted under constant displacement rate are better presented as a function of stress rate rather than of nominal strain rate. It was shown later [4] that this method of presentation also removes some of the ambiguities in the results, caused by the effect of the stiffness of the test system. Stress rate analysis of strength is, in fact, extremely well suited to field tests, as may be seen in the work of Frederking and Timco [5] on sea ice. Conventional laboratory results can be compared more easily with conventional field results if they are presented on the basis of stress rate. The dependence of strength on the stiffness of the testing
machine can also be handled more conveniently by this approach [6]. Ice has proved to be stronger in this decade than previously supposed because of the use of bigger testing machines. In the following it is shown how the proposed creep equation can be used to predict the stress-strain diagram under constant stress-rate conditions. Analysis is limited to the initial period of loading that is usually used for the determination of effective modulus.

Theory

A constantly increasing stress path can be represented by a series of positive stress steps, \( \Delta \sigma \), each acting during an equal interval of time, \( \Delta t \), and approximating the stress rate \( \dot{\sigma} = \Delta \sigma / \Delta t \). Stress \( \Delta \sigma \) applied at \( t = 0 \) produces, according to the first term of equation (1), an elastic strain \( \Delta \sigma/E \) immediately after loading \( (t = 0^+) \), with negligible contributions from the second and third terms. At \( t = 1 \Delta t^+ \) there will be an elastic strain due to the total stress of \( 2 \Delta \sigma \), whereas the delayed elastic and viscous strains (according to equation (1)) will be the amounts produced by \( \Delta \sigma \) applied for the first period. At \( t = 2 \Delta t^+ \) elastic strain will correspond to the stress \( 3 \Delta \sigma \). For ice exhibiting \( s = 1 \) [1], delayed elastic strain at \( t = 2 \Delta t^+ \) is given by the sum of the strains produced by \( \Delta \sigma \) applied for \( 2 \Delta t \) and \( \Delta \sigma \) for \( \Delta t \). Viscous strain will be the sum of the strain produced by \( \Delta \sigma \) applied for the first increment of time \( \Delta t \) and the component produced by \( 2 \Delta \sigma \) applied for the second increment of time \( \Delta t \). Thus, at \( t = N \Delta t^+ \), equation (1) gives

\[
\varepsilon_t = \left( \frac{n+1}{n} \right) \frac{\Delta \sigma}{E} + \frac{c_r}{E} \sum_{i=1}^{N+1} \Delta \sigma \left[ 1 - \exp\left\{ - (a_T \left[ N+1 - i \right] \Delta t)^b \right\} \right] + \varepsilon_{v_1} \Delta t \sum_{i=1}^{N} \frac{1}{\sigma_0} \left( \frac{i \Delta \sigma}{\sigma_0} \right) (3)
\]

The above analysis is based on the assumption that the principle of superposition associated with a standard linear solid is applicable to delayed elasticity for the class of polycrystalline materials having \( s = 1 \). The analysis also assumes the applicability of the commutative law of creep for viscous flow. Successful application of the above treatment to the more general case of variable load has been discussed by Sinha [7].

The constants in equation (1) were determined from creep experiments to be [1,2]:

\[
E = 9.5 \text{ GPa} m^{-2}; \quad c_r = 9; \quad n = 3; \quad b = 0.34; \quad s = 1; \quad a_T (-10^\circ C) = 2.5 \times 10^{-4} \text{ s}^{-1} \quad \text{and} \quad \varepsilon_{v_1} (-10^\circ C) = 1.76 \times 10^{-7} \text{ s}^{-1}
\]

for the system using \( \sigma_0 = 1 \text{ MPa} \) and \( \varepsilon_{v_1} = 1 \text{ mm} \). Activation energy was found to be 67 kJ/mole. These values were used for the calculations.

Figure 1 presents a family of stress-strain curves computed from equation (3) for several stress rates at constant temperature and grain size. The temperature and grain size dependence of stress-strain diagrams are shown, respectively, in Figures 2 and 3, which show that the slope of the early portion of stress-strain curves, and hence the effective modulus, increases with increase in loading rate and decrease in
Figure 1. Dependence of stress-strain behaviour on stress rate ($\text{MN} \cdot \text{m}^{-2} \cdot \text{s}^{-1}$)

Figure 2. Dependence of stress-strain behaviour on temperature
Figure 3. Dependence of stress-strain behaviour on grain size

temperature. A decrease in the effective modulus with decrease in grain size is also evident.

Grain size (in the conventional sense) could be quite large. Sea ice, however, has sub-grains (often called platelets) owing to the high salt content of the water from which it is formed. There is a general belief that the sea ice substructure controls its mechanical properties. Weeks and Assur [8] reported that average plate spacing varied in the range of 0.3 to 0.6 mm in laboratory-made saline ice 30 cm thick. Nakawo and Sinha [9] found that the width of the platelets varied from 0.4 mm to about 1.0 mm in natural first-year sea ice from the High Arctic. They give information on the dependence of the microstructure on weather conditions. If deformation behaviour depends on the sub-grain size, sea ice could be considered as fine-grained ice. It would be expected, therefore, to have a lower effective modulus than the larger grain size fresh-water ice.

Figure 4 presents an example of the relative contributions of the elastic, delayed-elastic and viscous components during the initial $10^{-3}$ strain. It shows that the slope in the apparently linear part at the beginning of the stress-strain curve is governed substantially by the delayed-elastic effect; the viscous component becomes significant only at the higher strains. As the first and the last terms in
Figure 4. Stress versus strain showing strain components

Figure 5. Comparison of theory and measurement of the stress rate dependence of the effective modulus
equation (3) do not have any grain size effect, it is the second term that determines
the grain size dependence presented in Figure 3. Finer grained ice is shown to have
more delayed-elastic strain and hence more recoverable strain. This explains why sea
ice is apparently more "rubber-like" than fresh-water lake or river ice.

Comparison with Experiments

Observations by Sinha [3] during compressive strength tests on S-2 ice of average
cross-sectional grain diameter of 4 to 5 mm at -10°C are presented in Figure 5.
Secant moduli to 0.5 and 1.0 MPa are given, as well as some measurements for sea ice
made by Murat [10] during four-point beam bending tests at -5°C. The ice had a
salinity of 5 to 7%. Strain gauges were attached to the compression as well as
tension surfaces of the beam. Both sets of measurements were in good agreement; the
average values are given in Figure 5. Levels of stress were low (< 0.5 MPa) and
varied during each test (private communication).

A quick glance at Figures 1 to 4 shows that the stress-strain curves do not have
a linear section in the strict sense. It would be more realistic, therefore, to
calculate the secant modulus $E_\sigma$ to some chosen stress level $\sigma$. Figure 5 gives
examples of calculations relevant to the experimental observations for both coarse­
grained and fine-grained ice. The grain diameter of 0.5 mm for the fine-grained ice
was chosen because Murat's [10] measurements were on saline ice made in the laboratory
and should have a platelet spacing similar to that of the ice used by Weeks and
Assur [8].

Agreement between the theoretical predictions for fine-grained ice and the
measurements by Murat can be considered as fair. The salinity of the ice used is
similar to that usually observed for first-year sea ice. The presence of brine at
grain and sub-grain boundaries could certainly influence the deformation properties,
but it appears that the lower value of the elastic modulus for sea ice, and hence its
greater ductility, can be predicted reasonably well on the basis of grain size alone.

Acknowledgement

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References


MID-WINTER MECHANICAL PROPERTIES OF ICE IN THE SOUTHERN BEAUFORT SEA

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Ice property measurements were carried out during ice breaking trials of the "CANMAR KIGORIAK" during the winter of 1979-80, primarily in landfast ice in the Southern Beaufort Sea in the vicinity of McKinley Bay. They included crystallographic analysis to establish ice type and structure, temperature and salinity profiles, and uniaxial compression and "Brazil" strength tests. The work was performed on board ship at the time of the trials.

The ice under study included samples of first-year ice covers and multi-year pressure ridges. Sampling and measurement techniques are described. The strength results are discussed in terms of salinity, loading rate, grain structure, sample orientation and position in the ice cover. Strength results were found to agree generally with values in the literature.
In assessing the ice breaking performance of vessels it is necessary to have a complete knowledge of ice properties. Thickness, temperature, salinity where applicable, friction coefficient and flexural strength are normally of interest. In January 1980 the National Research Council of Canada collaborated with Dome Petroleum Ltd. during the ice breaking trials of the Canmar Kigoriak in a program that included grain fabric analysis to determine ice structure, type, grain size and orientation; ice temperature and salinity measurements for brine volume determinations; and compressive and "Brazil" strength tests. Measurements were carried out at the time of the trials in an improvised cold room on board ship.

The primary test area was in the Southern Beaufort Sea in the vicinity of McKinley Bay. The ice cover in this area was dynamic, with frequent movements into December, but it was landfast at the time of the January trials. Samples of ice from the trials area (first-year ice) were analysed, as was multi-year ice collected during a probe into the polar pack in October 1979. Measuring techniques are described, selected data presented, and strength results discussed in terms of salinity, loading rate, grain structure, and position in the ice cover. Complete results of the measurements appear in an earlier report [1].

Experimental Procedures

A total of 17 vertical cores of first-year sea ice were collected and analysed. Core diameter was 75 mm. In some cases temperatures and salinities were measured along the core immediately after retrieval in order to obtain representative profiles through the ice cover. Initial analysis involved a visual description of the core, including ice colour, air bubble content, brine drainage channels and sediment bands. Following this a decision was made on the further analyses to be performed. The cores were then marked and cut into appropriately sized pieces depending on whether specimens for thin section studies, salinity or strength measurements were required. All work was done in the improvised cold room on board ship, where temperature was a few degrees warmer than the ambient temperature, which ranged from -35 to -25°C.

Thin sections were made to determine general grain structure, size and orientation. A warming plate was used to prepare the thin sections, which were observed and photographed between crossed polaroids.

The compression and Brazil tests were done on a 50 kN capacity, motorized screw-drive test machine (Figure 1). Although this press is capable of cross-head rates from $3 \times 10^{-3}$ to $5 \times 10^{-2}$ mm/s, only a nominally constant rate of $3 \times 10^{-2}$ mm/s was used for these tests. Load was measured with a 50 kN capacity load cell and recorded continuously. The ends of the specimens were cut on a band saw, care being taken to make the ends perpendicular to the axis of the cylinder, parallel to each other and flat. Compression specimens were typically 120 mm long by 75 mm diameter cylinders.
No further end preparation was done, other than brushing the cylinder clean. The length and diameter of each specimen were measured. "Maraset" compliant platens (2) were used to minimize the effect of irregularities and to reduce the radial stresses at the ends of the specimens. Loading was monotonic to yield or brittle failure. Strength was calculated from the maximum load each specimen was able to support and its cross-sectional area. From the continuous records of load versus time, loading stress rates [3] were determined for each test. The Brazil tests were performed on cylinders about 30 mm long by 75 mm in diameter. Load was applied perpendicular to the axis of the cylinder. Brazil strength, $\sigma_B$, was calculated using $\sigma_B = 2K \frac{P}{\ell d}$, where $K$ is concentration factor (assumed to be 6 in this case), $P$ is failure load, $\ell$ is length, and $d$ diameter. After testing, the salinity of each specimen was measured.

Results and Discussion

Numerical results of over 60 strength tests conducted on samples of first-year sea ice are presented in Tables I and II. As all specimens were obtained from vertical cores, the loading direction was parallel to the ice growth direction for compression tests, and perpendicular to ice growth direction for Brazil tests.

All the compression tests were run at a nominal strain rate of $2.5 \times 10^{-4}$ s$^{-1}$ and a temperature of $-26 \pm 1^\circ$C. It should be borne in mind that the test temperatures were much lower, particularly for the deeper samples, than the naturally occurring temperatures in the ice cover. Several trends were apparent from the results. First, there is a general increase in strength with increasing sample depth in the ice cover. Secondly, there is a pattern of ductile yield in the upper part of the ice cover and brittle failure in the lower part. Figure 2 illustrates a thin section of a typical core. Grain structure analysis showed the upper part of the ice cover to be small-grained (approximately 2 mm diameter) granular ice, and the lower part to be larger-grained (approximately 10 mm diameter) columnar-grained ice. A typical temperature
TABLE I. Uniaxial Compression Tests - First-Year Ice Vertical Cores

Test temperature = -26°C; Stress rate = 0.13 - 0.30 MPa·sec\(^{-1}\); Strain rate = 2.5 \times 10^{-4} sec\(^{-1}\) (nominal)

<table>
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<th>Sample Depth, cm</th>
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<th>Stress Rate, MPa·sec(^{-1})</th>
<th>Time to Failure, sec</th>
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* brittle type failure

228
TABLE II. Brazil Tests - First-Year Ice Vertical Cores

Test temperature = -20°C; Stress rate = 0.15 - 0.42 MPa·sec⁻¹

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<th>Sample Depth, cm</th>
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and salinity profile is given in Figure 3. In general, the top 20 cm of the core had an average salinity of 10-15%, and the remainder an average salinity of 5-6%. Similar profiles have been observed for first-year sea ice in the Beaufort Sea [4].

Brine volume is often used as an independent variable in evaluating the strength properties of sea ice [5]. The compressive strength results have been presented on this basis in Figure 4a. A gradual decrease in strength with increasing brine volume is evident, but it is not particularly strong. Recent investigations of the mechanical properties of ice have shown that they are load history dependent [3]. The same results, plotted versus stress rate in Figure 4b, show less scatter and indicate a trend towards increasing strength with increasing stress rate.

Peyton [6] has found a similar strength trend for first-year sea ice. The specimens have been categorized in two groups to make comparison with Peyton's results easier: "top ice," granular ice from the top 20 cm of the ice cover; and "bottom ice," large-grained, columnar-grained ice from the lower part of the ice cover. Also shown in Figure 4 are strength values of multi-year ice [7]. In terms of brine volume, multi-year ice is weaker than first-year sea ice, whereas in terms of stress rate its strength is indistinguishable from that of first-year sea ice. The strong influence
Figure 3
Typical temperature and salinity profile of first-year sea ice, Beaufort Sea, January 1980

Figure 2
Vertical thin section of first-year sea ice showing grain structure variations with depth
(Total core length ~0.85 m, Beaufort Sea, January 1980)
Figure 4. Compressive strength vs (a) brine volume and (b) stress rate; test temperature -20°C to -27°C, nominal strain rate $2.5 \times 10^{-4} \text{ s}^{-1}$.
of stress rate on compressive strength shows the importance of this factor in comparing the strength values reported by various investigators.

With regard to the Brazil tests, the results indicate strengths of the order of 3 MPa for granular ice and 2 MPa for columnar ice. This is in good agreement with other measurements of Brazil strength for sea ice of comparable brine volume [8].

Figure 5a presents the stress-time curves from two tests; the solid line illustrates ductile yield of granular ice, the dashed line, brittle failure of columnar-grained ice. Also shown are straight lines approximating the loading stress rate for each case. Sinha [9] has suggested that it is possible, with an appropriate rheological model and a knowledge of ice characteristics (temperature and grain size), to generate a stress-strain curve from stress history. This approach has been applied to the actual stress histories shown in Figure 5a, and the resulting stress-strain curves are presented in Figure 5b. Unfortunately, strains were not measured during

Figure 5
Representative stress histories (a) and calculated stress-strain responses (b) for first-year sea ice, -26°C (solid line ------, 2 mm grain diameter; dashed line ----, 10 mm grain diameter)
the tests now reported so that it is not possible to verify these predictions. Peyton [6] indicated strain modulus values of up to 5 GPa at a temperature of -21°C and loading rates comparable to those shown in Figure 5, indicating that the predictions are of the right order. There is a real need, however, for stress-strain-time data to help in developing a constitutive equation for sea ice that takes into account grain structure and size, temperature, and salinity. In future field tests it is planned to obtain strain-time in addition to stress-time data.

Conclusions
(1) Compressive strength and Brazil strength values determined in this study show general agreement with comparable data in the literature.
(2) Compressive strength and failure behaviour of vertically loaded specimens of first-year sea ice vary through the depth of the ice cover in a systematic fashion that appears to be related more to grain structure than to salinity or brine volume, at least for test temperatures less than -20°C.
(3) Stress rate appears to be an appropriate basis for the measurement and interpretation of compressive strength under relatively uncontrolled field test conditions.
(4) Measurements of ice characteristics and properties can be carried out successfully on board ship in improvised facilities, thereby reducing the problems associated with transporting ice samples to "southern" laboratories.

Acknowledgements
The authors wish to thank Dome Petroleum Ltd. for the opportunity of participating in the ice-breaking trials of the Canmar Kigoriak. The logistic, transportation, and accommodation support provided by Canmar made the involvement possible. Finally, the encouragement, moral support and aid of the trials team, Captain, cook and crew are gratefully acknowledged.

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References


ABSTRACT

The Navigation Season Extension Program is an effort to move toward year-round waterborne commerce using the Great Lakes, St. Lawrence Seaway, and their subsystems. These systems are comprised of channels and rivers which must be cleared sufficiently to permit year-round transit. Ice disposal is an integral part of this clearing effort, and the dimensions of the ice that must be handled is of fundamental importance in sizing any hardware for this purposes. Presented in this paper are the results of data collected during the 1978 - 1979 winter ice trials of the U. S. Coast Guard 140' WIGB icebreaking tug. The trials were conducted in the St. Marys River and the Whitefish Bay area of Lake Superior. A statistical examination is made of the broken ice sizes produced as those ice pieces were affected by the ship speed, the air bubbling system, and the type of ice involved. All of this ice was snow covered when broken. Presented are the mean and standard deviation of the two horizontal dimensions (the largest dimension and the corresponding orthogonal dimension) of the broken ice pieces generated for each run. The data collection and analysis were conducted photographically, and the ice sizes are accurate to within ±5%. The results of this study are compared to a similar statistical study of broken model ice dimensions generated during model tests of the "dual draft" icebreaker.

Introduction

The Navigation Season Extension Program is an effort to move toward year-round waterborne commerce on the Great Lakes, the St. Lawrence Seaway, and connecting channels and rivers of these water systems. The full realization of this commercial potential awaits a solution that will keep the rivers and channels sufficiently clear so that bulk and ore carriers can negotiate these ice filled waterways. The critical factor in the winter navigation is the ice clogged channel. One of the principal elements needed for the development of successful clogged channel clearing procedures/equipment is the dimensions of the broken ice. This paper is a unique attempt to document statistically the expected ice piece dimensions produced during an icebreaking operation.

The product of vessel traffic through solid sheet ice, repeated icebreaking and refreezing of broken ice, is the problem ice. The types of ice that archetypally cause the channel blockages are usually frazil and brash ice. Frazil ice is defined by the world meteorological organization
as fine spicules of plates of ice, suspended in water. Brash ice is defined
by the WMO as an accumulation of floating ice made up of fragments less than or
equal to 2 m across [1]. Brash ice is often used synonymously with slush ice. A
combination of brash and frazil ice can reach a thickness 3 to 5 times the
original uniformly thick ice covering the channel.

In-situ ice thickness measurements were made by Voelker and Friel [2] in the
Little Rapids Cut area of the St. Marys River system. Large variations in the
thickness of brash ice were measured, some which exceeding the 3.4 m capability of
the measuring equipment. Gatto [3] noted that a large discharge of fragmented
solid ice, or an obstacle in the channel that impedes downstream passage of ice
can lead to ice jams. The types of channel obstacles (man-made or natural) are
limitless. Examples could be natural changes in channel width or changes caused
by bridge piers and abutments, and ice booms; changes in channel depth caused by
sand bars; changes in channel slopes; or changes in river or channel direction
producing sharp bends. With the exception of the natural Spring melt, ice must
first be broken to dispose of it.

Icebreaking is the first step in the process of clearing a channel. The new
140' W102 Coast Guard icebreaker will likely accomplish this initial step although
an air cushioned vehicle could also be used. The second step is ice disposal.
Major [4] suggests that brash ice disposal is only a requirement in restricted
channels, where ice build-up can block or delay shipping. Thus, the evidence
implies that ice disposal, and/or ice control are required in restricted channels.

Research and Engineering Laboratory (CRREL) conducted a preliminary investigation
for the U. S. Coast Guard of various clogged channel ice disposal methods for the
St. Marys River system. A follow-on study was conducted for the Coast Guard by
Vance to assess in detail the four most economical ice disposal methods and to
identify disposal sites along the
St. Marys River. He concluded that mechanical displacement on top of adjacent ice
cover was the most feasible. The first CRREL study did not consider ice
dimensions that specific hardware could effectively handle. This study was
initiated to estimate the expected sizes of ice with which any disposal method
must deal.

Instrumentation

A 35-mm still camera possessing a 55-mm focal length lens, was mounted on the
deck of the 140' W102 cutter and was oriented to photograph a region of the wake.
For calibration purposes, a pole with a .3048 meters demarcation was also placed
on the deck. The geometry of the instrumentation setup, namely, the relationship
between camera, calibration pole and ice level, reveals that lines of sight from
camera lens to calibration pole to ice level intersect the water at angles of less
than
11.5 degrees. The shallow angles of intersection greatly influence the film
measurements of width for a piece of ice. To simplify the critical problem of
scaling, film measurements for any particular frame were restricted to

This number in brackets is the reference number.
those pieces of ice that were contained in the 2.32 degree vertical plane angle that subtended the .3048 meters calibration mark. This also eliminated the possibility of lens edge distortions. Precautions were taken to avert any temperature effects on the film and camera lens. Throughout the film reading procedure, which used an IBM Telstar Computer and Universal Film Reader, an effort was made to select randomly frame by frame pieces of ice that ranged from large to small. The ship pitch angles averaged 1.3 degrees. Ship propeller shaft vibration frequencies were between 10 and 20 Hz, which did not affect the film reading accuracy because of the high speed film and shutter speeds used. Thus the ship motions and vibrations did not significantly affect the accuracy of the results. The ship speed was measured by both a mirage finder and a doppler radar. All data were taken during steady ship motions and no ramming. It is considered that the film measurements in most cases are accurate to ± 5%.

Ice Statistics

An icebreaker moving in a channel breaks ice into pieces of various dimensions. Uniformly thick ice failure during continuous icebreaking is characterized by a series of radial cracks eliminating from the icebreaker bow. The radial cracks are closely followed by one or multiple rows of circumferential cracks just prior to failure, Milano [6, 7]. The broken ice then subdivides into four or more principal wedges. The number of circumferential rows of cracks depends mainly on the ice thickness and the icebreaker speed [6]. As the ice thickness increases, the number of rows decreases. As these ice wedges break, they leave side cusps which also break along the side of the vessel. The maximum cusps width theoretically varies linearly with ice thickness. A similar pattern occurs for brash ice [8]. This broken ice forms the sampling population. A sample size greater or equal to 50 is sufficient to empirically confirm a statistical assumption, Blalock [9]. If the sample size is less than or equal to 30, one should be cautious unless the population has a known distribution. Thus a type II or B error is possible for this statistical study. Photographic statistical inference is not new to cold regions research. Hibler, Weeks, and Mock [10] used empirical data to derive pressure ridge statistical distributions. They found that the pressure ridge height and distances between ridges to be close to a poisson distribution. Hibler, et al., used NASA satellite photo data to determine pressure ridge spacing along a homogeneous linear track. The sampling sizes used were 40, 48, and 63. This study used the goodness of fit test to check the null hypothesis, or the assumed statistical distribution.

Results

The null hypothesis for this study was that the ice size distribution was lognormal. A typical lognormal cumulative distribution functions (CDF) of the length of a broken ice slab are given in figures 1, 2, and 3. These figures represent the ice lengths generated when, respectively, the air bubbler system was off, on, and only the front air bubblers were in operation. The snow covered solid ice thickness for the test run 1130, figure 1, was 35.6 cm. For test run 2100 (figure 2), the ice was 33 cm thick; for test run 4400 (figure 3), the brash ice was approximately 122 cm thick.
Figure 1. Empirical CDF Log Normal Overlay for Test Run 1139

Figure 2. Empirical CDF Log Normal Overlay for Test Run 2190

Figure 3. Empirical CDF Log Normal Overlay for Test Run 4400
The probability density function for the lognormal random variable \( X \) is

\[
f(X) = \frac{1}{\sqrt{2\pi} b X} \exp\left[ -\frac{\ln X - a}{b} \right]^2, \quad X > 0, \quad a > 0, \quad b > 0.
\]

\[
\text{Prob}(X \leq X_0) = \int_0^{X_0} f(X) \, dX,
\]

where \( X \) = length of ice piece, \( E(X) = \exp(a + b^2/2) \), \( \text{Var}(X) = (\exp(b^2) - 1) \, E(X) \).

A summary of the ice dimension statistics are given in Tables I and II. For the distributions that are not log normal, the data could be plotted, and an exact analytical expression developed. The mean values in the tables are the expected dimension (length) of the longest side of an ice slab. The maximum length of ice found during a run and the orthogonal width found during a run are listed also in the tables. The sample sizes for two-thirds of tests runs were greater than or equal to 30. Generally, the number of ice pieces appears not to affect the expected value of the maximum length.

**TABLE I. SUMMARY OF MAXIMUM ICE LENGTHS FOR TEST OF COAST GUARD CUTTER (WAGB)**

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\( a \) Expected length based on lognormal distribution of lengths
\( b \) Square root of variance of length based on lognormal distribution of lengths
\( c \) departure from normality (if skewness is 0, the population is normally distributed)
\( d \) Braak Ice
\( e \) Forward and Aft
\( f \) Goodness of fit of the lognormal distribution questionable; use of empirical CDF preferred.
The skewness of the data was relatively small. A normal distribution, however, would have zero skewness. The standard deviation given in the tables indicates the relative amount of dispersions of the ice lengths from the mean. Although both the length and the orthogonal width were recorded, the expected value of the largest dimension will be the basis of fashioning ice disposal methods or hardware. The ice thickness for this part of the ice trials ranged from 27.9 to 121.9 cm.

Figures 4 through 7 are least square fit to data plots of the mean ice length versus the product of dimensionless ice flexural strength and propeller thrust. The thrust was uncorrected for thrust deduction and open-water model test results. The thrust is an estimate of the magnitude of the icebreaking force exerted on the ice. The dimensionless parameters are composed of

$$\sigma_f = \text{the ice flexural strength},$$
$$h = \text{ice thickness},$$
$$T = \text{propeller thrust force},$$
$$\rho_w = \text{mass density of water},$$
$$g = \text{acceleration due to gravity},$$
$$Sc = \text{snow-cover depth}.$$ 

Figures 4, 5, and 6 used measured and assumed flexural strengths, to calculate $\sigma_f/\rho_w gh$. This appears to be a reasonable approach since the ice for those cases was approximately equal in temperature and thickness. Figure 7 includes the effects of snow cover. Figure 8 shows the relationship between mean ice length and the icebreakers speed in relatively uniformly thick ice and brash ice of the same thickness. Larger ice dimensions for thinner solid ice sheet compared to the thicker brash ice were observed in figure 8. This result could be anticipated based on Milano's [8] findings. However, the data given in figure 8 also indicates that brash ice lengths increase and thinner uniform ice lengths decrease for a corresponding breaker speed increase. Ice thickness was the dominant parameter determining broken ice dimensions.
Figure 4. Mean Ice Length versus Dimensionless Parameters (Air Bubbler System Not in Operation)

Figure 5. Mean Ice Length versus Dimensionless Parameters (Air Bubbler System in Operation)

Figure 6. Mean Ice Length versus Dimensionless Parameters (Forward Air Bubblers in Operation)
The operation of the air bubbler system had relatively little affect on mean ice length. The plot indicates a slight increase in mean ice size with dimensionless parameters increase. Milano [7] questions the significance of snow-cover on icebreaking with the exception of increased frictional resistance to icebreaking. Edwards, Lewis, Wheaton, and Coburn [11] show that one foot of snow increased resistance 25.48 tons during trials of the USCG Icebreaker Staten Island.

Arctec, Inc. conducted model tests of a proposed Coast Guard dual draft icebreaker in their model basin in a frozen saline solution. A 1/48th geometric scale was used for the model tests, using the geometric similitude techniques. Using the data in figure 4, one notes that the model ice was broken into pieces the same order of magnitude as the full scale ice.

Conclusions

Although it is desirable to use large sample sizes, this was not generally possible during this investigation. However, using the goodness of fit hypothesis test, it was found that most of the data was lognormal. There was general agreement with Milano in terms of the relationship between broken ice dimensions and ice thickness. There was agreement also for thinner ice and the relationship to icebreaker speed. Finally, although this work was directed toward the solution to the clogged channel clearing problem, the possibility of modeling broken ice dimensions is presented.
Acknowledgements

The authors wish to thank Mr. Lawrence Ross, Mr. Robert Nelson, and Mr. W. Garnier of the U. S. Army Aberdeen Proving Grounds for the photo data collection and the statistical analysis of the data. Dr. George Vance formally of the U. S. Army Cold Regions Research and Engineering Laboratory, and Mr. Robert Majors of Arctec, Inc. for the unpublished full scale and model data, respectively.

Bibliography


CONDITIONS IN BRASH ICE COVERED CHANNELS
WITH REPEATED PASSAGES

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Water Resources Engineering
University of Luleå

Sweden

ABSTRACT
Repeated ship passages in an ice covered channel cause an accelerated growth of brash ice distributed in the track and under the solid ice. In order to investigate the growth and the distribution of brash ice, field experiments were made. The harbour ice breaker VALKYRIA passed with different frequencies in two channels, 200 metres long, during the winter 1978/79 in the harbour of Luleå.

Observations were made in drilled holes on lines across the tracks. 42 profiles were observed. An equivalent brash ice thickness, $h_{ekv}$, is used. $h_{ekv} = 2.5$ metres was found in the end of March. The water content was found to increase almost linearly with depth from about 20% at the surface to 100% at the bottom of the brash ice. The brash ice growth is calculated by the degree-day method in modified form, and compared with $h_{ekv}$. The observed distribution of block sizes is presented.
INTRODUCTION

In the Gulf of Bothnia the harbours on the Swedish coast have been open for year-round navigation since 1970. Most of the harbours are situated in shallow river estuaries inside large archipelagos with landfast fresh water ice. Due to the fact that the ships are restricted to dredged channels in the harbour and in the archipelagos the same tracks will be used during the whole winter with repeated passages and freeze-ups. Due to the presence of water close to the surface in the broken ice there will be an extra growth of ice in the tracks hindering further navigation.

In order to investigate the main factors causing the accelerated growth of ice, the distribution of the ice and to present a growth model, full scale field experiments have been made in 1978/79 in the Luleå Harbour, Sandkvist [6].

Brash ice is usually defined as fragmented ice in which individual pieces are between 0.02 to 2.0 metres across, Mellor [3]. The properties and distribution of the ice in a ship's track depend on the form of the ship, the kind of ice the ship is passing through, the speed, etc. The propulsion system also influences the character of the ice generated.

EXPERIMENTAL PROGRAM

Because of the intense traffic with different kinds of ships under varying conditions in the regular fairways an isolated experimental area was used.

Ice breaking

In the harbour two different tracks have been broken with different time schedules. The only vessel passing through the channels was the harbour ice-breaker VALKYRIA. The tug is 32 metres long, with a deep-draught of about 5 metres and is powered by a 3500 hp engine. When passing the tracks the speed, rpm and pitch of the propeller, number of stops, etc. were observed over the two 200-metre stretches. The VALKYRIA is classed as IA Super.

Figure 1. The harbour ice breaker VALKYRIA passing the tracks for the first time Dec.8.
It was planned to break the north track once a day and the south track once a week with several passages each time. Because of the severe winter and a heavy shipping season the planned program was changed. Still, the breaking frequency was different in the two tracks.

When the VALKYRIA entered the tracks she had achieved her equilibrium speed using constant rpm and constant pitch of the propeller. At each passage the crew noted the passing time, number of stops and rammings, used rpm and pitch, and also the starting time for every passage and the air temperature, fig. 1.

**Observations**

Between the passages observations of the ice conditions were made. Observation lines were placed perpendicular to the track. Along a measuring tape holes were drilled from undisturbed ice at one side of the track over the brash to undisturbed ice at the other side. For each hole along the measuring tape snow depth, distance from ice surface to water level, total ice encountered, water layer thicknesses and the amount of ice slush were observed, fig. 2.

![Figure 2. Definition sketch showing the different parts of a drilled hole. In each hole all ice thicknesses and water lenses were observed. Snow thickness and distance from water level to ice surface were also observed. $X_i$ is the distance noted from "0" at the measuring tape. Every hole (i) influences half the distance to the next adjacent holes.](image)

The amount of ice and water in the channel can be approximated from the core data. Each hole represents the conditions in the profile half way to the adjacent holes.

The brash contains water. The quantity and the distribution of water in the brash was determined from the observations.

The presence of heavy blocks and irregularities in the distribution of brash prevent the ice-breaker from holding a perfect course along the centerline of the track. Here it is assumed that the used width is equal to $2 \times \text{BEAM}$, where BEAM is the width of the ice-breaker. A width of the track corresponding to $2 \times \text{BEAM}$ was used when the water content in the observed profiles were calculated from the observed results.
In order to relate the accelerated growth of ice to a representative ice thickness the broken ice was assumed to be evenly distributed over the width of the track as shown in fig. 3. The observed distribution is, however, more complicated. The ice is "piled down" along the sides of the track as is also shown in fig. 3.

![Figure 3. Definition sketch showing a typical transverse ice profile, A, observed perpendicular to the track. The profile produced when using an equivalent ice thickness, $h_{ekv}$, describing the accelerated growth of ice in a track, B.](image)

**RESULTS**

**Air temperature**

All daily mean air temperatures were observed at a weather station near the harbour. During the winter 1978/79 the total sum of negative degree-days reached $1700^\circ$C-days. The winter was colder than normal. During the last 90 years the mean yearly sum was $1190^\circ$C-days. The weekly mean temperatures are shown in fig. 4.

![Figure 4. Weekly mean air temperature for Luleå November 1978 - April 1979. The total sum of negative degree-days was 1700 ($^\circ$C-days). The average sum for the last 90 years is 1190 ($^\circ$C-days).](image)
**Ice-breaking results**

The breaking frequency was different for the two tracks. All dates for negotiating the tracks are presented in table 1 where the velocities $V_0$ and $V_1$ are noted. $V_0$ is the velocity which the VALKYRIA would obtain in open water using the same rpm and pitch used in the ice when she obtained the observed velocity $V_1$.

Table 1. The breaking results in the two tracks during 1978/79. $V_0$ is the resultant velocity when negotiating the track. $V_1$ is to be compared with $V_0$, the velocity in open water with no ice using the same power. All velocities in metres/sec. $\Delta S$ is the sum of neg. degree-days since last passage ($^{\circ}$C-days)

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* including several stops and rammings.
In table 1 are also noted the sum of degree-days since last passage, \( \Delta S \). The velocity \( V \) increases when passing more than once the same day in the south track.

**Static growth of ice**

The ice period in Luleå usually lasts from mid November till mid May. During the winter studied the first ice cover was formed by November 20. The ice left the harbour, carried by the spring flood, on May 15. The maximum ice thickness, 82 cm, occurred in the beginning of April, see fig. 7.

**Brash ice profiles**

During the winter 24 perpendicular lines in the north track and 18 perpendicular lines in the south track were observed. Two typical observation lines are presented in fig. 5, which shows observation results from December 20 after 8 passages and 430°C-days since ice cover formation and from February 20 after 27 passages and 1460°C-days, both observed in the north track.

Difficulties occurred when observing the distribution of water and ice at the brash ice surface, since the ice cover was not safe for walking on. However, the distribution of water at the brash ice surface can be roughly estimated to about 20 %. Noble et al. [4] observed a surface ice water content of about 20 % in their laboratory tests.

The water content in all 24 plus 18 observation lines was found to increase almost linearly with distance from the ice surface, see fig. 5.

**Block sizes**

When passing the track the ice breaker penetrates the ice by bending, cracking and crushing. The form and size of the broken ice mass, the brash, depends on many factors as the hull form, the ice thickness, the block sizes before the passage, etc.

By using all observed vertical ice block thicknesses a representative block size for each observation line has been determined. In fig. 6 the results from some observation lines are presented.

During the first passages the ice cover was thin and was broken into large sheets, which were packed above each other. The sheets froze together and the shape changed from thin sheets to more spherical blocks. The almost spherical blocks just tumbled around and did not crack when passing the track. The block sizes can be compared with the static ice thickness, fig. 7. The mean block thickness, which was 0.35 metres in mid December, increased to about 0.7 metres in March, observed in both tracks.

**The growth of brash ice**

The growth of brash ice is presented as an equivalent brash ice thickness, \( h_{ekv} \), fig. 3. \( h_{ekv} \) from the 24 and 18, respectively, observation lines is presented in fig. 7.

As seen in fig. 7 the equivalent ice thickness, \( h_{ekv} \), varied from one transverse section to another on the same day. This shows that there were irregularities along the tracks. Ice mass concentrations in some parts of the track was probably due to stops that the ice-breaker made. When the ice-breaker could not make headway in the track after a freezing period it had to back and ram. The direction changes formed ice packs behind the ice-breaker which froze together and provided increased resistance later. There are no major differences in the ice production although there are differences in breaking frequency.
Figure 5. Observed brash ice profiles, December 20 and February 20, in the north track including the estimated water content in the brash ice. All drilled holes, top, and the estimated water content in percent at each 5-centimetre-level from the surface down to the deepest part of the brash profile, below, are plotted. 0% to the left and 100% to the right. The left figure shows the profile after 8 passages and with a sum of 430 negative degree-days and the right figure shows the profile after 27 passages and 1460 negative degree-days.
The produced volume of ice between two passages depends not only on atmospheric heat losses but also on how cold the ice blocks, which are pressed beneath the water surface, are since these blocks act as heat sinks. The supercooled ice blocks are heated from the surrounding water which freezes to the blocks. The presence of ice slush and snow in the water do also accelerate the nucleation of ice in the water.

Figure 6. The accumulated distribution of observed blockizes, some typical examples.

![Figure 6](image)

Figure 7. Ice observations in the two tracks showing the static growth and the growth of brash ice, $h_{ekw}$. The calculated results using the degree-day method and the Bêrenger-Michel algorithm are presented.

FORECASTING THE GROWTH OF BRASH ICE

The growth of brash ice in the tracks can be described by the degree-day method, although in a modified form. All ice is assumed to be produced in the track over a width equal to the broken width, BEAM. After each passage the ice is broken and there is free water at the surface. After $k$ number of passages each with its sum of negative degree-days since last passage, $\Delta S_k$, and the knowledge of the ice thickness in the track before the first passage, $h_{\text{start}}$, the calculated brash ice thickness, $h_{\text{calc}}$, in the tracks can be estimated.
\[ h_{\text{calc}} = h_{\text{start}} + \sum_{k} \text{ALFA} \sqrt{\Delta S_k} \]

where ALFA is an empirical coefficient depending on the usual degree-day coefficient and determined from the static ice growth and factors occurring in a broken track.

This method was used for calculating the growth of ice in the tracks and the results were compared with the observed values of \( h_{\text{ekv}} \). Best curve fitting was obtained using \( \text{ALFA} = 1.2 \text{ cm/}(^\circ \text{C-d} \text{ays})^{\frac{1}{2}} \) in both tracks. The calculated results are plotted together with the observed ice thicknesses in fig. 7.

The results from using the degree-day method can be compared with the results from using the algorithm presented by Bérenger-Michel [2], Bengtsson [1]. Using a degree-day coefficient of \( \alpha = 2 \text{ cm/}(^\circ \text{C-d} \text{ays})^{\frac{1}{2}} \) and a water content in the brash ice of 25% the best agreement between the results were obtained.

A water content of 25% is to be compared to the results from the observations, see fig. 5.

Using the degree-day method the growth of brash ice in the long dredged fairway into the Luleå Harbour can be calculated. Under normal winter conditions with about 200 passages we get an equivalent brash ice thickness, \( h_{\text{ekv}} = 5.8 \text{ metres} \) which is fairly close to previously observed brash ice thickness in Luleå, Sandkvist [5].

ACKNOWLEDGEMENTS

I would like to thank the Harbour Authority of Luleå and the crew of the harbour ice breaker VALKYRIA for all their help and assistance in performing the field work.

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DYNAMIC ICE LOADS AND STRESS ANALYSIS ON THE PROPELLER OF THE ARCTIC SHIP; MODEL TEST IN ICE

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H. Kawakami Do.
F. Yamamoto Do.

ABSTRACT

Propeller of Arctic ships have often brought about great or small failures because of very serious ice loads as impact and milling loads in ice navigation. To establish the strength design of propellers in consideration of the ice load, the following studies were made in collaboration with Japan Marine Machinery Development Association:

the first stage: Propeller-ice impact and milling model tests making use of saline ice were made, and dynamic stress, torque and thrust of the propeller were grasped.

the second stage: Strength tests of big-diameter model propeller statically applied with the above ice load were made and a detailed stress level by the FEM analysis were grasped.

These researches were made on a Fixed Pitch Propeller (FPP) of built-up type and a Controllable Pitch Propeller (CPP). By the studies, the detailed relationship between the dynamic ice load and the stress level produced on each member of FPP or CPP has become clear.

1. Introduction

Generally, propellers of Arctic ships are repeatedly subject to extremely a severe ice load, the many actual failure of propellers have been reported. These history have shown that propellers constitute one of the most fragile components of these ships. As an up-to-date precedent, cited here are propeller failures in a USCG icebreaker, "Polar Star" and CCG icebreaker, "B. Franklin". In ice tranitting merchant ships to be thought capable of independent navigation in remote high arctic regions, failures in their propellers are shafting systems not only give serious influence upon subsequent navigations but also become a very dangerous matter from the viewpoint of the safety of the crew and secondary damages caused by leak of loading cargoes; and accordingly, it has become one of
very important problems to establish a design of propellers safe from damage.
In the studies on the strength design of icebreaker propellers, analytical theories, full scale and model experiments have been made by Messrs.
V. Jagdakin [1], M.A. Ignatiew [2], E. Enkvist [3], R.Y. Edwards [4], etc. long since the beginning of 1960s and have developed to some extent, but there seemed something yet to be studies. In addition, the Arctic Shipping Pollution Prevention Regulations (ASPPR) proclaimed in 1972 is the most concrete regulations at present basing upon the long experience in the ice sea navigation, but this is no more than the regulations of comparatively a low ice load level which should be satisfied with; accordingly, the high powered arctic propeller designed in such regulations cannot be said of the existence of inspiring us with confidence as it is.
So it is very important to study the arctic propeller strength more quantitatively.

For this purpose KHI carried out the following researches by use of FPP and CPP model; namely, as the first step, experimental study of dynamic ice impact and milling load, and as the second step, static load test and FEM analysis of big propeller models.

Both propellers are designed for polar arctic merchant vessel having about 60,000 SHP/shaft and these configurations and blade thickness are decided in consideration of estimated dragging ice load and cavitation criteria. Blade contours are shown in Fig. 1.

2 Ice Impact Load and Milling Load

2-1 Ice Impact Load:

Numerous sizes of ice fragments broken by the bow part of ice transmitting ship flow through the ship side or bottom into the propeller. Comparatively a small fragment is accelerated and fractured by the propeller, but the propeller is subject to an impact load as its reaction. Propeller failures which seem resulting from such phenomena have been reported in numbers.
Since the magnitude of the impact load is more than difficult to be assumed because of it relying not only upon the propeller geometry, working condition, ice thicknesses but also upon such unstable factors as the size and shape of ice fragments, ice contact condition, local fracture of ice, etc. It is, however, very important to acquire a quantitative grasp in the strength design of propellers as well as in case of the milling load which is stated later. For the purpose, model impact tests were made to measure the dynamic blade stress, shaft torque, and axial force of the propeller and assume the full scale load applied to the actual ships propeller by the simple impact theory. The FPP and CPP were operated under the various propeller revolution number in the water tank and given collision of ices having various sizes and strengths by a shooter, and blade stresses shaft torque and thrust were measured.
Model propellers used are 320 mm dia, NiAlBr make FPP and CPP. Saline ice and paraffin wax having the compressive strength 27.5 kg/cm² to 1.5 kg/cm² are applied.
The geometry is a rectangular prism having 75 mmsq x 85 mm thickness - 500 mmsq x 200 mm thickness.

The compressive strength of the arctic ice has been said to be about 29 kg/cm² - 42 kg/cm², but its average value is adopted here as 36 kg/cm², and 1.5 kg/cm² which is reduced from the average value of arctic ice by the scale ratio = 25 in accordance with the ice similarity law was made the lower limit of the model ice. Freezing tests were made to produce homogeneous saline ice with various strength by applying various kinds of additives and it was found that the ice to meet the required conditions can be produced by a mixed water solution of NaCl and CMC (Calboxy Methyl Cellulose).

An experiment was made using various kinds of ice under the Bollard Pull Condition where the propeller is fixed and only the number of revolutions of the propeller changed.

Shown in Fig. 2 as an example is the time history of the blade stress, torque, and thrust when a paraffin of 50 mmsq x 200 mm (corresponding to 12.5 mmsq x 5 m for the full scale propeller Dp = 8.0 m) is given impact to the FPP.

Comparatively small fragments will be sent flying at one blow simultaneously with the contact with the tip of the propeller. However such a big ice will ride over the propeller after the impact and carried backward as it is being cut. These phenomena are similar to the milling mentioned later and big and sharp peaks are occurred. Such a sharp impulse torque is similar to that measured by icebreaker "Fuji" of Japan [5].

It is now an important problem how experimental values obtained from model test can be converted into that of the full scale propeller. However there is no theory to explain fully this three dimensional collision problem of ice having an elastic - plastic property as forementioned. So this problem was replaced with a nonelastic collision against extremely a simplified spring system shown in Fig. 3-1 and a conversion to the full scale value was made making use of the property of this solution.

If an ice block having a mass m and a coefficient of restitution e = 0 impact a propeller having a mass M and a spring constant k at a speed v, the maximum impact force applied to the propeller will be

\[ F_{max} = \sqrt{\frac{k}{M+m}} \cdot mv \]  

(1)

Supposing that not only the geometries of the propeller and ice but also the posture and position at the time of collision is quite similar between that in the model test and the full scale test, the relationship between the model and the full scale in the strain, force and moment produced on the propeller will be

\[ \frac{\varepsilon_s}{\varepsilon_m} = \lambda \frac{n_s}{n_m} \sqrt{\frac{E_s}{E_m}} \]
Complying with the impact theory, the full scale additional ice impact torque $Q_1$ converted from model test values is shown in Fig. 3-2. The number of revolution of the full scale propeller is supposed to be in proportion to $\sqrt[4]{N_{R}}$ of model propeller revolution number.

$\frac{F_s}{F_m} = \lambda^3 \cdot \frac{\gamma_s}{\gamma_m} \sqrt{\frac{E_s}{E_m}}$

$\frac{Q_s}{Q_m} = \lambda^4 \cdot \frac{\gamma_s}{\gamma_m} \sqrt{\frac{E_s}{E_m}}$

where, $D_p$: propeller diameter $\lambda$: scale ratio ($D_p/D_p$)
$E$: strain, $F$: force, $Q$: torque,
$\gamma$: revolution number of propeller $E$: rigidity of propeller blade
suffix (s: full scale, m: model)

This is the result under the condition which comparatively small ice fragment of 150 mmsq x 85 mm thick (2.75 m sq. x 2.1 m thick in the actual one) having four kinds of strengths from 1.5 kg/cm$^2$ to 27.3 kg/cm$^2$ were applied two kinds of propellers having wide and thick blades and moderately wide and thin ones were used. Each mark shows the maximum value obtained by about twenty times trial under the same condition. The following conclusion is obtained.

(1) As presumable in Eq. (1), the impact torque looks proportional to the number of revolutions of the propeller.

(2) In case of the thick-bladed propeller, the impulse torque seems to be not affected very much by the property and strength of the ice.

In case of the thin-bladed propeller, the torque caused by an ice of 1.5 kg/cm$^2$ appeared extremely low. But the other torques of hard ice is seen about 1.5 - 2 times bigger than that of thick blades and changed to be 500 t-m - 1000 t-m along with the increase in the number of revolutions. The same tendency with it is true of the blade stress.

(3) In the Fig., the hydrodynamic bollard pull torque curve ($Q_{\text{hydro}}$) and the curves of 2, and 3 times thereof are shown in the same Fig. 3-2. According to it, the followings are obtained.
\[ Q_1 = (1.5 - 2.8) \times Q_{\text{hydro}}, \text{for thick-bladed propeller} \]

\[ Q_1 = (1.5 - 4) \times Q_{\text{hydro}}, \text{for thin-bladed propeller} \]

According to a few actual measurement of icebreakers,

\[ Q_1 = 1.5 \times Q_{\text{hydro}} \text{ for Antarctic Icebreaker, "Fuji"} \ [5] \]

\[ Q_1 = (3 - 4) \times Q_{\text{hydro}} \text{ for USSR Icebreaker by Mr. M.A. Ignatiew} \ [2] \]

were observed and these load magnification factor seem to be the same order to the our measured values.

(4) There is a tendency of the impact torque becoming bigger because the impact direction is nearly the same as the revolving direction of the propeller. To have this reduced to certain extent, it is necessary to deteriorate the impact conditions as much as possible. For this purpose, it is essential to decide the number of revolutions, propeller diameter, contour, expanded area ratio of the propeller, blade thickness around the leading and trailing edges, etc. from above viewpoint.

2-2 Milling test

When ships transit thick ice and ridge, larger fragment of ice will be forced under the ship and may pass through portion of the propeller disc. These ice pieces may be massive enough that rather than accelerated out of the propeller disc as above-mentioned impact test, they are milled by the propeller blade with very large instantaneous torque. And furthermore if the milling torque may exceed the allowable torque and the engine stalls, the propeller may be dragged through the ice. Many arctic propellers have been subject to serious damage by milling and dragging load. By the use of test apparatus various kinds of tests were made to estimate these load, by changing propeller-ice interaction condition, namely propeller revolution number of velocity, propeller-ice cutting depth and ice strength. Models of FPP and CPP used for the tests are the same as those of impact test, and the size of ice and paraffin test pieces is 500 x 500 x 200 mm; and compressive strength is 12.5 kg/mm\(^2\) - 1.5 kg/mm\(^2\).

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![Fig. 4 Ice milling record](image)

![Fig. 5 Blade stress distribution](image)
An example of CPP milling test results are shown in Fig. 4. F1 - 7 and B1 - B3 show time history curves of blade stress at r/R = 0.475 on blade face and back side respectively. In this case, ice comes onto blade back side and so compressive stress peak occurs on face side and tension stress peak occurs on back side at a rate of one peak per one revolution. Time histories of propeller torque, thrust, shaft bending stress and etc. are also measured under various propeller-ice interaction condition.

Fig. 5 shows blade stress distribution of FPP under 75% dragging condition (propeller speed \( \neq 0 \), revolution No. = 0), by changing angular blade position. (for actual prop. D = 8.0 m, ice strength 25 kg/cm²)

This is one of the most severest condition which sometimes occurs at transient ramming condition in case of ship installed FPP. Stress peak level varies with angular blade position largely, even if ice cutting depth is the same, and so it is considered that there is some possibility for FPP that local stress level near tip exceeds the yielding stress value of propeller material.

The non-dimensional maximum blade bending moment obtained from the milling test of the CPP is plotted in Fig. 6, along with the angle of ice incidence as an typical example. The parameters are ice strength of ice, and ice cutting depth.

The ship's speed at continuous ice-breaking operation of the Polar Arctic Vessel was supposed to be about 3 knots - 6 knots, number of revolution of the propeller being about 110 rpm.

If milling occurs under such condition, the propeller, the propeller will work with \( \chi \) around "M" of the Fig. 6. On the other hand, if the engine stalls due to large ice torque, the number of revolutions of the propeller will come down to zero, i.e. dragging condition, and fall into \( \chi \) around "D" of the Fig. 6. As a result, the following matters are made clear.

(1) The experimental value of the dimensionless blade bending moment is scattered comparatively wide, and a tendency of rough sine curve is observed.

(2) Under the milling condition, the non-dimensional value comes to be around 0 - 1.0 about 0.5 in average. On the other hand, under the dragging condition, it comes to be 1.0 - 1.75, being around 1.4 in average. The value 1.4 of the latter comes near 1.5 which is the penetration coefficient Nakajima and others [6] have obtained on pillars, etc.

(3) The value of the bending moment when dragging remarkably increases to about 3 times the value of the milling time.

The fact that it has often been reported that the propeller failure of icebreakers occurs very much at the time the propeller transiently stops is concerned with the above descriptions.

In this connection, it is necessary to give the engine torque enough allowance and allow the propeller not to stop even a moment irrespective of any magnitude of ice load. It therefore comes to understand that the FPP where the propeller must be stopped under an astern transient state is disadvantageous in such point; and that, on the contrary, the CPP where it kept revolving in the same direction is very effective.
3. Static Load Test and FEM Analysis

In order to examine the detail of stress level on the propeller blade and blade changing inner mechanism of CPP under the above mentioned ice load, static load tests were made using 1.1 m diameter FPP and CPP models. These model propellers are similar to 320 mm diameter model propellers and especially CPP's inner mechanism for blade angle changing is completely similar to the one designed for actual Arctic propeller.

Measuring positions of the strain gage for the FPP and CPP are propeller blades, blade flanges, bolts, and hubs with 3 filament gages pasted at 150 positions in total.

For the CPP inner mechanism, 2 filament gages and concentration gages were pasted at 27 positions in total on the blade, trunnions, crank ring, joint bar and caps.

Static load testing machine was made for this measurement. This machine is length, 3.6 meters breadth, 3.6 m, height, 2.56 m. Propeller axis is fixed vertically on the loading table and the equivalent ice load is (max. 20 tons) added at the load point on the back or face side of each blade by the hydraulic jack.

Centrifugal forces (max. 10 tons) is also added. Only concentrated load was applied to blades in this experiments and effect of distributed load was investigated by FEM analysis. Positions of concentrated load were chosen widely on the blade is consideration of deep ice immersion depth (25 % - 100 % of blade length) and eccentric ice load occurred according to the angle of blade rotation as shown in Fig. 5.

Applicability of FEM to conventional propellers has been recognized by many calculation results. (for example [7]) But in arctic propellers there is several problems as yet, namely the method of model mesh subdivision due to extremely thick and wide blade and calculating accuracy for a complicated mechanism like CPP. For this purpose the following calculations were done step by step because of complexity of inner mechanism; (1) analysis of blade (2) analysis of hub, (3) analysis of bolt (4) analysis of trunnion and (5) analysis of crank ring. NASTRAN'S structure analysis program is applied to this calculation and 3 dimensional isoparametric solid elements and spring elements are used. Mesh subdivision of blade, flange trunnion and hub are shown in Fig. 7.

Fig. 7
FEM Mesh subdivision

Fig. 8 Blade stress distribution
(Experiment/FEM)
Fig. 8 shows calculated and measured blade stress distributions on face side of CPP and FPP. Concentrated load is normally added at $r/R = 0.76$ near midchord on the back side of the blade. It was found that FEM values agree very well with experimental ones even for such wide and thick blades of the arctic propeller.

As abovementioned, magnitude of ice varies according to blade rotation angle and stress distributions of each bolt also depend largely upon this. So optimization of size, number and arrangement of bolts under various load conditions is very important problem.

As abovementioned, magnitude of ice varies according to blade rotation angle and stress distributions of each bolt also depend largely upon this. So optimization of size, number and arrangement of bolts under various load conditions is very important problem.

Fig. 9 shows blade stress distribution by FEM corresponding to full scale under the condition that CPP blade is dragged in the 25% ice immersion depth and that blade rotation angle is $\theta = -30^\circ$, 0$^\circ$ and 30$^\circ$.

Ice crashing pressure of 41.63 kg/cm$^2$ is added on the each ice immersion area. These calculating results clearly show relationships between stress distribution and local ice distributed load in company with change of blade rotation angle. maximum stress occurs near ice immersion at blade angle $\theta = 30^\circ$ because blade in the aft part is generally thin and so consideration of local blade thickness is very important.

Fig. 10 shows FEM calculation results about static blade stress of FPP in case a concentration load is applied to the fore end or aft end of the propeller as in case of the ice impact load. The estimated concentrated load 195 tons was added at $r/R = 0.9$, corresponding to the full scale torque 700 t-m.

(1) Fig. 10-1 shows the face stress distribution generated by the impact to the fore edge but only comparatively low compressed stresses of 4 - 2 kg/cm$^2$ were produced around the loading point. However, comparatively a high stresses of 9 - 3 kg/cm$^2$ were calculated in certain area near trailing edge by the impact to the aft end seems to be subjected when propeller revolving astern.

(2) Even such a big load as 700 t-m of added torque should work, the load is applied in the propeller rotating direction having big rigidity in the blade; therefore, big stress will not occur all over the blade surfaces. But, comparatively a big stress may take place near the loading point of the fore and aft ends of the blade because of not being thick. As well as the explanation of Fig. 9 the blade thickness of the fore and aft ends of
the blade must be made thick enough.

4. Conclusion

From the above results, the following matters have come to be clear:

(1) Like high Arctic class propellers, the relationship between ice load and stress relative to extremely a thick and wide propeller was grasped by experiments.

(2) The level and its order of the stress of flanges, bolts, and further of the inner mechanism of the CPP were made clear.

(3) It was made sure that the FEM is most effective method for the propeller strength design of not only as FPP but also as CPP having very complex mechanism.

References

4. Edwards, R.Y. "Method for predicting forces encountered by propellers during interactions with ice, 2nd Lips Symposium".

Appendix Photos of testing apparatus

Photo 1: Model CPP (320 mm dia.)
Photo 2: Milling test of CPP

Photo 3: Ice after milling test

Photo 4: Static load test of CPP (1.1 m dia.)
AN EXPERIMENTAL INVESTIGATION OF TWO CANDIDATE PROPELLER DESIGNS FOR ICE CAPABLE VESSELS

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Abstract

The objective of this study was to examine the performance of two candidate propeller designs with respect to their hydrodynamic characteristics and ice milling capability.

Two model propellers, one with ogival and the other with lenticular blade section, were designed for the wing shafts of a triple screw ice-going vessel of 124 m in length. Hydrodynamic performance in open-water was determined in the ship model basin of the Nagasaki Experimental Tank and cavitation performance in the cavitation tunnel. Special emphasis was placed on bollard conditions. Ice milling tests were carried out in a specially designed ice milling lathe employing patented synthetic ice, at the Kanata laboratory of ARCTEC CANADA LIMITED.

The open-water performance of the lenticular propeller was found to be better than that of the ogival propeller, although there was some deterioration in the cavitation performance in the 'behind ship' condition. In the ice milling tests, the peak blade bending moment during passage through model ice was found to be lower for the lenticular propeller.

Model experiment results have been used to demonstrate and compare the performance in both water and ice of two candidate propeller blade designs. The results are useful in assisting with the selection of propeller type and material for ice-going vessels.

1. Introduction

The propellers of ships operating in Antarctic and Arctic regions frequently encounter fragments of ice. Damage to the propellers or the propeller shafts of ice transiting ships is experienced when the ice loading is severe.
Two factors are important in the design of propellers for ice-going vessels from the hydrodynamic point of view, namely high bollard thrust in the ramming condition and reasonable efficiency in the cruising condition. However, since ice loads on propeller blades are several times more severe than hydrodynamic loads, it is common for the blade geometry to be dictated by the requirement to provide adequate strength against these ice loads. Thus, the selection of the blade section, in conjunction with the propeller material, is one of the key factors in obtaining a balanced propeller design. Since backing characteristics of an ice-capable ship are important, the performance of a propeller when turning astern is as important as when turning ahead, and blade sections which are symmetrical about midchord such as the ogival and lenticular sections are widely used.

In the study reported herein, two candidate propellers designed for a new Japanese Antarctic icebreaking research vessel underwent systematic experimental investigation of the effect of these two blade section shapes on ice loads and hydrodynamic characteristics. The results highlighted specific performance features of both ogival and lenticular blade sections. This data will be useful in the design of propellers for icebreakers and ice-going vessels both with respect to the arrangement of propellers and shafts and also the selection of materials for blades and shafts.

2. Design Condition and Design Procedure

2.1 Design Condition

The two candidate propellers were designed for the new Antarctic research vessel which will replace the "FUJI" operating in Lutzow-Holm Bay. The principal dimensions of the ship are shown in Table 1. This ship has to have the capability of continuous icebreaking in level ice of 1.5 m thickness at 3 knots [1].

Table 1 - Principal Dimensions of the Ship

<table>
<thead>
<tr>
<th>Hull</th>
<th>Length Between Perpendiculars</th>
<th>124.0 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Waterline Breadth</td>
<td>27.0 m</td>
</tr>
<tr>
<td></td>
<td>Depth Moulded</td>
<td>14.5 m</td>
</tr>
<tr>
<td></td>
<td>Designed Draught</td>
<td>abt. 9.2 m</td>
</tr>
<tr>
<td></td>
<td>Displacement Moulded</td>
<td>abt. 17,000 tonnes</td>
</tr>
</tbody>
</table>

| Propulsion | Electric Motor Output | abt. 7.5 Mw per shaft x 3 |

<table>
<thead>
<tr>
<th>Speed</th>
<th>Maximum Speed</th>
<th>abt. 19 knots</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cruising Speed</td>
<td>abt. 15 knots</td>
</tr>
</tbody>
</table>
2.2 Design Procedure

Both propellers were designed following the procedure described in Figure 1, after Ignatev [2]. Ship resistance during continuous icebreaking was estimated to be about 300 tonnes. To estimate open-water characteristics, the Wageningen B-4 series chart [3] and Ignatev's design chart [2] were utilized. Blade thickness was chosen based on the propeller ice milling torque estimated by Jagodkin's formula [4], applying Ignatev's formula to estimate blade bending moment due to ice milling and blade section coefficient.

Figure 1 - Flow Chart of Propeller Design for Ice-Going Vessels
The strength of the Antarctic ice in Lutzow-Holm Bay was assumed to have the following values, based on published data [5].

Crushing strength: \( \sigma_c = 2550 \text{ kPa} \)

Shearing strength: \( \tau = 640 \text{ kPa} \)

The material for the propellers was chosen to be a special 13 Cr stainless steel (known commercially as MSS [6]), which is the same type of stainless steel used for "FUJI". The mechanical properties of this material are:

Tensile strength: \( \sigma_t = 8.34 \times 10^5 \text{ kPa} \)

0.2% proof stress: \( \sigma_y = 5.40 \times 10^5 \text{ kPa} \)

Elongation: \( \varepsilon = 0.15 \)

The principal particulars of the propellers as designed are shown in Table 2.

Table 2 - Principal Particulars of Propellers

<table>
<thead>
<tr>
<th>PROPELLER DIAMETER</th>
<th>PROPELLER A</th>
<th>PROPELLER B</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.90 m</td>
<td>0.86</td>
<td>0.87</td>
</tr>
<tr>
<td>PITCH RATIO (CONSTANT)</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>EXPANDED AREA RATIO</td>
<td>0.306</td>
<td>0.306</td>
</tr>
<tr>
<td>BOSS RATIO</td>
<td>0.5594</td>
<td>0.0613</td>
</tr>
<tr>
<td>THICKNESS-CHORD LENGTH RATIO AT 0.7 RADIUS</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>NUMBER OF BLADES</td>
<td>4</td>
<td>LENTICULAR</td>
</tr>
<tr>
<td>BLADE SECTION</td>
<td>OGIVAL</td>
<td>BUILT-UP</td>
</tr>
<tr>
<td>TYPE OF PROPELLER</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3. Apparatus for Model Tests

To examine the characteristics of both propellers, model tests were conducted. Hydrodynamic characteristics were explored in a ship towing tank and 500 mm square cavitation tunnel at the Nagasaki Experimental Tank of Mitsubishi Heavy Industries Limited in Japan, and ice milling characteristics were studied using a special lathe and patented synthetic ice at Arctec Canada Limited in Canada. The model propellers were manufactured in a special aluminum alloy selected to withstand ice loads and cavitation. The diameter was chosen to be 250 mm, in consideration of the capacity and operating convenience of the testing apparatus. Figure 2 shows the model propellers and typical blade sections.
4. Hydrodynamic Characteristics of the Candidate Propellers

4.1 Open-Water Characteristics

Propeller open-water tests were first conducted in the towing tank. Thrust and torque were measured changing advance coefficient. Non-dimensionalized thrust and torque and propeller efficiency are shown in Figure 3, where the non-dimensional coefficients are defined as follows:

\[ K_T = \frac{T}{\rho n^2 D^4} \] : Thrust coefficient
\[ K_Q = \frac{Q}{\rho n^2 D^4} \] : Torque coefficient
\[ \eta_p = \frac{J.K_T}{(2\pi.K_Q)} \] : Propeller efficiency
\[ J = \frac{V}{(nD)} \] : Advance coefficient

Where:  
\( T \) : Thrust,  \( Q \) : Torque,  \( D \) : Diameter
\( n \) : Revolutions of propeller,  \( \rho \) : Density of water
\( V \) : Advance speed of propeller

From these results, the ship speed at maximum motor power output (22 MW), power for cruising speed (15 knots) and bollard thrust were calculated and the results are shown in Table 3. Although the bollard thrust is almost the same for both propellers, propeller B gave better performance in the 'free running' condition.
4.2 Cavitation Characteristics

The cavitation characteristics of both propellers were examined both in uniform flow and non-uniform flow. As a result, thrust breakdown is not expected to occur with either propeller, so far as the effect of propeller cavitation on ship performance in open water is concerned. At bollard condition, the cavitation effect on thrust is small for both propellers.
In the case of wing propellers, where the flow distribution at the propeller is relatively uniform being affected mainly by the wake of the bossing and oblique flow along the hull, the difference in cavitation patterns between the propellers is small. On the other hand, the flow distribution for a center propeller is so affected by the wake of the ship hull that cavitation occurs which can cause cavitation erosion and hull vibration. The propellers were therefore tested in the wake estimated for the subject ship. Figure 4 is the comparison of cavitation patterns at the maximum motor output and bollard conditions. Judging from these cavitation patterns, at the maximum motor output, propeller B will suffer damage due to unsteady cavitation, while propeller A will be free from cavitation damage. At the bollard condition, cavitation on the blades is stable and no damage should occur to either propeller, but since the cavity is larger for propeller B it is more likely to experience thrust breakdown than propeller A. Thus it can be said that propeller A, with the ogival section, is preferable from the viewpoint of cavitation performance.

Figure 4 - Comparison of Cavitation Patterns in Non-Uniform Flow
5. Ice Milling Characteristics of Candidate Propellers

5.1 Test Parameters

The following measurements were made during testing on the special ice milling lathe [7]. In Figure 5 Propeller B is shown in the lathe operating at a 320° advance angle.

- Blade bending moment (BM) - Strain gauges on the root of the blade
- Thrust (T) and Torque (Q) - Strain gauges on the propeller shaft
- Radial Force (F) - Force block at the radial bearing
- Ice Speed (Vs) and Revolutions of Propeller (n).

All the gauges were calibrated before and/or between test runs by applying known axial and radial forces at three points on the blade.

The advance angle of the propeller blade to the ice ($\alpha_v$) is defined in Figure 6 where $R_p$ is the radius to the center of the 'immersed' portion of the blade during milling. The test apparatus permitted adjustment of $\alpha_v$ from 0 to 360 degrees by various combinations of the ice speed and propeller turning speed.

Figure 5 - Propeller Milling Lathe
5.2 Strength of Synthetic Ice

The patented synthetic ice used in these tests was prepared by Arctec Canada Limited. The synthetic ice was prepared to a strength appropriate to the model scale [8]. Since the scaling ratio \( \lambda = 19.6 \), the estimated ice strength at model scale was as follows.

\[
\sigma_{c}(\text{model}) = \lambda^{-1} \sigma_{c}(\text{ship}) = 130 \text{ kPa}
\]

To check the actual strength of the synthetic ice, crushing and bending tests were conducted during the tests by using test samples taken from the same block of ice as was used in the ice milling tests. Results are shown in Table 4. There is considerable scatter in the data, but it can be said that the average value of the crushing strength is close to that targeted.

<table>
<thead>
<tr>
<th>Table 4 - Mechanical Strength of Synthetic Ice</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CRUSHING STRENGTH</strong> (( \sigma_c ) : kPa)</td>
</tr>
<tr>
<td>95.6 \pm 23.5 at 0.03 m/s (11)</td>
</tr>
<tr>
<td><strong>FLEXURAL STRENGTH</strong> (( \sigma_f ) : kPa)</td>
</tr>
<tr>
<td>* Number of test pieces</td>
</tr>
</tbody>
</table>

NOTE: UPPER BLADE VIEWED FROM ABOVE PROPELLER
To establish a correlation between the ice milling data from tests in synthetic ice with ice milling data in saline ice, indentation tests were carried out by pushing a flat plate into the synthetic ice block. The indentation condition corresponds to the ice milling test at an advance angle of 90 degrees. The results are shown in Figure 7 in comparison with existing data for piles [9]. In view of the differences in the shape of the test pieces and the indentation configuration (the piles were supported above and below the indenting ice whereas in the current case the lower end of the plate is within the ice and is thus supported as a cantilever) the present results in synthetic ice correlate satisfactorily with those in saline ice.

Figure 7 - Indentation Test Data

5.3 Ice Milling Test Results

Figure 8 shows the blade bending moment, propeller thrust and propeller torque divided by the crushing strength of ice (namely $BM/\sigma_c$, $T/\sigma_c$, and $Q/\sigma_c$) versus advance angle $\alpha_v$ for 33% cutting depth of the blade for both propellers. The data points on which these curves are based exhibited some scatter due to the local differences in ice crushing strength. However, reference to the data presented in [7] permitted construction of the full curves shown to the extent necessary to perform a comparison of the two propellers.

For ice-going vessels the ice milling condition is encountered most frequently at $\alpha_v = 0 - 40$ degrees (ahead condition) and $170 - 210$ degrees (astern condition). In the case of the ahead condition, both propellers show almost the same ice milling characteristics, since the design point is taken in this region. In the astern condition, propeller B shows less ice load on blades. Over the other advance angles
where very severe ice milling can occur, propeller B experiences less ice load. Thus it can be said that propeller B is preferable from the viewpoint of ice loading on the propeller and shaft.

6. Concluding Remarks

Evaluation of two candidate propellers designed for the new Antarctic icebreaker research vessel of Japan was conducted both from hydrodynamic and ice load points of view. Two model propellers, one with an ogival section (propeller A), and the other with a lenticular blade section (propeller B), were tested in both towing tank and cavitation tunnel, and also on a special lathe designed and built for conducting ice milling tests.
The relative performance of the propellers in major performance parameters is summarized below where the superior propeller is identified.

- Efficiency (open sea characteristics): Propeller B
- Bollard Thrust: No difference
- Deterioration of thrust due to cavitation: Propeller A
- Cavitation in non-uniform flow: Propeller A
- Ice milling capability: Propeller B

These results provide assistance in selecting the geometry of propellers for ice-going vessels. In the case of the triple screw icebreaker, for example, since wing propellers have a greater probability of interacting with ice blocks but are relatively free from cavitation problems, propellers with lenticular sections would be preferable. On the other hand, for the center propeller where cavitation problems may be severe, the ogival section will be better.

**Acknowledgements**

The authors wish to express their appreciation to Mr. R.Y. Edwards, Jr., former president of Arctec Canada Limited and now president of Offshore Technology Corporation, for his guidance in conducting ice milling tests, and to Mr. K. Takekuma, Project Manager, and Dr. Y. Kayo, of Nagasaki Experimental Tank of Mitsubishi Heavy Industries, Limited for their useful discussion in preparing this paper. They also thank all the staff of Arctec Canada Limited and Nagasaki Experimental Tank who contributed to the project. Finally the authors wish to acknowledge Transport Canada who contributed to the original development of the ice milling lathe.

**References**


ENGINEERING FOR VESSEL ICE ACCRETION WITH PARTICULAR REFERENCE TO THE ALASKAN FISHING FLEET

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ABSTRACT

Ice accretion on fishing vessels in northern waters has been a persistent long-term problem, contributing to vessel damage, loss of productive time, and occasionally loss of vessels and life. Past research (which has been sporadic, widely dispersed, and often in response to disasters) has pointed out several useful paths for engineering solutions. Key strategies suggested by these investigations include increased education of vessel operators about the general nature of the problem, understanding the meteorological conditions required for ice accretion, the physical mechanism of drop deposition and ice growth, ship design modifications, active and passive anti-icing and deicing measures, and well-publicized systems of forecasting icing conditions and advising evasive action.

Existing methods dealing with ice accretion were examined, with emphasis on adapting them to the Alaskan fishing fleet. Relevant design criteria include adaptability to retrofitting, semiautomatic operation usable in an emergency, and usability in dangerous situations. Methods that appear to hold promise include pneumatic inflatable membranes, thermosiphons using engine waste heat for masts and railings, high-pressure seawater jets, and methods of surface vibration. The best engineering strategy will likely be a combination of improved vessel design, judicious use of weather forecasting, good seamanship and use of active deicing devices.
INTRODUCTION

With increased use of Alaska's northern seas, vessel icing is becoming a great concern to the fishing and maritime industries. Expansion of the territorial fishing limits and increased petroleum development has greatly increased vessel traffic. High capital investment increases pressure to operate in the winter months when there is a high potential for vessel icing.

The danger and cost of vessel icing may be lowered through several means. Better understanding of the icing processes leads to improved techniques for avoidance, prevention and removal. Improved forecasting techniques greatly enhance a vessel captain's opportunity to take timely and useful evasive measures. Finally, development of economical and well-engineered equipment for icing prevention and removal will allow vessel owners to operate in northern waters with more confidence and with a longer season.

This paper presents the results of a study sponsored by the Alaska Sea Grant Program (1) to assess the current state of knowledge of vessel icing processes, forecasting, prevention and removal. The primary goal was to identify engineered devices that could be implemented on typical vessels of the Alaskan fishing fleet.

Superstructure icing has been a problem in polar waters throughout history. The English trawlers Lorella and Roderigo went down off Iceland with 40 deaths; Iceland lost the trawler Juli off Newfoundland with all hands; a Canadian vessel went down about the same time. The trawlers Ross Cleveland and Kingston Peridot went down off Iceland in the 60s, and the Romulus sank in the North Sea. Forty lost their lives when three Canadian vessels iced and sank. The list goes on.

Winter fisheries in the North Pacific are in a developmental stage, but the same story is developing. The 26-m trawler John & Olaf iced up off Alaska's Kodiak Island in 1979. The 4-man crew abandoned ship but didn't survive the storm. In 1980 The Gemini, a 34-m crabber, iced and rolled; 3 of the 5 crewmen survived. Unfortunately, 30 men (the entire crew) were lost when the Lee Wang Zin went down while enroute from Prince Rupert to Japan in December 1979 with a load of iron ore pellets. One hypothesis is that icing reduced its maneuverability.

Icing rates are often reported in descriptive terms. Scales for quantifying the rates vary, but general categories are shown in Table 1. A conversion of ice thickness to tons is stormspecific, but an example described by Stallabrass (3) is indicative: a 620-gross ton stern trawler

<table>
<thead>
<tr>
<th>Catagory Name</th>
<th>Icing rate per 3 hours (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>light</td>
<td>25 to 64</td>
</tr>
<tr>
<td>moderate</td>
<td>64 to 127</td>
</tr>
<tr>
<td>heavy</td>
<td>127 to 188</td>
</tr>
<tr>
<td>very heavy</td>
<td>188 to 318</td>
</tr>
<tr>
<td>extreme</td>
<td>318+</td>
</tr>
</tbody>
</table>

TABLE 1. Icing rate catagories. (Adapted from Wise & Comiskey (2)).

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arrived in port after severe icing was encountered. Maximum ice accumulation on the superstructure was 03 m; test measurements led to a calculated estimate of the total weight, which was just over 27 metric tons. The center of gravity was located 2.84 m above deck. This vessel's calculated ice allowance for stability was 27 metric tons with center of gravity of 3.03 m.

A higher center of gravity is the most serious consequence of a heavy icing episode. Simple buoyancy relationships show that an increasingly unstable configuration develops as this shift becomes greater (Figure 1).

![Figure 1 Stable vs. unstable configurations showing relative locations of center of gravity (g), buoyant force (b) and metacenter (m).](image)

The conditions leading to superstructure icing are principally controlled by air temperature, sea temperature and winds. Dangerous combinations tend to occur more frequently in some localities than others, and maps can be drawn to delineate these areas. Figure 2 shows the most current map for Alaska.

![Figure 2. Icing severity map of the Gulf of Alaska. (Adapted from Wise and Comiskey (2)).](image)

The fishing fleet using these waters consists of relatively small (20 m to 45 m), owner-operated vessels. Recent developments in the Alaskan fishery have occurred during mild icing winters. We can expect an increasing number of unfortunate incidents in the future unless deicing equipment and techniques can be developed that are appropriate to this fleet. This means
that they must not only be effective after long dormancy but these measures must also have a
low initial investment for the small businessman, must be adaptable to movable gear, and must
have low maintenance requirements.

As a prelude to a research and development effort, a state-of-the-art study was carried
out. A standard library search (including computerized bibliographic bases) yielded very few
publications, but personal contact with several researchers in the circumpolar nations netted
more information. This report is both statement of a little-publicized problem and a summary of
accumulated information. Both applied and theoretical aspects of the problem were investigated
to provide a solid background and suggestions for designing practical devices.

APPLIED ASPECTS

Current Methods

Historical methods for dealing with the problem are brute force and running for shelter.
Sledge hammers, mallets, baseball bats, picks and axes are often used to remove ice (3). The
Soviet Union, in dealing with this problem, has developed a 10-point litany for good seamanash
that they have shared through the United Nations Intergovernmental Marine Consultative
Organization:

1. Head for warm water or protected coastal areas.
2. All movable gear should be placed below deck or fastened to the deck as low as
possible.
3. Cargo booms and derricks should be lowered and secured.
4. Deck machinery and boats should be covered.
5. Storm rails should be secured.
6. Gratings and all such possible drainage blocks should be removed.
7. Ship should be made watertight, all hatches closed.
8. If freeboard is high enough, take on ballast in any bottom tanks that are empty.
9. Establish two-way radio contact with a shore station or ship.
10. Set course and speed to minimize spraying.

The risk to these historical methods, however, is that crew are exposed during stormy, high-risk
conditions.

Experimental Methods

Of the many experimental methods that have been developed in the past, nine basic
approaches seem particularly useful.

1. Pneumatic Methods: This approach is a direct extrapolation of aircraft deicing
technology. British researchers at the White Fish Authority (Hull, England) contracted
Palmer Aero Products Ltd. and Decca Radar Ltd. to do trials with a marine modification of Palmer's aircraft product which consists of a tube, or series of tubes, that can be flexed upon demand. Pressurized air is introduced when the tubes are ice-covered; the tubes expand, introducing a tensile stress in the ice, and tube elongation introduces a shearing stress. The combination breaks the ice formation. Trials on the side trawler Boston Phantom were quite successful (5, 6). Unfortunately for this project, Great Britain no longer fishes arctic waters and the trials have been terminated.

Following these efforts, Stallabrass (National Research Council of Canada) ran a successful laboratory experiment with the same type of equipment and it seems to hold great promise (7).

2. Anti-icing Fluids: This concept has a sound basis, but it is difficult to apply. Hickman (6) found that sea spray dilution was problematical and that wind of any degree interfered with placement of sprayed fluid. Lock (4) just dismissed the idea. Minsk (8) mentions an apparently successful field test on a U.S. Coast Guard cutter. Stallabrass reported that ethylene glycol was effective in reducing the strength and adhesion of ice on flat surfaces in a laboratory situation. He noted, however, that slippery residues on decks and possible contamination of catch were disadvantages. Soviet researchers have investigated possible reagents more thoroughly; Semenova (9) tested 11 chemical reagents in the laboratory and recommended 3 for sea trials.

3. Heat (Electric): Decca Radar Ltd. experimented with developing a heated window for scanners but gave up the effort due to cost and design problems (6). Lock (4) regards electric resistive heating as expensive and possibly unsafe, but he suggests that infrared radiation may be worth investigating. Stallabrass (3) concurs that this method is too expensive for general use, but he mentions possible applications such as an electrically conducting layer on bridge windows. Laboratory tests conducted by Dunlop Company Ltd. (10) indicated that this technique may be more useful as an anti-icing device rather than for deicing.

4. Heat (Sea Water): High-pressure jets of seawater are capable of eroding ice cover because of the combined effects of temperature and pressure. References to sea trials indicate that this may be a promising approach (4).

5. Heat (Waste Engine): This technique involves the use of a thermosiphon to transfer heat from below deck to the icing area. It could involve either an open system, allowing air to circulate in hollow structures, or a closed system involving heat exchange using a liquid that would circulate in the hollow structure. This method would best be incorporated into original vessel design (4).

6. Coatings (Ice-Phobic Surfaces): Several researchers have pursued this concept (11, 12, 7, 13). It is attractive because painting a vessel is an ordinary maintenance routine. A compound that could be easily applied would assure success without retrofit or design
modifications. Results of laboratory and sea trials have isolated a small number of compounds that, though they ice up, have a low bonding strength. Questions still arise concerning wear, since fishing vessels receive hard use.

7. Deformable Surfaces: Limited work has been done both in North American and England using a polyethylene bag or a PVC envelope over a radar scanner window. Ideally, the wind flaps the cover and the ice is thrown off. However, this works under only some actual sea conditions (6).

8. Shape Modification: Again, this technique, which involves placing aerodynamically shaped covers over fixed equipment, has limited application. Sea trials with covers for radar scanners demonstrated highly variable effectiveness under different conditions (6).

9. Steel Cable Substitutes: Stallabrass (7) experimented with parallel filament polyethylene rope, finding little difference between it and steel cable for ice removal, though it may have the limited benefit of low surface adhesion. Parafill stays tested by the White Fish Authority (5) also had this property, so that ice pieces would slide down the rope as the lower chunks were removed. However, strength and resistance to abrasion limit the replacement of steel cable.

THEORETICAL ASPECTS

Theoretical investigations have received more attention from the laboratories of the circumpolar nations: the U.S.S.R., Finland, England, Iceland, Canada, the U.S. and Japan, among others. The research has involved two main fields, hydrometeorology and ice physics.

Hydrometeorology

Defining the conditions leading to an icing episode is a necessary prerequisite to developing an adequate forecast/warning system. Factors involved include air and water temperatures, wind, velocity relative to the ship, wind duration, wave development stage, swell, local depth, precipitation and water salinity. Sea spray is the primary source of icing (15). Major factors that determine sea spray conditions are wind, and air and sea temperatures. Borisenkov and Pchelko (16), after four winters of investigations in three different seas, established that minor icing will occur at zero wind and air temperatures 0°C to -3°C, some bow icing will occur at a weak wind (7-10 m/sec) and temperature below -3°C, and heavy icing will occur at a strong wind (15-25 m/sec) and low air temperature (to -15°C). These distinctions are based solely on wind and air temperature. Sea temperature has been incorporated by others as can be seen in the nomograms in Figure 3. The Icelandic approach to icing prevention is heavily reliant upon forecasting based on the air and sea temperature, wind force and wind direction (sea spray)(18).
Ice Physics

Physical studies of the icing process have treated several aspects of the problem. Droplet trajectories are the resultant of drag force (which makes drops follow streamlines) and inertial force (which makes them follow a straight line). Droplet size has greater effect on inertial force, so larger droplets deflect less around an object. Deflection is also affected by size of the obstacle, being small around small or slender objects. As deflection decreases, collection efficiency increases.

Thermodynamic analyses of a droplet in flight establishes that its final temperature is dependent on droplet size, surface water temperature, air temperature, and travel time (19). The thermodynamics at the object surface have been treated by several authors (20, 4). Without going into detail, the result is that droplets frozen on vertical surfaces have a conical form that reflects phase relations (pure water in the small upper portion and increasingly concentrated brine in the larger lower portion) (8).

These thermodynamic and chemical interactions affect ice structure, adhesion, and strength. Golubev (21), during winter field tests in 1969, investigated ice structure by looking at the size and shape of crystals and orientation of the optical axes. He found that the type of contact material, its inclination and ice thickness were also factors. Crystal size increases as ice thickness increases (base crystals are larger on wood and on sheet metal with organosilicon films than on steel or painted ship steel), and they are larger on horizontal surfaces. Optical axes of the ice crystals tend to be normal to painted steel surfaces, and metal surfaces with organosilicon and epoxy coatings. The axes were parallel to wooden or ship steel surfaces. These all have ramifications on ice strength.
Smirnov (22), however, maintains that there is no relation between ice thickness and strength. He investigated hydrometeorological conditions as they affected ice strength, and he isolated air temperature as one of the chief factors. Increased wind intensity also reduces ice strength. Time has a notable effect; "fresh" ice has much less adhesive strength than ice that has been allowed to "age". This is a reflection of thermodynamic bonding (3, 12).

CONCLUSIONS AND RECOMMENDATIONS

Fishing vessel icing in Alaskan waters is an important and dangerous occurrence. A considerable amount of crew time is lost avoiding or recovering from icing each year. Vessels often sustain damage, and loss of vessels and lives is always a possibility.

A study of the available literature supports that idea that an increased understanding of the icing process will lead to improved ship design and operation. Particularly good progress has been made on providing better icing advisory forecasts. Improved engineering measures should lead to economical and effective means to combat an icing episode if it should occur.

Because the Alaskan fishing fleet is largely owner-operated, deicing devices should be inexpensive, adaptable to retrofitting, capable of use often after long dormancy, semiautomatic, and effective. Five type of devices that appear promising and are being considered for further research include adaptation of inflatable membranes, improved high-pressure sea water jetting, mechanical surface vibrations, and waste engine heat supplied thermosiphons.

REFERENCES


DESIGN OF WHARVES
FOR WINTER NAVIGATION
IN THE ST. LAWRENCE RIVER

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President
Jacques Déry & Ass. Inc.
Groupe Lavalin
CANADA

ABSTRACT

Although there are records of winter navigation on the St. Lawrence river dating back to a great number of years, navigation in ice-bound waters throughout the winter season from Montreal to the ocean has been on a regular schedule for the last twenty years only.

In this paper, ice and climate conditions prevailing in the St. Lawrence valley during the winter season will be established, and a short history of wharf design in the area will be given.

Ice problems encountered in wharf design are identified and solutions are given.

The paper will be illustrated with a series of slides on a few pertinent examples of recent wharf designs in the St. Lawrence with appropriate comments.
As implied by the title of this paper, the design of wharves and other structures for winter operations in the St. Lawrence river is the subject under scrutiny.

Fig. 1 is a map of the St. Lawrence River showing principal ports of interest.

We are concerned with the portion of the St. Lawrence river from Montreal to the Atlantic ocean, a distance of 1500 km, not the St. Lawrence Seaway. This appellation applies to the 325 km of the river between Montreal and Lake Ontario upstream and it designates the series of locks, canals and dredged channels inaugurated in 1959 and jointly funded by the U.S. and Canada, mostly by the latter.
It is closed down for 3½ to 4 months every year from December 15th to April 15th on the average - Obviously because of ice conditions.

From Montreal to the ocean there is year-round navigation.

Governing depths at L.W.L. are as follows at:

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montreal</td>
<td>11 m</td>
</tr>
<tr>
<td>Three-Rivers</td>
<td>11 m</td>
</tr>
<tr>
<td>Quebec</td>
<td>12,25 m</td>
</tr>
</tbody>
</table>

The downstream extremity of the 12,25 m dredged channel is located just upstream of Ile-aux-Coudres. Downstream from this Island, natural depth of the channel increases quickly to 30 m for a stretch of 27 km then to a depth of 60 m up to the mouth of the Saguenay river, after which it attains a depth of 185 m up to the Gulf of St. Lawrence where it reaches a depth of 350 m.

One km wide at Montreal and at Quebec, the river widens to 25 km at Tadoussac at the mouth of the Saguenay River, to 65 km at Baie-Comeau, and at Sept-Iles it reaches its greatest width of 150 km which is maintained up to the entrance of the Gulf of St. Lawrence.

Taken as a whole, the St. Lawrence river constitutes a system of navigation which is under certain aspects unparallelled in the world. The main seaports within this system are Montreal, Quebec, Three-Rivers, Sept-Iles and Chicoutimi, on the Saguenay river, all administered by the National Harbours Board. In addition there are several other ports of importance, private and public, distributed along both shores of the river, the most important being Contrecœur, Sorel, Bécancour, Gros Cacouna, Rimouski, Matane on the South Shore and Baie Comeau, Port Cartier and Havre St-Pierre on the North Shore.

The total volume of cargo handled in all those ports is over 125 000 000 tons per year.
CLIMATOLOGY

a) **Temperatures**

The range of temperatures along the river is very wide. In Montreal for example it varies between extremes of 40ºC to -40ºC; variation is much less pronounced in the lower reaches of the river where summer temperatures are cooler.

Michel and Béranger have made the following recurrence statistical study of winter temperature at Mont-Joli near Rimouski, expressed in degree (ºC) days of frost with corresponding maximum thickness of ice in cms.

<table>
<thead>
<tr>
<th>ºC - days</th>
<th>Ice-thickness (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>1 027</td>
</tr>
<tr>
<td>10 years</td>
<td>1 350</td>
</tr>
<tr>
<td>20 years</td>
<td>1 478</td>
</tr>
<tr>
<td>50 years</td>
<td>1 633</td>
</tr>
<tr>
<td>100 years</td>
<td>1 755</td>
</tr>
<tr>
<td></td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>102</td>
</tr>
<tr>
<td></td>
<td>107</td>
</tr>
<tr>
<td></td>
<td>112</td>
</tr>
</tbody>
</table>

b) **Winds**

Winds will affect the docking of ships; they generate waves and they are the prime movers of floating ice over large water surfaces.

As winds in the estuary of the river will cause all of the above, we will therefore give wind data for Mont-Joli which is located half-way down the estuary.

Maximum mean hourly wind speed at Mont-Joli with a predicted return period of 50 years is 116 km per hour.
Is should be pointed out here that prevailing winds in the winter time are from the West and Northwest with the result that most of the time ice is to be found on the South Shore of the river, the North Shore being practically free of ice for the corresponding period.

Evidently winds from the South or South East will carry the ice to the North Shore.

Winds from the N.E. along the axis of the river are the winds that produce the worst storms, the fetch being in the order of 450 km at the mouth of the Saguenay.

They sometimes last for more than 24 hours and are most feared by navigators and port operators in the downstream stretches of the river (Fig. 1).

c) Waves

Winds of this magnitude over wide open stretches of water will generate waves of corresponding values. For example, National Research Council hind-cast studies for Gros Cacouna give a significant wave of 3.30 m.
All the ports mentioned before, from Cacouna downstream, are therefore either natural harbours, such as Sept-Iles and Rimouski or with man-made enclosures: rubble mound breakwaters at Gros Cacouna and Matane the latter with tetrapods as armor, or the perforated concrete crib at Baie Comeau.

Although breakwaters can be a barrier against floating ice, the opposite is true if the wind blows towards the gap in the breakwater. Ice can then fill the harbour basin in a very short time.

TIDES

The St. Lawrence river is a tidal stream over most its length, from the gulf to Three-Rivers, 145 km downstream from Montreal.

Tidal amplitudes for large tides along the river are as follows:

<table>
<thead>
<tr>
<th>Location</th>
<th>Amplitude (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Three-Rivers</td>
<td>0,30</td>
</tr>
<tr>
<td>Quebec</td>
<td>5,80</td>
</tr>
<tr>
<td>Pointe-au-Père (Rimouski)</td>
<td>4,60</td>
</tr>
<tr>
<td>Sept-Iles</td>
<td>3,70</td>
</tr>
</tbody>
</table>

Tides by themselves have no effect on the formation of ice. However, because of the currents they create and the icing of vertical surfaces, the ice problem in tidal waters becomes compounded for the wharf designer.

CURRENTS

Current velocities along the St. Lawrence vary of course according to the basic laws of hydraulics and the following table is a summary of the higher values at different locations.
Montreal | 5.0 kns | Downstream
Three-Rivers | 2.8 kns | Downstream
Quebec | 4.0 kns | Reversing

Downstream of Quebec City, at certain locations, for example at Cap-aux-Oies and at the mouth of the Saguenay, strong currents, sometimes changing in direction with depth are encountered. But these are localized and generally speaking currents will vary from 1 to 2.5 knots. Currents are mentioned here because in narrow ice infested stretches of the river, they cause a hazard to the docking and mooring of ships.

One of the worst locations for such hazards is right here in front of Quebec City.

ICE

a) Salinity

Salinity in the waters of the St. Lawrence river becomes noticeable at the downstream end of Orleans Island some 40 km from Quebec City and one can say that all the ice upstream of this point is fresh water ice. Some of this ice is carried downstream by winds and currents and can be encountered in the lower stretches of the river where it then acquires salinity from flooding by sea water.

b) Movement of ice

Because of the prevailing winds from the West and Northwest in winter time floating ice in the river is pushed against the South Shore of the river below the mouth of the Saguenay river, for long periods at a time. An ice pack of up to 5 or 6 meters in thickness and over a width of several miles can be encountered by ships heading for a port on the South Shore.
This phenomenon has been observed all along the South Shore right up to the Gulf and at times some harbours on the South Shore have been closed to navigation for a few days because of it.

Conversely all the ports on the North Shore in the river estuary particularly downstream of the Saguenay are ice free most of the winter season.

c) Thickness

The thickness of ice used for the design of structures in the St. Lawrence is 1 meter. In the gulf this figure would be too low.

HISTORICAL

At this stage, something should be said about the history of winter navigation and the design of wharves and other structures in the St. Lawrence.

a) Navigation

At the 1979 conference, J.V. Danys of Transport Canada presented a paper entitled "Development of Winter Navigation in the St. Lawrence Below Montreal".

It is not intended to give a summary of this paper but it should be recalled that winter navigation to Montreal by ocean going vessels operating on a regular schedule is little over 20 years old.

For some years before that time, there was some coastal navigation by small tankers bringing fuel supplies to oil terminals in small ports along both shores of the river but mainly to those ports located beyond the head of rail. This of course still goes on.
b) **Wharf designs of the past**

In this country, for the first 250 years of its history, timber was the material with which wharves were built. It was plentiful and available within short distances from the river. Timber was used to fabricate rock-filled timber cribs, fenders, decking, etc...

There were thousands of such timber cribs built in this country to serve as wharves whether on rivers and lakes or as bridge piers and sometimes for very large structures in tidal waters.

To reduce damage by waves and floating ice, the exposed faces of a crib wharf were faced with planks. Timber was also tarred and later on creosoted to protect against rot. They required maintenance but were easily repaired using local labor.

As rot increased in the upper portions of these structures, above low water level, that portion was demolished and replaced by concrete gravity walls resting on the cribs.

Up to a very few years ago, there were still some such structures in the harbours of Montreal and Quebec City in depths of water of 9 to 11 meters below low water. With their flexibility they could resist earthquake forces without suffering too much damage.

Timber piles were also used as wharf structures. As timber piles can be easily damaged by ice, they were used as support for a timber or concrete platform. As timber piles have a low bearing capacity, they were replaced in time by "H" shaped steel bearing piles supporting a platform on which a concrete wall is erected at the edges to retain backfill.

Wakefield sheet-piling, which is lapped wooden planks, was also used quite extensively. A wharf of this nature built in 1902 lasted until 1978 when the tie-backs gave in. Surprisingly, the wood was still in very good condition, the structure being located at Three-Rivers in fresh water.
Of course this wooden wall was capped by a concrete cope wall from L.W.L. up to deck level. The depth available at L.W.L. was 9 m.

Steel sheet piling walls for water-depths of up to 9 m were used extensively in the 1920's and 1930's. Beyond this depth, steel sheet piling can become uneconomical as earth pressures increase rapidly with the obvious necessity of using very heavy sections.

There are along the St. Lawrence a few examples of steel sheet piling walls with relieving platforms but this method in most cases can prove to be uneconomical.

Although most sheet piling walls were built in the upper St. Lawrence and in the Great Lakes in 8.25 m depth of water, some were also built in tidal waters, mostly in sheltered harbours.

Some sheet piling walls exposed to wave action in the tidal water of the estuary of the St. Lawrence and in the Gulf have collapsed and in two cases at least, failure has been attributed to accelerated corrosion of the sheet piles at or near sea bottom. In both cases the sea bottom was made up of sand. This material is suspended in the water under wave action and thus creates a sand-blasting action on corroding steel and repetition of such action will accelerate corrosion.

In 1930 the first concrete crib wharf structure was built in the harbour of Montreal. This type of structure is now widely used throughout the St. Lawrence river system and across Canada. It is permanent if this term can apply, it is almost maintenance free and in case of accidental break, it can be easily repaired.

Steel sheet piling is still being used nowadays in the St. Lawrence river system where suitable. Piles are also used to a great extent. However, "H" beams are now jacketed over the exposed portion in the tidal region and some distance below L.W.L.

Precast concrete piles are not favored by designers in the area.
On the other hand, large circular tube piles, filled with reinforced concrete, are in favor where soil conditions are such that they are the most economical solution.

Other types of structures, called ice-breakers or deflectors were also built in the past, either to protect bridge piers in smaller affluents of the St. Lawrence or to prevent floating ice from infesting berth areas. They were usually massive timber crib structures with a sloped top covered with a concrete slab so the ice would rise up on them and break in flexure. Those are not used any more but are replaced by much more efficient structures. As can be seen, most of the massive structures of old vintage were capable of resisting ice forces, with in most cases, no concern, on the part of the designer, with the problem caused by ice conditions in the docking and mooring of ships since winter navigation up until 1960 was the exception and not the rule.

But, for the navigation that did exist, there were problems and some of them remained unsolved and caused a lot of misery and hazards.

The advent of regular winter navigation and its quick expansion and the great advance in ice technology accomplished over the last twenty years have given the designers the incentive and the tools to solve most of these problems successfully.

ICE PROBLEMS IN WHARF DESIGN AND SOLUTIONS

Before we go into a discussion of ice problems in wharf design, we should remember that the type of structure chosen for a particular site is governed by soil conditions first. The structure is then designed to resist to prevailing ice conditions.

The most common problems caused by the presence of ice conditions in the St. Lawrence river are as follows:

A - Impact of ice sheets on exposed structures
B - Icing by accretion on vertical walls and fender systems
C - Floating ice in the berth area
A- Impact of ice sheets

The impact of ice sheets on exposed structures is the problem that causes most concern to designers as it involves the stability and security of the structures.

In ice infested waters, lighthouses and typical terminals made up of individual breasting and mooring dolphins are the structures most exposed. J.V. Danys of M.O.T. has discussed quite competently at previous P.O.A.C. conferences the forces on isolated lighthouses in the St. Lawrence and we refer those interested to these previous papers.

As to petroleum terminals for V.L.C.C.'s, there is one opposite Quebec City, more precisely at St. Romuald (Fig. 3). Breasting and mooring dolphins are made up of circular sheet pile cells. There are three similar cells at 56.4 m centre to centre located at the upstream end of this wharf in a line at right angles to the berth. Their purpose is to stop the ice and deflect it towards midstream.

![Fig. 3 - Oil terminal at St. Romuald](image)

Although preliminary designs were made for at least two other such terminals elsewhere in the river they have not materialize thus far. In both cases, stability of the structures was governed by the forces produced by the impact of ice sheets. Dolphins and ice deflecting structures are designed to withstand the impact of ice floes.
B- Icing by accretion on vertical walls

This, on most wharves causes some problems. However, accumulation of an ice belly on the walls of a ferry wharf can become so severe as to impede operation of the ferry boat. This is due to accretion of ice from two causes: tides and clapotis from frequent berthings and departures of the ferry boats.

In Quebec City and in Levis, the ferry landing wharves before they were redesigned and rebuilt presented this problem. Up to 2.5 m of ice was measured on the quay walls preventing the automobile ramp from reaching the ship. There are twin ferry boats here, with landings every half-hour.

Fig. 4 - Icing on the old Levis landing wharf before redesign.

The solution in this case was to install heaters in the walls during reconstruction. To insure better convection, the outside form for the concrete quay wall was made of steel anchored into it as a permanent facing.

Fenders were removed from the wharf and instead, a rubber fender was installed all along the ship bumper. These ships of course are small at 1500 tons. To save on power demand, heaters are arranged in groups which are switched on alternately and are shut off entirely when motors actuating the mobile ramp mechanism are being run.

The system was also designed to allow ice to accumulate to a permissible thickness before switching on the heaters.
The same system was applied elsewhere on similar vertical concrete walls but without the steel plate facing.

Both system have operated in a satisfactory manner for several years. Heaters were also installed in the steel framework on piled fender systems and near the hinges of some movable ferry ramps. To prevent icing of these structures which may unduly increase the weight of the fender systems or block the free movement of the ramp. There again, heaters are operated in small groups at a time to reduce power demand.

Icing on piles is also a common problem. Icing will add weight to the piles and may impede docking if piles are not properly located. Piles supporting dolphin platforms for example should be set back from the wall face at the berth side.

Fig. 5 - Breasting and mooring dolphins on piles, Ferry landing - Baie Comeau.

C- Floating ice in the berth area

In enclosed harbours, winds blowing towards the gap in the breakwater can push floating ice inside the harbour and quickly fill the protected basin.
Generally speaking, if there is sufficient traffic in and out of the harbour, a solid ice sheet will not be formed. As breakwaters are designed so the gap is not facing the direction of origin of the prevailing winds, floating ice resulting from non-prevailing winds, does not stay in the harbour for very long.

RO/RO and ferry ramps should be designed so that the ship backing-in to the ramp can chase the ice from under the ramp.

Under such adverse conditions, even in very cold temperatures, ships designed for navigation in ice infested waters will have enough installed power to manoeuvre and reach the berth.

Wharves in fast waters, such as marginal wharves along the shores of the river or terminals for V.L.C.C.'s or similar large ships, located in open waters, usually need ice deflectors to protect the berths from the undue presence or impact of floating ice. Without proper protection, berths in such situations would be almost unserviceable.

There are two such ice deflectors across the river from Quebec City.

First there is the oil terminal at St. Romuald that was mentioned and briefly described above (Fig. 3). The other one is just upstream of the ferry landing wharf at Levis opposite Quebec. This ice deflector used to be made up of a large timber crib. When the ferry wharf was redesigned the deflector was also redesigned.

As it is now, it consists of a triangular concrete box, prefabricated and floated over predriven piles of the drilled-in caisson type. Sockets in the concrete box were positioned to fit over the piles. The space between the piles and socket walls was then filled with concrete to ensure a fixed connection. The concrete box was later filled with rock. The walls of the box start at just under L.W.L. and extend above H.W.L. The ice is thus deflected past the berth area with the least possible restriction in flow area.
It is hoped that this short description of this mighty river and the parameters of climate, tide and currents that condition its environment have given an idea of the challenge that these represent to the wharf designer.

As more and more data becomes available and as research and technology advance, there will perhaps appear more refined solutions to the problems described in this paper.

The solutions described above have all given so far very satisfactory results at minimum costs.

ACKNOWLEDGMENT

The collaboration of Mr. Pierre Lamarre, P. Eng., and Mr. Yvon Morin, P. Eng., of "Groupe Lavalin", in the preparation of this paper, is gratefully acknowledged.

REFERENCES

Michel - Ice Mechanics


Abstract
For wide structures with small freeboards, such as artificial or natural islands, there is a risk during freezeup that the ice sheet may ride up the beach and onto the island working surface. The ice defense system considered for Challenge or Alaska Islands, located in the Maguire chain of natural barrier islands in the Alaskan Beaufort Sea, is discussed, and actual active defense procedures deployed are described.

1. Introduction
Ice rideup on a wide sloping beach at freezeup can occur only if the environmental driving force, usually wind-related, can overcome the frictional resistance of the beach material and gravity to move the blocks up the beach. Documentation of historical ice rideup events have been presented by Shapiro, et. al.[1] and Kovacs and Sodhi[2]. To prevent rideup onto the working surface it is desired to create an ice pileup, preferably at the toe of the beach.

During freezeup thin ice can slowly rideup gravel beaches without breaking or piling up. At Challenge Island, shown in Figure 1, it was intended to promote a shoreline pileup with a graded berm along the beach causing compression instability and to create an ice jamming mechanism near the top of the beach through deployment of triangular concrete tank traps (5' high x 8' base x 3' wide). A sketch of proposed ice defense mechanisms is shown in Figure 2. If rideup did occur and went beyond the back of the tank traps a D-8 Caterpillar tractor would be available to push the ice off the island working surface. These methods are similar to those suggested by Croasdale, et.al. [3].
Figure 1  Aerial view of Challenge Island in October 1980 looking northeast.

Figure 2  Sketch of beach cross-section showing proposed ice defense mechanisms.
2. Beach Survey

An initial beach survey of Challenge Island was performed during September 24-26, 1980, just after freezeup. Two cross-sections of the northern beach are shown in Figure 3. The beach slope averaged $5^\circ$-$7^\circ$ where the tank traps were positioned. The tank traps were embed­ded in the beach 6"-17" deep, spaced 12'-15' apart. The toe of the tank trap line was located only 12' from the shoreline, due to lack of storage space on the island (see Figure 4). A close-up of one of the tank traps is shown in Figure 5. There was no room to grade a berm along the beach front.

Quite a few multiyear ice fragments had collected along the shoreline in front of the row of tank traps. At the time of the beach survey, it was thought that these multiyear frag­ments might help stabilize the landfast ice and retard, if not prohibit, ice rideup; however, ice rideup did occur anyway as described in the next section.

3. Ice Rideup Assessment

After the first-year ice sheet had grown to 6"-8" thick, ice rideup occurred along the northern beachfront, due to a 20-25 knot northeasterly storm wind. The ice sheet remain­ed relatively intact as it moved slowly up the beach. As the ice encountered the tank traps and adjacent beach, the change of slopes caused the ice sheet to shear into wide fingers that rode up the concrete. The overhanging ice fingers failed under their own weight and fell or tilted into the area between the tank traps (Figure 6). Ice movement stopped with ice resting on the full length of the tank trap slope and coming up the beach between the traps just to the vertical backface of the concrete line, a total of 20'-25' (see Figure 7). The ice movement rate was estimated to be approximately 2'-5' per hour.

The force P per foot of beachfront resisting rideup can be calculated using the following equation taken from Croasdale, et. al. [3],

$$ P = L \cdot t \cdot \rho_i (\sin \alpha + \mu \cos \alpha) \quad (1) $$

where

- $L$ = distance ice rode up the slope
- $t$ = ice thickness
- $\rho_i$ = weight density of ice
- $\alpha$ = slope angle
- $\mu$ = coefficient of friction between ice and slope material
Figure 3. Surveyed profiles of beach and bathymetry nearshore.
Figure 4. Northern beach of Challenge Island with line of tank traps and multiyear ice fragments at the shoreline.

Figure 5 Closeup of concrete tank trap embedded in the beach gravel.
Figure 6  Ice rideup failed between tank traps and caused ice jamming.

Figure 7  Ice rideup on entire slope of several tank traps.
To calculate the effectiveness of the tank traps in retarding ice rideup, three key parameters need to be considered: 1) the coefficient of friction of ice on concrete versus gravel, 2) the slope angle of the concrete versus the beach, and 3) the percentage of beachfront covered by the tank traps. In general, the ice will slide easier on smooth concrete than on beach gravel; however, energy of moving ice is dissipated in lifting ice blocks onto the tank traps. Letting

\[ f(\alpha, \mu) = \sin \alpha + \mu \cos \alpha \]  

from equation (1), the effects of changes in slope and friction coefficients can be seen in Figure 8. For example, if \( \mu = 0.3 \) for ice-concrete and \( \alpha = 32^\circ \), then \( f(\alpha, \mu) \) becomes 0.78, exactly the same value as taking \( \mu = 0.7 \) for ice gravel and \( \alpha = 5^\circ \) for the beach slope. If these assumptions for \( \mu \) are valid, ice rideup is equally likely to occur on the tank traps or the beach; however, friction tests should be performed to verify assumed \( \mu \) values. The primary advantage of the tank traps lies in their ability to cause the ice to jam between the traps and initiate ice pileup seaward. Since the tank traps covered only 20-25% of the beach, spacing between tank traps (12'-15') appeared to be too great to insure ice jamming. In the future, spacing should be a function of tank trap width and height and the characteristic length of the ice sheet.

During the recent ice rideup event, the tank traps did function as expected, but their effectiveness in retarding ice rideup is inconclusive since the ice stopped before any ice pileup had a chance to form against the initial ice jamming between tank traps.

In several places rafting had occurred with one ice sheet thickness overriding another after moving about 15' onto the beach. As rideup begins, the force due to slope resistance increases as the ice sheet moves further up the beach. If the wind-generated driving force is sufficient to keep the ice sheet moving, then a combined buckling and flexural load can increase until the ice sheet fails near the shoreline, and rafting may occur.

If buckling governed ice sheet failure during rafting the stresses can be calculated using the following equation presented by Wang [4].

\[ \sigma_b = C \sqrt{\frac{E}{\gamma_w D}} \frac{t}{t} \]  

where the coefficient \( C = 0.85 \) for a long beach relative to ice thickness, \( D = \frac{E t^3}{12 (1-\nu^2)} \) is the flexural rigidity of an ice plate, and \( \gamma_w \) is the seawater density. If assumed values of \( E = 100,000 \) psi and \( \nu = 0.3 \) for the 6" ice sheet, \( \sigma_b = 38 \) psi.
Figure 8  Graph showing effects of changes in slope and friction coefficient in Equation (I).

Figure 9  Looking west along northern shore of Challenge Island.
On the other hand, if bending failure governed ice sheet behavior, the following equation given by Croasdale [3] may be used to determine the stress.

\[ \sigma_f = 1.5 Z (\delta_i)^{0.75} (E/t)^{0.25} f(\alpha, \mu) \]  

(4)

where \( f(\alpha, \mu) \) is given by equation (2) and \( Z \) is the maximum island freeboard or tank trap elevation. If \( Z = 4 \) feet and \( f(\alpha, \mu) = 0.78 \), then \( \sigma_f = 49 \) psi. Comparing \( \sigma_b \) and \( \sigma_f \) indicates that buckling may have governed the failure causing rafting, but it probably was a mixed-mode failure mechanism.

Some small multiyear pieces (3'-5' across) were pushed ashore along with the ice sheet rideup. Larger multiyear ice fragments offshore appear to have moved, both translated and rotated during the ice movement. There is no pileup against or rideup on the multiyear fragments, indicating the multiyear ice moved or the ice sheet slid past so slowly it deformed plastically without breaking. By studying pictures and observing the ice sheet after rideup, it seems that possibly both events took place (note the "wrinkles" in the ice sheet offshore in Figure 9). It is likely that if the multiyear ice had not been there, the rideup may have more extensive and penetrated further onshore.

4. Slotting Procedures

As midwinter precautions against ice movement and rideup on the island, a series of 1000' long trenches were cut parallel to shore with a Ditch Witch R-100 rubber-tired ladder-type trencher (Figure 10), equipped with a reinforced chain having carbide-tipped permafrost teeth and slant-cutting boom capability having 12° tilt to each side. Each 1000' slot through 40"-43" thick ice was trenched at an average rate of 7-9 feet per minute along the ice. The trencher cutting boom was slanted toward the island so any ice push from offshore would tend to raft over the landfast ice sheet nearshore. Two slots were cut at one time, and each week thereafter, one of the previous slots was reopened and a new parallel slot trenched. Slotting operations were conducted approximately 1000 feet offshore where the water depth was 14'-15'.

After completion of the initial 1000' trenches, a large smooth ice pan (1000' long x 400' wide) was chosen for a slotting experiment, approximately one mile northeast of the rig camp where water depths ranged from 25-30 feet. A series of "square waves" were laid out according to Figure 11 to promote ridge formation, since most compression ridges begin by finger rafting. The long axis of the experiment was laid out along a NW-SE line, anticipating major storms to occur out of the northeast. The Ditch Witch cut nine vertical, 150' long slots (PQ
Figure 10  Ditch Witch ice trencher performing slotting operations.

Figure 11  Diagram of slotting experiment layout to induce ice ridge formation.
and RS typical) while slanted slots (QR and ST typical) were each 75' long. The initial ice thickness was 42". During the four weeks that the experiment was kept open, no noticeable ice movement occurred even though two northeasterly storms blew. It appeared that the landfast ice sheet just north of the barrier islands had become relatively stable after the ice thickness reached 12"-15".

5. Acknowledgment

The authors gratefully acknowledge Sohio Petroleum Company for its permission to present this paper.

6. References


FIELD TEST STUDY OF "PACK ICE BARRIER"

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1. Abstract

Based on model and field test studies of various ice resistant structures, a unique offshore structure for protection of marine life from damage by pack ice was developed and tested in ice covered sea of Okhotsk.

The structure is composed of inclined pipes and horizontal tubes which are filled with concrete.

In order to investigate the effect of the structure by field tests, large test structures were installed in 4.0 meters of water.

The field tests were conducted over a period of two years. The structure encountered a heavy storm on October 19, 1979 and a significant wave height of 3.6 meters was recorded. However, no heavy pack ice reached near the coastline during the two winters.

No specific measuring equipment was installed on the structure, but investigations of the behaviour of pack ice near the structure were conducted by using 8 m/m movie and 35 m/m still cameras and under-water observations.

It was observed that the structure was effective in keeping out the pack ice and that marine life was not affected.
2. Introduction

At the beginning of winter, pack ice areas of the Okhotsk sea expand east-ward from the east coast of Sakhalin Island and drift to the south with winds and ocean current, etc. In late January the pack ice approaches the coastline of Hokkaido. Pack ice normally ranges from 40 cm to 50 cm in thickness and they include rafted ices and pressure ridges.

Because of the great need for marine products in Japan, the fishermen and Government of Hokkaido are making efforts to facilitate cultivation of marine products such as tangle, scallops and sea urching under pack-ice conditions. Such development projects are presently located at Saroma Lagoon and off the Saruru Coast. See Fig-1.

At the Saroma Lagoon, the culture facilities were damaged by heavy pack ice in January of 1974.

On the Saruru Coast, cultivated tangle was heavily damaged by the grounding of ice floes.

In order to prevent damage by pack ice, unique offshore structures called "Pack ice barriers" were developed by Mitsui Engineering & Shipbuilding Co., Ltd. The plan of the "Pack ice barrier" is shown in Fig. 2.

The design was based mainly on model and field tests of various ice resistant structures conducted by Mitsui.

In the summer of 1979 three test structures comprising steel pipes of 56 cm diameter filled with concrete were installed in 4.0 meters of water on the Saruru coast near Mombetsu.

The main purpose of this field test was to investigate the effect of pack ice on the structures and to observe the behaviour of ice floes around them.

In this paper the design concept of pack ice barriers and the results of field test studies are presented.
3. Design Concept

Design Conditions

The assumed design conditions were derived from field tests and are shown in table - 1.

Table-1. Design Conditions

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Water Depth</td>
<td>4 meter (L.W.L. = 0.0) (H.W.L. +1.2)</td>
</tr>
<tr>
<td>(2) Sea Ice</td>
<td></td>
</tr>
<tr>
<td>1) Ice Thickness</td>
<td>50 cm</td>
</tr>
<tr>
<td>2) Flexural Strength</td>
<td>5 kg/cm²</td>
</tr>
<tr>
<td>3) Young's Modulus</td>
<td>10,000 kg/cm²</td>
</tr>
<tr>
<td>4) Friction factor between</td>
<td>0.26</td>
</tr>
<tr>
<td>structure and Sea Ice</td>
<td></td>
</tr>
<tr>
<td>(3) Wave Height</td>
<td>4.0 meter (In H.W.L.)</td>
</tr>
<tr>
<td>(4) Current</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

Design

On the basis of the design conditions stated above, eight types of structures including a concrete block structure and a steel pipe truss structure were developed conceptually in order to derive optimum pack ice barrier. Considerations such as the ability to keep off pack ice, ice and wave resistance, water permeability, ease in foundation erection, and construction costs were taken into account. The general configuration of the optimum structure is shown in Fig. 3, and its principal features are shown in Table 2.

General configurations of alternative structure are also shown in Figs. 4 and 5; the former showing an example of structure to be installed in water of considerable depth and the later showing an example of structure built with piles for installation on a sea bottom of soft material like sand or clay instead of a rock sea bottom.
Table-2. Principal Features of Experimental Structure

<table>
<thead>
<tr>
<th>Feature</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>10 m x 3 units = 30 m</td>
</tr>
<tr>
<td>Width</td>
<td>11.7 m</td>
</tr>
<tr>
<td>Height</td>
<td>5.9 m</td>
</tr>
<tr>
<td>Diameter of steel pipe</td>
<td>558.6 mm</td>
</tr>
<tr>
<td>Spacing of steel pipe</td>
<td>1.8 mm</td>
</tr>
<tr>
<td>Weight</td>
<td>50 ton</td>
</tr>
<tr>
<td>Water Depth</td>
<td>4.0 m</td>
</tr>
<tr>
<td>Maximum Ice Force</td>
<td>16 ton</td>
</tr>
<tr>
<td>(Horizontal component)</td>
<td></td>
</tr>
<tr>
<td>Max. Wave Force</td>
<td>30 ton</td>
</tr>
<tr>
<td>Sea Bed</td>
<td>Rock</td>
</tr>
</tbody>
</table>

The pack ice barrier designs developed by us offer the following features:

(a) The structure can effectively prevent the pack ice from passing through to the shore.

(b) Horizontal ice force exerted on the structure is reduced considerably by inclined pipes. Therefore the force exerted on sloping structure to cause flexural break-up of ice is small.

(c) Water can flow freely through so that the environmental conditions are not adversely affected.

(d) The structure is easy to construct and install.

(e) It is stable against pack ice and waves.

Angle of Tilt

The theoretical ice forces (horizontal component) acting on the pack ice barrier are shown in Fig. 6. As is clear from this figure, the smaller the angle of tilt (which is formed with the vertical plane of the structure), the larger the ice force. Experimental result for this relationship between ice force and angle of tilt was obtained by Saeki [1] and has also been demonstrated in our model tests.
The ice force can be reduced by use of a large angle of tilt (e.g., 60 deg.), but a large angle of tilt dictates a larger structure which is costly to build. A small angle of tilt (e.g. 30 deg.) permits a reduction in the size of structure, but strong ice force exerted on the structure require costly structural members and foundations. The advantages and disadvantages of these two alternatives were studied and it was concluded that the optimum angle is around 45 deg.

**Member Spacing**

The pack ice barrier is made up of inclined steel pipes which are arranged at fixed spacings. When pack ice acts on the structure of this design, it will not pass between the steel pipes but ride-on the inclined plane of the structure and break by bending as one plate if steel pipes are arranged at narrower than seven times of pipe diameter. This has been demonstrated in our model tests. The structural members must be strong enough to bear the ice force.

4. **Field Tests**

Three full-size experimental structures (each having a length of 10 m) shown in photo-1 were installed at the coastline site in the summer of 1979, and field tests are still continuing. This section deals mainly with the observations made in the 1979-1980 period.

1) Pack ice prevention ability

In the winter of 1980 pack ice reached the Hokkaido shore on the Okhotsk sea side a little later than usual. At the site of the experimental structures, pack ice reached the shore on February 12. On arrival of pack ice the experimental structures began to exhibit its pack ice prevention ability. It formed an effective barrier against the pack ice and it facilitated the freezing of the sea.
However, the experimental structures which had a total length of 30 m were not wide enough to keep out all pack ice as a result of which some pack ice flowed behind the experiment structures at the sides.

On February 18, pack ice left the shore for the open sea but on that occasion land fast ice inside the experimental structure line remained.

On February 24, pack ice approached the shore again, but the land fast ice at the coastline area near the experimental structures (that is, the fishing grounds) prevented pack ice from further advancing towards the shore.

These observations suggested that the pack ice barrier could prevent the inflow of pack ice and facilitates freezing behind the structure.

(2) Observation by diving around the structure revealed that pack ice was stopped coming to the coastline by the structures and the ice inside the structure was not a state of rubble. See Photo. 3.
Throughout the pack ice period, pack ice did not adfreeze the experimental structure because of the change in tide level.

(3) Wave Resistance
In October 1979 Typhoon No. 20 swept across Hokkaido. In the midst of the typhoon, a significant wave height of 3.6 m ($H_{1/3} = 3.6$ m, $H_{max} = 10$ m) was recorded at the Monbetsu Wave Observatory near the site. The experimental structures at a water depth of 4 m were located in the breaking zone. It was therefore assumed that the structure encountered waves of about 3.2 m high.

They resisted the enormous forces of such waves very well; neither did they slide nor overturn, but remained firm and fast throughout the typhoon.
5. Conclusions

(1) A design of a unique structure for the protection of marine life against damage by pack ice, designated "pack ice barrier," was developed and full-scale experimental structures were installed in the sea. As result of observations continued over 2 years, it has been shown that the experimental structures of this design are effective against damage by pack ice.

(2) The experimental structures which feature an exceedingly high degree of water permeability did not allow drift sand to be deposited or produce any adverse effects on the marine growth.

(3) It was also demonstrated that the experimental structures neither slid nor overturned, but remained firm and fast throughout the typhoon which gave rise to braking waves (deep water wave height $H_{1/3} = 3.6$ m). The tests provided evidence that these experimental structures possessed adequate margin of safety against such breaking waves.

(4) The experimental structures were installed on the rock at a depth of 4 m. It was shown in the present study that these structures would give satisfactory performance even in the following circumstances:
   (a) Still deeper water (see Fig. 4)
   (b) Sea-bottom of soft material like sand and clay (See Fig. 5)

(5) The experimental structures which feature a very simple construction is well within current state-of art capabilities, no matter how unfavorable the local conditions may be, and are cheap to build.
ACKNOWLEDGEMENTS

The cooperation of Saruru Fishermen's Cooperative Associations and Government of Hokkaido is gratefully acknowledged. This work was partly supported by the Japan Marine Machinery Development Association (JAMDA).

REFERENCE

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Fig. 1
Location of Site

Fig. 2
Plan of "Pack Ice Barrier"
Fig. 3
Isometric View of "Pack Ice Barrier" used in Field Test

Fig. 4 Pack Ice Barrier Alternative for Deep Water

Fig. 5 Pack Ice Barrier Alternative for soft material sea Bed.

Fig. 6
The Relation between Horizontal Ice Force and angle for the Pack Ice Barrier
Photo. - 1
Pack Ice Barrier

Photo. - 2
Pack Ice around the "Pack Ice Barrier"

Photo. - 3
Under Water Observation
DOCK FLOATS SUBJECTED TO ICE

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Abstract

Results of full scale field testing and observations of floating docks placed year round in a boat harbor subjected to severe ice conditions are presented. Information on material performance, shape and color effects, methods of installation, and other details is included. The information is prepared from a controlled four year field experiment at Bayfield, Wisconsin employing many types of floating docks, and from field observations of floating dockage systems in Great Lakes harbors.

Introduction

Dockage facilities in northern marinas and harbors are damaged by ice. This paper reports on dock pontoons left in the ice. A companion paper on fixed harbor structures is included in these proceedings.

Some marinas remove all docks in the fall and replace them in the spring. This is costly and not feasible at many sites. In other harbors ice suppression systems are used to keep floating docks free from ice.

There is a need for pontoons and floats that are not damaged by ice during an annual freeze in. Accordingly, in 1977 a number of floating dock manufacturers and construction firms were invited to participate in a field research project. In part the letter of invitation stated,

"We plan to make a comparative and quantitative evaluation of several commercially produced floating dockage systems subjected to severe winter ice conditions and to develop recommendations for materials, shapes, colors, method of installation, and other factors pertinent to creating ice-resistive floating dockage systems. We believe floating dockage may be a viable choice in the Great Lakes where water levels fluctuate if resistance to ice damage can be improved. Energy charges can be eliminated if dock floats can be left in all winter without operating a supporting deicing system; or on the other hand cost savings can result if docks do not have to be removed each winter."
We invite you to furnish and erect (and subsequently disassemble) one of your standard dock modules using your customary procedures; or perhaps using design or erection variations you would like to try out. The project will be located in the protected harbor of refuge at Bayfield, Wisconsin. This harbor has severe climate and ice conditions. It is, however, free of floes.....

Four organizations accepted the invitation and placed different products in the Bayfield harbor. This paper reports the results of four winters of observations and measurements on these floating docks in ice.

Description of Test Site and Procedures

The floating docks were placed in the public harbor at Bayfield, Wisconsin on Lake Superior (Longitude 91°, Latitude 47°). The harbor is protected by breakwaters and not subject to ice floes and movements. In the area where the docks were placed the water is 4 m deep. This area is unshaded and exposed to wind and snow. Ice forms in late December and melts out in late April, a period of 4 months. On very cold nights the temperature can reach -35°C. Average annual snowfall is about 3 to 4 m.

A survey grid was placed on the ice (as well as on the floating docks frozen in the ice). Readings with surveying instruments were periodically made to note horizontal and vertical movements. Ice thicknesses and water temperatures were recorded several times during the winter. Visual inspections of the site and docks and freeboard measurements were made at the same time.

The docks were towed in the summer to marina and other areas in Bayfield where they were used by boaters and swimmers. In the fall they were towed back to the test site area. A barge and crane were used to pick the docks out of the lake for a careful inspection both before freeze up and after spring meltout. Pontoon and dock damages were recorded and identified as ice related or summer-use related.

Description of Floating Docks

The floating docks observed during the tests are described in the following paragraphs. Figure 1 is a view of some of the docks and the harbor area. The immersion depths ranged between 10 and 25 cm and the deck freeboard height above water between ½ and 1 m.
With one exception the docks were free to move, that is they were not restrained with cable and anchors, pilings or other restraint systems. Telescoping spuds were used on the MEECO dock (described subsequently) during the first winter. The docks were frozen in several meters away from one another and not in the area of ice cracks near the breakwaters and shorelines.

A complete marina dock system is articulated with many piers joined together. These tests do not duplicate this important feature. The docks tested were separate floating elements.

a. MEECO “Admiral” Dock, MEECO Marinas Inc. (Figure 2). Installed in fall 1977. U-shaped pier with 10 ft. x 20 ft. (3.0 m x 6.1 m) berth area enclosed with 3 ft. (0.9 m) finger extensions from a 4 ft. (1.2 m) header pier. Floatation is a hollow white expanded polystyrene rectangle formed with 4 in. (10 cm) sides and bottom and covered with a thin lightweight concrete deck poured on a corrugated metal form. The floatation material is covered with a 15 mil (0.4 mm) coating of polyurethane and reinforced with metal truss work. The ends of the fingers are tied together with a below-the-water truss frame.
b. MEECO "Commodore" Dock, MEECO Marinas, Inc. (Figure 3). Installed in fall 1979. T-shaped pier supported on four black high density polyethylene pontoons about 3 ft. x 3 ft. x 2 ft. high (1 m x 1 m x 2/3 m high). The pontoon has a wall thickness of 3/16 in. (5 mm) and is filled with a foamed-in-place polystyrene. The deck is wood, supported on a metal truss work frame.
c. Rotocast "DuraFloat" Dredge Pipeline Float, Rotocast Flotation Products (Figure 4). Installed in fall 1979. Dredge pipeline support to simulate a rectangular dock (without a deck) on three blue 26 inch (66 cm) round pontoons 3 ft. (1 m) long. The pontoon has a wall thickness of \( \frac{1}{8} \) in. (6 mm) and is filled with closed cell urethane foam.

![Figure 4. Rotocast "DuraFloat" Dredge Pipeline Float](image)

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d. United Flotation Systems "Heavy Duty" Dock, United McGill Corporation (Figure 5). Installed in fall 1977. Rectangular pier 6 ft. x 24 ft. (1.8 m x 7.3 m) supported on four 24 in. (61 cm) round 12 ft. (3.7 m) long pontoons. The pontoons are filled with expanded-in-place poly­­styrene foam. Two pontoon shells are galvanized steel, 20 gage (.039 in. minimum, 1 mm) with \( \frac{1}{8} \) in. (6 mm) high ribs on 2 in. (5 cm) centers formed outward; one is aluminum, minimum thickness of .040 in. (1 mm) with \( \frac{1}{8} \) in. (6 mm high ribs on 2 in. (5 mm) centers formed outward; and one is black high density polyethylene with wall thickness of 3/16 in. (5 mm). The deck is wood and metal decking supported on a metal truss work frame.
e. BERO "Marine" Dock, BERO Corp. (Figure 6). Installed in fall 1977 (and some pontoons that were substandard replaced in fall 1979). Four rectangular dock sections, joined in an L-shape, supported on either 4 or 6 pontoons: 6 ft. x 16 ft. (1.8 m x 4.9 m) wood dock with cross decking; 8 ft. x 16 ft. (2.4 m x 4.9 m) wood dock with long decking; 6 ft. x 20 ft. (1.8 m x 6.1 m) light duty aluminum dock; and 6 ft. x 20 ft. (1.8 m x 6.1 m) heavy duty aluminum dock. Pontoons are USS Chemical (now Zarn Company) PolyFloat black high density polyethylene, 4 ft. x 2 ft. x 1 ft. - 8 in. high (1.2 m x 0.6 m x 0.5 m high) filled with closed cell polystyrene foam. The wall thickness is 3/64 in. (1 mm).
f) BERO "QCI Marine" Dock, BERO Corp. (Figure 7). Installed in fall 1979. Rectangular pier 6 ft. x 20 ft. (1.8 m x 6.1 m) supported on six USS Chemical (now Zarn Company) PolyFloat black high density polyethylene 4 ft. x 2 ft. x 1 ft.-8 in. high (1.2 m x 0.6 m x 0.5 m high) pontoons filled with closed cell polystyrene foam. The wall thickness is 3/64 in. (1 mm). The deck is aluminum planks and frame.
Field Data and Observations of Pontoons

During the four year test period both an unusually severe winter and a mild winter were experienced. The maximum ice thickness measured was 75 cm. An average thickness of 50 cm would be representative of the winter test periods. The site was frequently snow covered with an average snow depth of 15 cm. At times it was free of snow, and at other times the docks were completely buried and not visible. Water temperatures (isothermal with depth) were recorded and ranged between 0.0° and 0.5°C. An average value of the water temperature representative of the winter would be 0.1°C.

Horizontal movement of the ice cover and frozen-in docks was 5 cm or less in any direction. The elevation of the ice cover dropped about 20 cm during the winter. Additionally some of the floating docks were observed to be drawn down about 10 cm further into the ice sheet. This was occasional in that it was not observed on all docks or during all winters. Round as well as side tapered pontoons went down—they did not pop-up and ride out of the ice.

Thermal melting at the ice surface pontoon interface was sometimes observed on sunny winter days when the docks were not covered with snow. This melting occurred next to colored pontoons as well as next to the aluminum, galvanized steel, and white expanded polystyrene pontoons. The melted area was about 1 cm wide and 2 cm deep.

No major damage or failure of any of the pontoons occurred. Some minor damages or deformations were observed.

The MEECO "Admiral" Dock was in the ice for four years and experienced several holes in the polystyrene pontoon walls. These were from burrowing animals and occurred during one summer and during one winter. Also during one summer a piece of the pontoon side wall was broken out. These non-ice related damages were not repaired and caused no subsequent problems. The polystyrene material was slightly creased at the water line and a 15 cm piece of it split off from an end of one finger pier.

The MEECO "Commodore" dock was in the ice for two winters and no ice effects were observed.

The Rotocast "DuraFloat" Dredge Pipeline Float was in the ice for two winters. One pontoon end was deflected inward a slight amount.

The United McGill "Heavy Duty" Dock was in the ice for four years. One end cap was loosened by the ice. When the dock was lifted from the lake, some water was observed to drain out from all cylindrical pontoons.

The BERO "Marine" Docks were in the ice for either four years or two years. Some of the PolyFloats were badly damaged during the first summer when a storm smashed the dock against a rock shoreline. Also some of the pontoons were not completely filled with foam and were deficient in wall thickness at the corners and bottom of the polyethylene encasement. Here the ice dimpled and pinched the pontoon corners. These substandard pontoons were replaced with new PolyFloats before the third winter. Both the replacement PolyFloats and the original standard PolyFloats were not damaged by the ice.

The BERO "QCI Marine" Dock was in the ice for two winters and no ice effects were observed.
Summary and Conclusions

All of the dock floats observed for a test period of either two or four winters experienced no significant damages from being frozen into the ice. The encasement material and its color, the size and shape of the pontoon, and the immersion depth into the water (ice) appear unimportant.

One dock product which was not completely filled with foam and which had a sub-standard wall thickness was damaged by the ice. It was removed from the tests. The unencased polystyrene material showed signs of wear and weathering after four years in the water. The encased floats showed some discoloration and water marking after four years of use.

These field tests covered only foam-filled pontoons with polyethylene and metal encasements, and styrofoam floats. Other products were requested but not furnished. They include hollow pontoons with polyethylene, metal and concrete walls.

However, from these observations, (and from other observations in many Great Lakes harbors), it is concluded that the furnished pontoons are capable of withstanding the effects of being frozen in the ice during the winter.

Postscript

There are many aspects to the successful design of floating dock systems that are to be left in the ice. The author is monitoring a number of harbors with floating docks and is preparing recommendations for dockage design. This paper has only covered the results of the test docks placed in the ice at Bayfield, Wisconsin.

Acknowledgements

This paper is prepared from work funded (in part) by the University of Wisconsin Sea Grant Institute under a grant from the Office of Sea Grant, National Oceanic and Atmospheric Administration, U.S. Department of Commerce and by the State of Wisconsin.

The writer is especially grateful for the interest, help and support of the manufacturing and construction firms that furnished dock components for the study. They are BERO Corp. of Waterloo, New York, MEECO Marinas, Inc. of McAlester, Oklahoma, Rotocast Flotation Products of Brownwood, Texas, and United McGill Corporation of Columbus, Ohio.

Additionally, the City of Bayfield, Wisconsin and Mayor Lawrence Wachsmuth, Nelson Surveyors, and NiChevo Ferry Line Constructors have provided sustained support and assistance throughout the entire project, and it is appreciated.
AN EXPERIMENTAL INVESTIGATION OF THE
CRUSHING STRENGTH OF ICE

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ABSTRACT

Results of an experimental program to measure the crushing strength of ice are presented. A portable, hydraulic test unit was built and used to perform a total of 38 field tests. Flat indenters of 1, 2, 4, 8 and 12 foot widths, and circular indenters of 1 and 2 foot diameter were tested on a fresh water lake near Calgary, Alberta. Laboratory tests were performed on samples from the lake. Testing included Brazil tests, direct shear tests, and three types of compression tests.

The natural ice sheet of the lake was of the vertical c-axis type. Test ponds were cut and "seeded" with snow to produce a fine-grained, horizontal c-axis type of ice. Both types of ice were tested. The thickness of the sheets ranged from 9 to 30 inches.

The relationship between ice strength and strain rate was established in the field in a series of tests in which the rate was varied between $0.06 \times 10^{-3}$ and $6.0 \times 10^{-3}$ sec$^{-1}$. For the remaining tests, a constant displacement rate corresponding to the maximum ice pressure was used.

Analysis of the data yields the following conclusions:

1. There is a significant "size effect" for the crushing strength. The strength decreases as the aspect ratio increases, where aspect ratio is defined as the width of the indenter divided by the thickness of the ice sheet.

2. Horizontal c-axis ice was much stronger than the vertical c-axis ice.

3. Pressure distribution through the thickness of the ice sheet is approximately uniform.
1. **INTRODUCTION**

In order to explore and produce the oil basins in the offshore Arctic, it will be necessary to build structures capable of withstanding the forces caused by the movement of the surrounding ice sheet. For vertical-sided structures, as well as local areas of other types of structure, the ice sheet will fail in the crushing mode. In this paper "crushing strength" refers to the effective pressure exerted by the ice; it is the total load divided by projected area. In order to predict the crushing strength of the ice sheet the mechanical properties must be incorporated into a valid theory of failure. Ice is a complex material; it is anisotropic, non-homogeneous; and, depending on the rate of loading and temperature; it may act as a visco-elastic, elastic-plastic, pressure sensitive, or brittle material. Thus the validity of any proposed theory must be established through field tests.

Previous field tests that relate to the design problem have been reported by Croasdale [1, 2]. These tests are directly applicable to the design of certain structures, such as pile-founded structures in which the aspect ratio (structure diameter/ice thickness) is small. Unfortunately the tests cover only a narrow range of aspect ratios.

To develop additional data a field program to obtain large scale measurements of the crushing strength of ice was initiated in the Winter of 1972-1973. A portable test device was built and used to perform 38 field tests on a frozen fresh-water lake near Calgary, Alberta. Principal objectives of the test series were to establish the effect of aspect ratio (D/t) and indenter shape on the crushing strength of ice. Samples of the lake ice were collected and tested in the laboratory at the University of Alberta. This paper presents a summary of the test program. More detail is presented [3, 5, 6].

2. **TEST PROCEDURE**

2a. General

The field tests were performed at Eagle Lake, an irrigation lake 30 miles east of Calgary. Crystal studies revealed that the lake ice was vertical c-axis in some areas, and horizontal c-axis in others. To ensure availability of horizontal c-axis ice, a total of 15 test ponds were cut and seeded with snow. This effort was successful.

Tests were conducted on both the "natural" ice sheet and test ponds. Thin sections were cut from samples taken from each test location and the ice was classified according to c-axis orientation.

2b. Field Apparatus

The portable test unit consisted basically of a pair of hydraulic cylinders and a front "load face" and rear "reaction face". The load and reaction faces were detachable so the unit could be converted to any desired width. Five flat load faces of 1, 2, 4, 8 and 12 foot widths and three semi-cylindrical faces of 1, 2, and 4 foot diameters were used. For the 8 ft. and 12 ft. units, the two hydraulic cylinders were mounted as shown in Figure 1. For the narrower widths a single cylinder was centrally mounted. For each width of load face, the reaction face was at least one foot wider than the load face to ensure that the ice always failed on the front side. Maximum weight of the test unit, the 12 ft. configuration, was 22,000 lbs.
The hydraulic cylinders developed 2,000,000 lbs. of thrust each at the rated pressure of 7,000 psi. Maximum stroke of the cylinders was 24 in. Power for the cylinders was provided by a Ford V-8 gasoline engine which delivered 104 horsepower at 2,500 rpm. A Funk gear drive was mounted on the engine housing and two hydraulic pumps were attached to the drive. The flow from the pumps was variable between 0 and 33.5 gpm which corresponds to a maximum extension rate of 23 in/min for the cylinders. Thus, the power for each cylinder was from a common source but the rate of displacement of each ram could be controlled independently.

The test unit was supported by a 12 ft. high by 18 ft. wide gantry, also shown in Figure 1. The gantry was mounted on skids so that it could be towed from test location to test location. Two 10 ton chain hoists were used to raise and lower the unit. Attached to the gantry were a pair of "jack-up" legs which were used to support the weight of the gantry and test unit during the test.

2c. Field Test Procedure

Test procedure is described more fully [4]. For each test a rectangular hole was cut in the ice sheet, the gantry was towed over the hole and the test device was lowered until it was centered in the hole. The weight of the apparatus was transferred to the jack-up legs and the test device was leveled in the hole. Hydraulic pressure to the cylinders was increased until the load face was forced into the edge of the ice sheet, crushing the ice. For the tests performed in the test ponds, the hole was cut so that the rear face reacted against the thick natural ice while the front face acted against the thinner test pond ice.
A ditching machine was used to make a central cut for each hole, then a chain saw was used to cut the front and rear edges. Great care was taken while cutting the front and back faces to ensure that the sides of the hole were as smooth, vertical, and parallel as possible. A jig was made for the chain saw which greatly simplified the operation. Basically, the jig consisted of an adjustable height, freely rotating bar attached to a sled. The chain saw was attached to the bar and the cuts were made by pivoting the saw around the bar. The runners of the sled were guided by an aluminum bar nailed to the ice sheet.

Three types of load were used for the tests: impact, pre-pressured, and frozen-in. In the impact tests the test device was left hanging clear in the hole until the beginning of the test. Then the flow of hydraulic oil was set at a constant rate so the load faces were moving at the test displacement rate when they engaged the ice sheet. This type of test was discontinued because in two tests the ice failed behind the reaction face. For the pre-pressured tests the rams were gradually extended until the load face just touched the sides of the hole and a small hydraulic pressure, corresponding to approximately 100 psi in the ice sheet, was left on the system for 30 minutes. It was hoped that the ice would creep sufficiently to form a perfect seat for the load face. An alternate method of obtaining a perfect seat for the load face is to allow the unit to "freeze-in" overnight. This method was used for a number of tests and appeared to work quite well, although it is not certain that the nights in late February and March were cold enough to ensure complete freezing.

Constant displacement rate loading was employed during the field tests. A rate effect experiment indicated that 4 in/min, which corresponds to a strain rate of approximately .005 sec⁻¹, produced the maximum ice pressures. This displacement rate was used for the remainder of the tests.

For the 8 ft. and 12 ft. wide tests it was possible to adjust the two hydraulic cylinders so they advanced at approximately the same rate, but the hydraulic controls were inadequate for precise synchronization. This is not considered to be a serious problem because both rams advanced at constant rates and the test device was designed to allow considerable variation in displacement. Examination of the pressure traces shows that the pressure buildup on the two cylinders occurred at approximately the same time, although to different values; and there is no evidence that the failure progressed from one side to the other. The pressure in both cylinders drops off at the same time. Further, the highest pressures were measured in those tests where the synchronization was the poorest.

Measurements for the tests included hydraulic pressure, displacement between the load and reaction faces, displacement of the reaction face relative to the ice sheet, air temperature and ice temperature. Hydraulic pressure was measured with a Varian pressure transducer and recorded continuously on a Texas Instruments four-channel recorder. The pressure transducers were calibrated with a dead weight tester and are thought to be more accurate than the resolution of the recorder, ± 25 psi. Displacement between faces was measured with a Heliport helical potentiometer which was also calibrated to the resolution of the recorder which was ± .1 inches. A linear motion potentiometer was used to measure the displacement of the reaction face relative to a
stake in the ice sheet located five feet behind the face. Accuracy of
the measurement was ± .1 inch. In many cases, the reaction face dis-
placement measurement was unsuccessful because the surface ice behind
the plate spalled up and interfered with the potentiometer. For later
tests, this measurement was abandoned.

Ice temperature was measured by means of thermistor string which was
frozen in place at the edge of the test area. Since the thermistor
string was far removed from the test holes, it provided the temperature
distribution through the undisturbed ice sheet, to an accuracy of ± .5°F.
Near the edges of the test hole, the temperature distribution is quite
different because of the warming effect of the water in the hole.

2d. Laboratory Tests

Concurrent with the field tests, an extensive program of laboratory
testing was performed by Prof. Nuttall of the University of Alberta.
The results of the laboratory testing are presented in detail [5].
Only a summary of results is presented herein.

Five different tests were used to measure the strength of the ice. In
addition to the three compression tests illustrated in Figure 2,
Brazil tension tests and direct shear tests were performed. The uncon-
fined compression test, Type I, is illustrated in Figure 2a. The load-
ing was in the plane of the ice sheet, perpendicular to the columnar
crystals. The size of the sample was 4 x 4 x 8 inches. Failure
occurred by shearing between the crystals. The biaxial, Type II, com-
pression test is illustrated in Figure 2b. Again the loading is in the
plane of the ice sheet but the sample is confined between restraining
plates so the shear failure is forced to occur across the crystals. In
the biaxial, Type III test, illustrated in Figure 2c, the load is
applied in the plane of the ice sheet but confinement is in the verti-
cal direction. Thus failure can occur between the crystals. For both
biaxial tests, the sample size was 2-1/2 x 6 x 12 inches.

![Figure 2 Laboratory Compression Tests](image)

3. RESULTS

3a. Field Tests

A total of 38 field tests were performed; the highlights of the tests
are summarized in Table 1. No measurements were made during the first
test and it is therefore omitted from the table. In three of the
tests the first failure occurred at the reaction face. In these cases,
the ice pressure for both the front and rear faces were computed and
a double entry is included in the table; for example, 24F and 24R cor-
respond to the front and rear values. The column for indenter width
refers to the width of the load face. Load type refers to the three
loading methods used, impact (I), pre-pressured (P), and frozen-in (F). "Ice type" refers to the crystallographic description of the ice tested. Natural ice with vertical c-axis is designated N-V; natural ice with horizontal c-axis is N-H; and the test pond ice which was all of the horizontal c-axis type is labeled TP-H.

"Ice thickness" is an adjusted measurement of the ice sheet thickness. The adjustment was necessary because in some areas the natural ice sheet was covered by a layer of snow which was infiltrated by water and then frozen. Based on the arbitrary assumption that "snow" ice is only half as strong as the underlying clear ice, one half of the snow ice thickness was added to the clear ice thickness to produce an "effective" thickness. The adjustment was generally small, the snow ice was typically less than 8 inches thick. No adjustment was necessary for the test pond ice because they were kept free of snow for the duration of the test program. Ice temperature is the average ice temperature through the thickness of the undisturbed ice sheet and is based on a frozen-in thermistor probe. The displacement rate noted is between the front and rear load faces, aspect ratio is indenter width divided by ice thickness. Ice strength is the crushing strength or "effective ice pressure" which is calculated as the maximum load during the test divided by the product of indenter width and ice thickness.

Since the reaction face was at least one foot wider than the load face in all tests, the occasional failure of the ice sheet behind the reaction face was surprising. For tests 4 and 5, which were conducted in the natural ice sheet, the failure was probably due to the impact loading. The sides of the test hole can never be perfectly parallel; therefore, if the front load face reached the ice first, the test unit would rotate until the front face was firmly seated and the edge of the reaction face would then be forced into the ice sheet. This hypothesis is verified by the fact that a reaction face failure never occurred in the natural ice sheet for the pre-pressured or frozen-in tests.

At first glance, the reaction face failures of tests 24, 35 and 36 seem even more surprising; the rear of the test hole was in the natural ice sheet with a thickness of approximately 30 inches, while the front face was in the 18 inch thick test pond. However, through all the field tests the natural ice was found to be substantially weaker than the test pond ice so that these failures may simply have occurred at the point of least resistance. Also, the centerlines of the natural ice and the test pond do not coincide; and a non-uniform stress distribution in the natural ice could cause progressive failure, beginning at the upper surface, at a reduced load. The results from tests 35 and 36 are considered to be valid data points for both the front and rear face because massive failure occurred on both faces even though the gross movement was limited to the reaction face. The pressure computed for the front face in test 24 must be considered only a lower bound to the strength of the test pond ice because there was no apparent failure there.

A sketch of a typical failure pattern is shown in Figure 3. It closely approximates the double-wedge failure predicted by Croasdale [1]. Shear planes are created in the ice sheet at angles of approximately 45° from the vertical. In one case, the entire lower wedge was recovered intact after the test, but typically, the failure wedges were fractured by a series of horizontal cracks as shown in the sketch.
3b. Laboratory Tests

Results of the compression tests are shown in Figures 4, 5, and 6. For the test pond ice, sufficient tests were performed to establish the effect of temperature on the strength. As shown in Figure 4, the unconfined compressive strength increases by a factor of six as the temperature is decreased from -1°C to -20°C. The biaxial Type II strength is much higher and even more temperature dependent than the unconfined strength.
The results from the natural ice are not as clear; Figure 5 shows the results for the natural horizontal c-axis ice. Strength values are of the same order as for the test pond ice and there is a strong dependence on temperature. Strength values from the biaxial Type III tests are approximately the same as the unconfined strengths. Test results for the natural, vertical c-axis ice are shown in Figure 6.

Brazil tests were performed on test pond ice and natural, vertical c-axis ice. The results are summarized in Figure 7. The temperature range covered was rather narrow, but it appears that the test results are not strongly affected by temperature. The test specimen was 3-1/2 inches in diameter and 1-3/4 inches thick. The direct shear tests [6] in Figure 8 show that ice is very sensitive to confining pressure.

![Figure 7 Brazil Test Results](image)

![Figure 8 Peak Shear Strength Envelope for Test Pond Ice](image)

4. DISCUSSION OF RESULTS

4a. Size Effect

Results of the field tests have been separated according to the orientation of the ice and plotted in Figure 9 and 10. The maximum ice pressure is shown on the vertical scale and the size effect ratio D/t, is shown on the horizontal scale. Three of the points in Figure 9 were front face pressures in tests where the primary failure occurred at the rear face. These points have been labeled with arrows to indicate that they are lower bounds to the ultimate pressure. The results for the horizontal c-axis ice were obtained primarily from test pond data; only tests 7, 8 and 10 on natural ice are included but these three points fit the test pond data. Tests 4 and 5 were omitted because the test hole was not cut properly. Test 9 was a creep failure and is therefore not included. All of the points in Figure 10 were obtained in natural, vertical c-axis ice. From these figures, two important conclusions may be reached. First, a pronounced size effect exists for both types of ice; i.e., the strength of the ice decreases with increasing aspect ratio. Second, the horizontal c-axis ice is much stronger than the vertical c-axis ice.
The theoretical size effect derived by Morgenstern [7] is shown as a solid line on Figures 8 and 9. For purposes of this comparison, the horizontal c-axis test results have been normalized by 593 psi and the vertical c-axis results have been normalized by 281 psi in order to make the ratio equal 1.0 for large aspect ratios. In both cases, comparison with the theoretical curve is good.

Field data can also be normalized by compressive strengths measured in the laboratory, but the resulting comparison with the theoretical curve is very poor. Reconciliation of this discrepancy will require a more sophisticated failure model that incorporates the anisotropic, pressure sensitive behavior of the ice.

4b. Comparison of Field and Laboratory Results

The laboratory tests show both in compression and Brazil tests that the vertical c-axis ice was much stronger than the horizontal c-axis ice; the field tests show the opposite. It is probable, but unproven, that the explanation for this contradiction is related to the relative crystal size of the two types of ice. It seems reasonable that the failure strength is strongly affected by the number of grain boundaries in the test specimen. In some cases, the vertical c-axis specimens were cut from a single crystal; and, therefore, high strengths are not surprising. Some support for this line of reasoning is given by Weeks [8]. There it is shown that the unconfined compressive strength is reduced as the cross-sectional area of the test specimen is increased. It is felt that this same relation would hold if "number of crystals" were substituted for "specimen area".

4c. Temperature Effect

In the previous section it was shown that in laboratory tests, ice strength increases dramatically with decreasing temperature. There is evidence which indicates this is not true in the field [8, 9]. The field data presented herein is insufficient to resolve the issue; but it is significant that the highest pressures measured occurred in warm ice.
Highest loads on a structure appear to be caused by ice failing in a ductile mode. This sort of behavior is illustrated in Figure 11. The pressure-time plot is typical for warm ice loaded at a moderate strain rate. There is evidence of extensive plastic deformation. The plot is for Test No. 38, the last one of the program, March 15. At that time the ice sheet was rapidly melting and appeared to be in a very deteriorated condition. Personnel were seriously concerned about the safety of remaining on the ice sheet. Yet the peak pressure measured was the highest of the entire test program.

In contrast, Figure 12 shows the very brittle pressure-time behavior typical of cold ice. Average ice temperature for this test was 2°F. Pressure drops immediately after reaching the first peak, subsequent pressure peaks are much lower. It is easy to visualize that cold ice would fail against a large structure in series of local, brittle failures resulting in a relatively low effective pressure.

4d. Strain Rate Effect

The effect of displacement rate on the strength of ice was studied in a series of three tests conducted with a four foot, flat indenter. A representative strain rate is obtained when the displacement rate is
divided by a characteristic length. For indentation tests at small D/t ratios, the primary mode of failure is confined flow around the indenter and the indenter width is the characteristic length. For tests at large D/t ratios, the mode of failure changes and the ice sheet thickness appears more appropriate for characteristic length. The tests in this series were all conducted at D/t ratios less than 2 and the strain rates shown in Figure 12 are therefore based on the indenter width.

Results of this test series are not definitive because of insufficient data points and variations in ice properties. Nevertheless, the data illustrate an important aspect of ice behaviour. At the low strain rates, creep failure occurs at rather low pressures. At high strain rates, brittle failure occurs at intermediate pressures. Highest pressures result from intermediate strain rates for which a plastic failure occurs. It is to be expected that strain rate causing maximum ice pressure would be strongly influenced by the temperature of the ice.

4e. Effect of Indenter Shape

In order to check the effect of indenter shape on the strength of ice, a one foot round and a 2 foot round load face were tested. A large auger was used to drill holes in the ice and the test holes were cut so that the front face split the round hole. The fit was not perfect; a 14 inch diameter hole was cut for the 12 inch load face and a 23 inch diameter hole was cut for the 24 inch load face. The units were left to freeze-in overnight, but the tests were conducted late in the season so the gaps between the load face and ice sheet were only partially frozen.

It is generally accepted that a round indenter should fail the ice at a lower load than a flat one and this trend was observed. The data points are very close to the curve for a smooth cylinder and thus tend to confirm the theory. However, in view of the scatter in the flat indenter data, the effect of indenter shape cannot be positively established from two tests.

4f. Effect of Type of Loading

Impact loadings were abandoned early in the program because of reaction face failures. The remainder of the tests were either frozen-in or pre-pressured. There is some evidence that pre-pressuring causes higher
strength values but an exact comparison is difficult because of scatter in the data and because of uncertainty as to the extent of freezing. However, it seems significant that the maximum value for each D/t ratio was obtained in a pre-pressured test.

4g. Pressure Variation Through Thickness of Ice Sheet

For the final two tests of the program, the two foot indenter was equipped with individual, load-sensing elements. The elements were 6 inch by 24 inch metal plates which were equipped with strain gages and calibrated on a load frame at the University of Calgary. As shown in Figure 14, the indenter was positioned in the test hole so that the thickness of the ice sheet was covered by elements 3, 4 and 6.

The purpose of the experiment was to determine if the upper and lower ice wedges illustrated in Figure 3 reach their peak pressures simultaneously, or fail in sequence. Figure 14 shows the pressure time plot for the three elements along with the average ice pressure calculated from hydraulic pressure. It is clear that the pressure build-up was uniform through the thickness of the ice sheet and that failure occurred simultaneously. This result is probably related to the ductile nature of the ice in this particular test.

![Figure 14: Vertical Distribution of Pressure](image)

Average peak pressure of the plates is slightly higher than the hydraulic based pressure, but the agreement is considered excellent. The hydraulic based pressure is thought to be the more reliable because a line load was used to calibrate the elements and the ice exerts a uniform load.

5. ACKNOWLEDGEMENTS

This project was sponsored by the Arctic Petroleum Operator's Association (APOA) and was funded by a number of companies. ESSO Resources Canada, Ltd. acted as operator and provided all personnel and equipment. Without the ideas, skills and dedication of Mr. A. S. McLatchie, Mr. R. E. Hedley, and Mr. T. R. Nalder, it would have been impossible to make the necessary equipment modifications and maintain a very tight test schedule. Also, the advice and counsel of Mr. K. R. Croasdale and Mr. C. Duncan are appreciated.
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UNIAXIAL COMPRESSION TESTING
OF ARCTIC SEA ICE

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ABSTRACT

In the past three years since EPR built its cold laboratory in Houston, many strength tests have been conducted on arctic sea ice as part of ice mechanics research. The majority of these tests were uniaxial compression tests under constant strain rate. This paper describes the procedures that we used in sample preparation, testing, and data reduction. Some of the experiences that we learned from these tests are also presented.

The tests were conducted with cylindrical ice samples on an MTS test machine with a closed-loop, servo feedback system. Constant strain rate was controlled by an extensometer directly attached to the sample. Successful tests have been conducted with strain rates ranging from as low as $3 \times 10^{-7}$ sec$^{-1}$ to as high as $10^{-2}$ sec$^{-1}$. LVDT's were used to measure the lateral deformation in two directions, parallel and perpendicular to the growth direction respectively, so that Poisson's ratios in these two directions can be computed. The deformation in these two directions was anisotropic for columnar ice.

The effects of using compliant flexane platens were also investigated. At slower strain rates (say $10^{-4}$ sec$^{-1}$ and below) the use of flexane platens was not necessary. Furthermore, the use of flexane platens often introduced erroneous readings on the lateral LVDT's in the beginning stage of a test.

Large data scatter is often encountered in testing sea ice samples. This paper shows that one of the causes is the basic inhomogeneity of the sea ice crystalline structure.
Introduction

The compressive strength of sea ice is a significant design consideration for Arctic Offshore Structures and has become a subject of interest for decades. Sea ice is a very complex material, whose structure is affected by conditions of growth, environment and history. Thus ice samples obtained from different locations may not be structurally the same and consequently may not have the same strength when tested. Furthermore, due to the high homologous temperature under which it normally exists, the strength is temperature dependent and very sensitive to the rate of loading. Under different temperature or loading rate, the mechanical behavior may be elastic, viscous, plastic or their combinations. In addition, sea ice often has large grain sizes, in the order of millimeters or even centimeters. Thus the sample size may also become an important parameter in the results of strength tests.

Having realized the complex nature of the compressive strength of sea ice, it is not surprising that test results reported in the literature often cannot be compared directly either due to insufficient information in the description of ice type, test environment, sample preparation, test method, etc., or due to the basic differences in these parameters. Employment of existing data to specific field applications still involves too much uncertainty and is not practical. To bridge the gaps in experimental data, and more fundamentally to advance in the understanding of sea ice mechanics, it is evident that more work is needed to be done.

In 1978, Exxon Production Research Company built a cold laboratory in its Houston facility, part of which was designated for ice strength testing. Since then, many compression tests on sea ice samples have been conducted. This paper is primarily written to discuss some of the experiences that we learned from them.

Sample Preparation

Sea ice sample blocks were stored in plastic bags in the storage room at a temperature close to -35°C. One day prior to the tests, a designated block was taken into the test room, where the temperature was maintained at the test temperature. The ice block was rough cut on a band saw into prisms and left in the room overnight. In most cases, the time was sufficient for the ice to achieve thermal equilibrium with the ambient temperature by the next morning. Occasionally, the ice might still be 0.5°C colder than room temperature by then. This difference in temperature, however, was considered to be insignificant and was not investigated further.
On the test day, the ice prisms was first machined on a lathe equipped with a special cutting tool to a nominal diameter of 6.92 cm (2.725\textquotedbl{}), then the ends were finished on a milling machine for a nominal sample length of 14.60 cm (5.75\textquotedbl{}). This provided a length over diameter ratio of 2.1 and was considered to be sufficient for the sample to be under a uniform uniaxial state of stress in its middle section under compression. All samples were machined with their sample axes perpendicular to the direction of growth. Since the end condition may have a significant effect on test results, the sample ends were carefully examined before testing. The criterion of acceptance was that on each end surface, the variation of height should be within 0.13 mm (0.005\textquotedbl{}). If the height variation exceeded this value, then the end would have to be refinished or the sample would be discarded. This tolerance was equivalent to a maximum inclination of the end surface from the horizontal plane by 0.10\textdegree. Since the test machine has a spherical joint on the upper load platen, this inclination was assumed acceptable. In most cases, the samples were tested on the same day when they were made. If they could not be tested on the same day, they were left in the test room overnight.

The Test Setup

The test machine used was model MTS810.15 manufactured by MTS Systems Inc. as shown in Fig. 1. This machine has a 100,000 kgf (220,000 lb) capacity load frame with a closed-loop, hydraulic feed back control system that can use either load, or strain or actuator displacement as the control parameter. The strain control is achieved by monitoring and controlling the output of an extensometer directly attached to the sample, thereby preventing machine stiffness and the stiffness of the load platen assembly from affecting the sample deformation. A constant strain rate test can be achieved this way. The extensometer was attached to the ice sample using rubber bands. Its gauge length was 25.4 mm (1\textquotedbl{}). Extenders have been used in some tests to make its gauge length either 50.8 mm (2\textquotedbl{}) or 101.6 mm (4\textquotedbl{}). The lower load platen was attached to the actuator while the upper load platen was connected to the load frame through a load cell. The upper load platen had a built-in spherical joint to compensate any deviation of the sample ends from the horizontal plane. In some of the tests, a set of compliant platens were also inserted between the sample and the load platens.

To measure lateral deformation of the sample under axial compression four LVDT's were installed around the mid-section of the sample to form two orthogonal pairs. One pair measures the lateral expansion in a direction parallel to the growth direction (the original vertical direction of the ice sheet) and the other pair measures that in the perpendicular direction (the original horizontal direction of the ice sheet) so that any anisotropy of the
lateral deformation could be detected. The LVDT's were installed on a frame which was attached to the lower load platen. Fig. 2 shows the arrangement of the LVDT's and the extensometer on the sample.

During a test, output from load and strain were recorded on an analog XYY plotter. At the same time, a Hewlett-Packard digital data acquisition system centered on a HP 9825 desk computer was also employed to record seven channels of data on disk including load, strain, platen displacement and the four LVDT outputs. These data could be analyzed later using an analysis program to give stress, strain, stiffness, Poisson ratio and any other desirable information.

Discussion of Test Results

Fig. 3 shows typical stress-strain curves under constant strain rate condition for the so-called oriented columnar sea ice obtained from the Prudhoe Bay area, which had a columnar crystal structure with a dominating orientation in the horizontal plane for the c-axes. The samples were machined with their axis parallel to the dominating c-axis orientation and were referred to as 0° samples. When the strain rate was less than $10^{-5}$ sec$^{-1}$, the deformation was always ductile with the stress gradually approaching an asymptotic value. When the strain rate was increased to near $10^{-4}$ sec$^{-1}$, the ice started to show a strain softening characteristic after the maximum stress was achieved. This strain softening behavior became more pronounced as the strain rate was farther increased. Finally, at about $10^{-3}$ sec$^{-1}$, brittle fracture started to occur. The strength ceased to increase with strain rate and became more scattered due to the brittle nature of sample failure. Some of the strength data of those tests have been presented elsewhere [1] and
will not be repeated here. Instead, in what follows, discussions will be focused on topics that have not been addressed before.

![Graph A](image)

**STRAIN RATE** $= 10^{-3}$ SEC$^{-1}$  
**TEMP.** $= -18^\circ$C

![Graph B](image)

**STRAIN RATE** $= 3 \times 10^{-5}$ SEC$^{-1}$  
**TEMP.** $= -10^\circ$C

![Graph C](image)

**STRAIN RATE** $= 10^{-5}$ SEC$^{-1}$  
**TEMP.** $= -10^\circ$C

![Graph D](image)

**STRAIN RATE** $= 10^{-6}$ SEC$^{-1}$  
**TEMP.** $= -13^\circ$C

**Fig. 3.** Typical stress-strain curves of constant strain rate tests

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**Poisson's Ratio**

For a long time, little attention had been paid to the lateral expansion of an ice sample under compression. Poisson's ratio was assumed to be isotropic even for columnar ice and was assigned a value near 0.3 for elastic analysis and 0.5 for plastic analysis. However, because of the anisotropic crystalline structure, it was suspected that Poisson's ratio may be anisotropic. Fig. 4 shows the axial strain (positive in compression) and the two lateral strains (positive in tension) versus time for several tests. Lateral strains A and B were in the horizontal and the vertical direction of the ice sheet respectively. From these curves, it is apparent that lateral deformation was anisotropic. The ice deformed much more in the direction perpendicular to the crystal columns (horizontal direction) than in the direction parallel to them (vertical direction). For the majority of the tests, Poisson's ratio in the horizontal direction ranged from 0.8 to 1.2 while that in the vertical direction from 0 to 0.2. This phenomenon suggests that during ductile deformation gliding of grain boundary and between platelets is the main deformation mechanism.
The results obtained here can be used to explain the plane-strain compression test results of Frederking [2]. Frederking conducted plane-strain compression tests on rectangular samples of columnar freshwater ice with the loading direction perpendicular to the crystal columns. He found that the plain-strain compression strength was much higher than the uniaxial compression strength if the lateral deformation in the direction perpendicular to the crystal columns was prohibited. However, if the direction of lateral restriction was parallel to the crystal columns, no apparent increase above the uniaxial compressive strength was observed. This was because the confinement in the latter case did not have a significant effect, since the ice did not expand much in that direction under uniaxial loading anyway.

Compliant Platens

The use of compliant platens in compression tests was originally practiced in rock mechanics and was introduced to ice testing by Hayes and Mellor [3]. The advantage of using compliant platens is to reduce the adverse effect of poor sample end conditions and that of "short samples" (length/diameter ratio less than 2). However, it also introduces
certain disadvantages, one of which is the difficulty in controlling strain rate in a conventional test machine because compliant platens make the machine system considerably softer. Therefore, in the IAHR recommendations on standardization of ice testing [4], compliant platens are recommended for field tests but not for laboratory tests because in the laboratory, high quality samples can be produced and tests can be conducted with good accuracy. In our test programs, compliant platens, made of flexane rubber cast in an aluminum ring, were used in some of the tests. These flexane platens had the same dimensions as was described in [3] and were one of the original sets used by those investigators.

Fig. 5. Stress-strain curves

\[
\text{Strain rate} = 10^{-5} \text{ sec}^{-1} \quad \text{FP = flexane platens were used}
\]

\[
\text{Temperature} = -14^\circ C \quad \text{No FP = no flexane platens were used}
\]

Fig. 6. Initial erroneous output of lateral strain due to flexane platens

Fig. 5 shows the stress strain curves for several samples tested first with flexane platens, then without them, or vise versa. The strain rate was $10^{-5}$ sec$^{-1}$. All samples showed that flexane platens did not have any noticeable effects on the sample response, which is generally true for strain rates of $10^{-4}$ sec$^{-1}$ and under. However, using of the flexane
platens often created an undesirable side effect, i.e. erroneous lateral deformation during the early stage of a test. One extreme case is shown in Fig. 6, while a less dramatic case is shown in Fig. 4C. This was believed to be caused by the small rotation of the sample when the sample was settling on the soft flexane platens at the inception of a test, which created an effect similar to an expansion of the sample at the LVDT’s.

It is therefore concluded that at moderate strain rates, the use of flexane platens were not necessary, which is consistent with the recommendation of the IAHR working group.

**Inhomogeneity of Ice Samples**

One of the major problems that has constantly puzzled investigators in ice testing, sea ice testing in particular, is the wide scatter of strength data. Of course, many parameters in the testing environment may contribute to the inconsistency of test data. However, the most important cause of it is the inhomogeneity of the ice itself. In contrast to most other materials, ice grown under normal conditions usually has large crystals or grains in millimeters or even centimeters. Thus unless a test sample is very large, the number of grains contained in a cross section is very limited. For sea ice, this is further complicated by the existence of brine pockets either between the crystalline platelets or between grain boundaries. Some brine pockets may be close to each other and form brine channels. Thus the material distribution in sea ice is inhomogeneous even in the scale of tens of centimeters. In addition, inhomogeneity in the crystalline structure is also a common fact. For instance, samples cut from a block of oriented sea ice next to each other exhibited a significant difference in strength when tested. A closer examination of the crystalline structure revealed the secret. Fig. 7 shows the relative locations of one set of these samples, their strength values and photographs of horizontal thin sections made after they were tested. These samples were nominal 0° samples, but in reality all the average c-axes orientations deviated from the sample axes by a certain amount. However, sample 4 deviated much more compared to samples 1, 2 and 3 and its strength was only about 50% of the average of 1, 2 and 3. No thin section was made for sample 5, but we believe that its crystalline structure was likely to be similar to sample 4.

This example illustrated that columnar sea ice, particularly oriented columnar sea ice, may be inhomogeneous and its crystalline structure may vary from sample to sample. Strength tests may therefore result in large data scatter. To assess the strength of this ice, test data must be carefully examined and interpreted.
Fig. 7. Variation of strength due to inhomogeneity of sea ice structure
References


FRACATURE TOUGHNESS OF SEA ICE

- IN-SITU MEASUREMENT AND ITS APPLICATION -

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ABSTRACT

In-situ fracture toughness tests of sea ice were carried out by means of three-point bending technique at Saroma lagoon, which is located at the most northern island in Japan. Critical stress intensity factor $K_{IC}$ at about $-2^\circ C$ was obtained for wide range of loading rates, since the fracture always occurred within the elastic portion of the load versus load-point displacement curve.

The $K_{IC}$ values showed almost constant if stress intensity factor rate $\dot{K}_I$ was less than $10^2$ kPa$\sqrt{m}$/sec, and decreased with increasing the $\dot{K}_I$ if the $\dot{K}_I$ exceeded $10^2$kPa$\sqrt{m}$/sec.

It was also shown that the $K_{IC}$ values have close relationship with sea ice structure. Calculated flaw sizes based on the linear elastic fracture mechanics concept coincided fairly well with subgrain sizes where the notch-tips were located in the specimens. Moreover, those flaw sizes were shown to be independent on the $\dot{K}_I$. Thus the relationship between the $K_{IC}$ value and inverse square root of subgrain size $d$ was tentatively determined as follows.

$$K_{IC} = 135 \exp(-1.9/\sqrt{d})$$

where $K_{IC}$ and $d$ are given in the unit of kPa$\sqrt{m}$ and mm, respectively.

From the viewpoint of fracture mechanics, dynamic similarity law (concerning elastic force) was examined for the "model" ice to be used in refrigerated towing tank laboratory. For the cracked body, it is important that the stress state in the vicinity of crack tip in model ice should be scaled down correspondingly from that in "prototype" ice, since natural sea ice contains crack-like flaws.
INTRODUCTION

Recent extensive increases in offshore exploitation and production of hydrocarbon resources in frozen sea area cause ice breaking ships and fixed structures to work under severer environmental conditions. For designing or analysing the safety assessment of the structures, it is primarily important to estimate the interaction forces between structures and ice at the site where it is practically interested.

On the other hand, many towing tank experiments for the models of structure have been performed in the ice tank without imposing cost burden for testing large-scale prototype structures in the actual frozen sea. Thus, the laboratory data could be applied to analyse the prototype structures, if both of the geometry of the structure and the mechanical properties of ice were properly scaled down correspondingly.

In order to get accurate results from the model tests, it is important that geometric, kinematic, and dynamic similarity must be satisfied between model and prototype [1]. The geometric and kinematic similarity will be satisfied easily, though, the dynamic similarities are generally impossible to meet simultaneously.

For the dynamic similarity, the scaling down practice due to Froude's law (concerning inertia force and gravity force) has been well established. Moreover, Cauchy's law (regarding with inertia force and elastic force) has been paid attention for the model tests in the frozen towing tank laboratory, since the fracture phenomena of model ice have also properly to be scaled down from those of the prototype ice [2], [3]. The latter condition deduces that both Young's modulus E and fracture strength $\sigma_f$ of the model ice must be scaled down by the amount of geometric scaling factor $\lambda$, and that the ratio of $E/\sigma_f$ must be greater than 2000 as the typical value of the natural sea ice.

According to the above mentioned analysis, investigations on the mechanical properties of synthetic ices as model ice have been extensively made for satisfying easily the $E/\sigma_f$ value [4].

In this paper, the fracture phenomena of sea ice was analysed by means of the fracture toughness tests. And the similarity law concerning the fracture of ice was examined by the concept of linear elastic fracture mechanics. The $K_{IC}$ value of the model ice should be adjusted to have the value corresponding to the fracture mechanical scaling method.

EXPERIMENTAL PROCEDURES

Sea ice sheet of more than 45cm in thickness was usually grown in the winter season at Saroma lagoon. Salinity of water in the lagoon was little bit dilute if compared with ordinary sea water, though, frozen ice showed quite uniformity in its thickness.
Preceding to the fracture toughness tests, crystallographic structures of the ice were surveyed on the thin sectioned samples under polarized light, and the average subgrain sizes were measured. The diameter of columnar grains usually increased with increasing the ice thickness. The subgrain size was defined as that the area which is bounded by the directionality of brine cells within a columnar grain. The subgrain sizes were also increased with increasing the ice thickness. At test site I, the average subgrain sizes changed from about 3mm to 22mm at the top portion and at the bottom portion of the ice sheet, respectively. But the test site II, the average subgrain sizes were almost constant as about 40mm from the middle to the bottom portion of the ice sheet.

While the temperature of sea water was fairly constant at about -2°C during the test period (early in February to mid-March), the atmospheric temperature was variable. Therefore, the tests were performed at those times when the atmospheric temperature was between -1°C and -3°C.

The portion of snow ice was removed and rectangular parallelepiped specimens sizing 20cm x 40cm x 170cm were then prepared in conformity with the method described in ASTM E399 code[5]. The specimens were cut out from the columnar grained portion of the ice sheet so that the direction of long axis of the columnar grains was always perpendicular to the 20cm x 170cm surface of the specimen. Saw-cut edge-notch was introduced normally at the middle of the same surface. The notch depth was, then, carefully chosen so that the notch-tip was situated at the proper position with respect to the desired ice structure. Three series of specimens were prepared such as the average subgrain sizes were 40mm, 22mm, and 3mm, respectively. The notch-tip was reshaped by a razor blade in order to increase the notch acuity.

The tests on unnotched specimens were also made to obtain the fracture stress. All the tests were performed on a loading apparatus, with loading span length being 160cm, which was constructed on the frozen sea. The details were given in elsewhere [6],[7].

Both load versus load-point-displacement curves and load versus time curves were recorded on recording apparatus placed in a cottage. An abrupt load drop within the elastic portion was always observed in the load versus load-point-displacement record at the instance of the fracture initiation. And critical stress intensity factor $K_{IC}$ was calculated from the following equation derived by Brown and Srawley [8]

$$K_{IC} = \frac{3P_f}{2B\sqrt{a}}f\left(\frac{a}{W}\right)$$

$$f\left(\frac{a}{W}\right) = 1.93 - 3.07\left(\frac{a}{W}\right) + 14.53\left(\frac{a}{W}\right)^2 - 25.11\left(\frac{a}{W}\right)^3 + 25.80\left(\frac{a}{W}\right)^4$$  \hspace{1cm} (1)
where $S$ is loading span length, $a$ is the notch length, $B$ is the specimen thickness, and $W$ is the specimen width. The $k_I$, i.e., increasing rate of the stress intensity factor with respect to time, was also calculated according to equation (1), using the loading rate $P$.

**TEST RESULTS**

The $K_{IC}$ values were plotted together with the three series of tests in Fig. 1 as a function of the $k_I$. The $K_{IC}$ values seem to be almost constant until the $k_I$ approaches $10^2$ kPa m/$\sqrt{\text{sec}}$, and if this $k_I$ is exceeded the $K_{IC}$ values decrease with increasing the $k_I$. Figure 1 also indicates that the $K_{IC}$ values are largely influenced by the notch tip locations, namely the subgrain sizes of the sea ice.

![Fig. 1 Relationship between stress intensity factor $K_{IC}$ and rate of stress intensity factor $k_I$ for sea ice at $-2^\circ\text{C}$.

In order to satisfy the small scale yielding condition, the notch length should be longer than $2.5(K_{IC}/\sigma_y)^2$, where $\sigma_y$ is uniaxial yield strength of sea ice [5]. While the typical notch length is about 8cm in this study, the required notch length is about 10cm, when it is calculated using the fracture strength instead of the yield strength, since the yield strength could not be obtained in this study because the fracture always initiated within the elastic portion of the load versus load-point-displacement record. If the required notch length were calculated from the actual yield strength, it could be much shorter than 10cm. Therefore, the typical notch length would satisfy the requirement.
DISCUSSION

Fracture Toughness and Subgrain Size

The plane on which the brine cells are arranged seems to be a crack [6]. Thus, it is needed to investigate the relationship between the fracture toughness values and the subgrain sizes.

Due to the concept of linear elastic fracture mechanics, there exists a relationship between the critical stress intensity factor $K_{IC}$, the critical fracture stress $\sigma_{cr}$, and the flaw size $a$, as follows,

$$K_{IC} = \sigma_{cr}\sqrt{\pi a}$$

(2)

The critical stress for the initiation of fracture was obtained by the three-point bending test on the unnotched specimens which have the same specimen geometry as the fracture toughness specimens, but the saw-cut notch was not introduced. The $\sigma_{cr}$ was calculated following equation,

$$\sigma_{cr} = \frac{3 P_f S}{2 B W^2}$$

(3)

where $P_f$ is the fracture load. The $\sigma_{cr}$ was plotted in Fig. 2 as a function of $K_I$. For the purpose of comparison between $\sigma_{cr}$ and $K_{IC}$, $K_I$ was calculated under assumption that the unnotched specimen has an imaginary notch of 8cm depth.

![Fig. 2 Relationship between $\sigma_{cr}$ and $K_I$ for the sea ice at about -20°C](image)

360
The flaw size was calculated according to the equation (2), using the best fit lines for $K_{IC}$ given in Fig. 1 and the $\sigma_{cr}$ values given in Fig. 2. The estimated flaw sizes for the three series of the tests were plotted in Fig. 3, against the $K_I$. It is clearly seen from Fig. 3 that the flaw sizes are almost independent on the $K_I$ and seem to have good correlation with the subgrain sizes.

![Calculated flaw sizes](image1)

**Fig. 3** Calculated flaw sizes which are predicted to be existed in the sea ice

The calculated flaw sizes were plotted in Fig. 4 as a function of the observed sugrain sizes. The slope of the line is slightly less than the unity, but a good linear relationship was observed between the flaw size and the subgrain size.

![Relationship between calculated flaw sizes and observed sugrain sizes](image2)

**Fig. 4** Relationship between calculated flaw sizes and observed sugrain sizes
Thus, the \( K_{IC} \) values were plotted in Fig. 5 as a function of inverse square root of the sugrain size \( d \). Referring Fig. 5, the relationship between the \( K_{IC} \) and the \( d \) was tentatively determined as follows,

\[
K_{IC} = 135 \exp \left( -1.9/\sqrt{d} \right)
\]

where \( K_{IC} \) and \( d \) are given by the unit of kPa√m and mm, respectively.

In Fig. 5, the same relationship for pure ice was also plotted with dotted line, using the results available in literature [9],[10],[11]. The relationship was also determined as follows,

\[
K_{IC} = 159 \exp \left( -0.59/\sqrt{d} \right)
\]

The slope of the line for the sea ice is steeper than that for the pure ice.

![Figure 5: Relationship between \( K_{IC} \) and inverse square root of grain size for sea ice and pure ice](image)

**Fracture Mechanical Scaling Method**

There exists difference between the law of similarity for the fracture phenomena and that for the elastic deformation.

Consider first two plates containing stress concentrators shown in Fig. 6(a). The "model" plate including the notch is scaled down by the geometric scaling factor \( \lambda \) from the "prototype" plate. When the plates are stressed, the maximum stress \( \sigma_{max} \) is given by equation (6),

![Diagram of two plates](image)
\[ \sigma_{\text{max}} = \alpha \sigma \]  

(6)

where \( \alpha \) is the stress concentration factor and \( \sigma \) is the applied stress. In order to be the same value for the \( \sigma_{\text{max}} \) in both model and prototype plate,

\[ \sigma_p = \sigma_m \]  

(7)

since \( \alpha \) depends only on the ratio of dimensions of notch and plate (hereafter subscripts \( p \) and \( m \) denote the prototype and the model, respectively).

Fig. 6 Difference of similarity law between strength of materials approach and fracture mechanical approach

On the other hand, if the plate contains crack-like flaws instead of the stress concentrator, the cracked plate breaks at lower stress level if compared with the notched plate. Thus, the fracture phenomena for the cracked substances must be handled by the fracture mechanics approach rather than the strength of material approach. The stress (or strain) state in the vicinity of the crack tip can be represented by the parameter; stress intensity factor, as follows,

\[ K_I = \sigma \sqrt{\pi a} f(a/W) \]  

(8)

where \( K_I \) is the stress intensity factor for plane strain condition, \( \sigma \) is the applied stress, \( a \) is the crack length, and \( f(a/W) \) is nondimensional function depending only on the ratio of crack length \( a \) and width of plate \( W \).

Consider next the model plate and the prototype plate shown in Fig. 6(b). Both of the geometry of plate and the crack length are again correlated by the geometric scaling factor \( \lambda \). In order to have the same value at the crack tip for the \( K_I \), the applied stress must satisfy the following condition,

\[ \sigma_p = \sigma_m / \sqrt{\lambda} \]  

(9)
In order to satisfy the condition given by equation (9), the ratio of externally applied forces on the prototype plate \(F_p\) and on the model plate \(F_m\) is \(\lambda^{3/2}\), while the ratio of those forces is \(\lambda^2\) when the condition given by equation (7) is considered.

For the model test in the frozen ice tank laboratory, the dynamic similarity law must be also satisfied simultaneously. The external force acting on the model ice sheet must be scaled down from that on the prototype ice sheet correspondingly. For example, the force ratio \(F_p/F_m\) is \(\lambda^3\), as far as the gravity force is concerned.

For the fracture initiation phenomena, Atkins [2] has already proposed a fracture mechanical scaling factor satisfying the dynamic similarity law,

\[
\frac{(K_{IC})_p}{(K_{IC})_m} = \lambda^{3/2}
\]  

(10)

In equation (10), the crack size in model ice was taken to be scaled down by the factor of \(\lambda\) from that in the prototype ice.

As mentioned earlier, the crack size existing in the sea ice is proportional to the subgrain size. And the \(K_{IC}\) value of the sea ice has a close relationship with the subgrain size. Therefore, the ratio of the \(K_{IC}\) values of the prototype ice and model ice is once determined, the model ice which has desired \(K_{IC}\) value will be obtained by adjusting the subgrain size according to equation (4).

The results were derived by taking notice on the fracture phenomena of sea ice, however, the data for the fracture toughness are quite limited in numbers [6], [12], even at the present time. As the data will be increased, the analysis will be completed.

CONCLUSIONS

In-situ fracture toughness tests on sea ice were carried out at about -2°C in Saroma lagoon. The main results obtained are as follows.

(1) The critical stress intensity factor \(K_{IC}\) at about -2°C showed almost constant value if the rate of stress intensity factor \(K_I\) was less than \(10^2\) kPa\(\sqrt{m}/sec\), and decreased with increase in the \(K_I\), if the \(K_I\) exceeded \(10^2\) kPa\(\sqrt{m}/sec\).

(2) It was shown that the \(K_{IC}\) value has a close relationship with the subgrain size. The calculated flaw size coincided with the average subgrain size. Thus, the relationship between the \(K_{IC}\) and the inverse square root of subgrain size was derived.

\[
K_{IC} = 135 \exp (-1.9/\sqrt{d})
\]

where \(K_{IC}\) and \(d\) are given in the unit of kPa\(\sqrt{m}\) and mm, respectively.
The $K_{IC}$ value of the model ice for frozen towing tank laboratory should be determined from the $K_{IC}$ value of the prototype ice. And the $K_{IC}$ value for model ice will be obtained by adjusting the subgrain size.

REFERENCES


[4] G.W. Timco; ibid, p719


[10] N. Urabe and A. Yoshitake; unpublished work


Abstract
Fracture toughness tests were performed on 46 beams of $S_1$ columnar freshwater ice. The beams were notched and loaded in three-point bending. A load rate of 4.8 N/s was used on beams 0.22 m long, 0.025 m thick and 0.05 m deep. This load rate corresponds to a maximum strain rate in the upper surface of an unnotched elastic beam of $3 \times 10^{-6}$ s$^{-1}$. Two beam orientations were used in the tests. The preferred orientation of basal planes was perpendicular to the notch in 28 beams tested; this is denoted the $S^v_1$ orientation. The average fracture toughness of these beams was $240 \pm 79$ kPa$\cdot\sqrt{m}$, where the range indicates one standard deviation. The orientation of the basal planes in the other 18 beams tested was parallel to the notch; this orientation is denoted $S^h_1$. The average fracture toughness of the beams was $186 \pm 82$ kPa$\cdot\sqrt{m}$. The fracture toughness as measured by these tests is significantly cantly higher for $S^v_1$ ice than for $S^h_1$ ice. The observed anisotropy in fracture toughness is consistent with the Griffith theory of fracture. If crack propagation is controlled by the production of surface energy near the crack tip, then anisotropy in ice surface energy could cause the observed effect. The observed critical strain energy release rates are, however, more than an order of magnitude higher than the solid-vapor interface energy in ice, indicating that some other mechanism must control the fracture toughness. An alternative theory developed for ductile materials is presented in which the critical strain energy release rate is given by the rate at which work is done in deforming a small region near the crack tip. This theory explains the rate dependence of fracture toughness in ice and, since ice is anisotropic in creep, may also explain the variation of fracture toughness with crystal orientation.
Introduction
The ability to predict failure criteria for ice is important in the design of structures for ice-covered waters, in ice breaker design and in studies of acoustic noise generation by an ice cover. One of the fundamental material properties used in predicting failure is fracture toughness. The linear theory of fracture mechanics is based on a theory by Griffith for rupture in brittle glasses [1]. Essentially, this theory holds that a crack embedded in a body under a load will grow when the rate of work done by the load in deforming the body exceeds the rate at which the crack can absorb energy. Griffith postulated that the energy absorbed by a crack was due to the creation of new surface area. Irwin and Kies [2] and Orowan [3] modified this theory to account for failure in metals, which have low surface energies, by postulating that the energy is absorbed by plastic deformation around the tip of the crack.

Ice grown on a lake or on the sea often has a preferred crystallographic orientation. The crystals form basal plates which may be oriented vertically or horizontally, depending on growth conditions. For example, in the presence of a current, a horizontally preferred orientation may result with basal planes perpendicular to the current direction [4]. Since both surface energy and dislocation mobility of ice change with crystal orientation [5], the fracture toughness should vary with orientation in a predictable way. These observations suggest that the fracture properties of ice sheets may be anisotropic.

This report describes results from our fracture toughness tests of freshwater ice with a weakly preferred c-axis orientation. In comparing fracture toughness tests, it is important that the test conditions are identified. Since test specimens have varying geometries and sizes, dimensional parameters such as load rate or displacement rate are not directly comparable. It is also necessary to apply the results of fracture toughness tests to real problems which are rate sensitive in ice.

The most useful measure of rate is strain rate, which is nondimensional. The strain rate in a notched beam subject to three-point bending, as used in our tests, is nonuniform, especially near the crack. As a representative measure of strain rate in such an experiment, we calculate the maximum strain rate under the same load conditions in an unnotched elastic beam with the same dimensions. By this means, the effect of geometry and sample size is eliminated uniformly for differing tests.
Figure 1. Stereographic projection of c-axis determinations of ice plates. North and east are horizontal and parallel to the ice surface. Plates were grown from the surface down in molds suspended in a large container of tap water. The plane of the plates lies east-west on the projection. This ice has an $S_1$ columnar structure with 50% of the c-axes being vertical. The rest of the crystals show a slightly preferred orientation of basal planes parallel to the sides of the mold.

Ice beams were cut horizontally and vertically from the plates. The poles to notch planes cut in the beams are shown. $S_1^h$ beams contain notches parallel to the basal planes of the preferred orientation. $S_1^v$ beams contain notches perpendicular to the preferred basal plane orientation.

**Description of Tests**

In order to test the possibility of fracture toughness anisotropy, tests were conducted on 46 beams of $S_1$ columnar freshwater ice. The ice was grown in molds which produced plates of ice 0.025 m thick with a columnar structure. The c-axis orientations of the ice were measured with a universal stage. The ice had an $S_1$ structure with 50 percent vertical c-axis crystals superimposed on a background of c-axes roughly parallel to the sides of the mold (see Figure 1). The crystals were in vertical columns approximately 0.01 m in diameter. Beams of ice were cut from these plates such that the long axis of each beam was either parallel or perpendicular to the ice columns. The parallel orientation was denoted $S_1^h$ and the perpendicular $S_1^v$. Figure 2 shows the structure of the two beam orientations.

The final beam dimensions were 0.025 m thick by 0.05 m deep by 0.22 m long. A notch was cut into the midpoint of each beam to a depth of 0.01 to 0.02 m. A razor blade was then run along the base of the notch to form a sharp crack tip. A short (approximately 1 to 2 mm) crack was observed to run from the tip of the razor cut in every case. The presence of this initial crack is critical since the fracture toughness measurement assumes the crack is as sharp as a naturally occurring crack. Any crack tip bluntness will increase the observed toughness value.
The notched beams were loaded in three-point bending, see Figure 3, as described in Reference [6]. The load span was 0.20 m. A load was applied at a constant rate of 4.8 N/s. In our tests, the temperature was maintained at -17 ± 1°C. The relationship between load and strain for a three-point bending test of an unnotched beam is a simple result from beam theory. The maximum strain in the upper beam surface is

$$\varepsilon_{\text{max}} = \frac{3}{2} \frac{S}{BW^2} \frac{P}{E}$$

where $E$ is Young's modulus. For ice at -17°C, $E \approx 8$ GPa; therefore, the load rate of 4.8 N/s corresponds to a strain rate in the upper surface of an equivalent unnotched beam of

$$\dot{\varepsilon} = 3 \times 10^{-6} \text{ s}^{-1}$$

The notched and cracked beams were loaded to failure and the failure load was recorded. In all cases, the crack grew from the notch tip directly to the opposite beam surface producing a roughly planar failure surface. The lower tip of the initial crack was easily distinguished on the failure surface and was used to determine the initial crack length.

Stress intensity factors for elastic beams of the same dimension and load geometry as used here have been measured [7]. These values were used along with failure loads and initial crack lengths obtained from our experiments to determine critical stress intensity factors. No corrections for crack tip plasticity were made.
Test Results

In the beams with the $S_1^v$ orientation, the crack runs roughly perpendicular to the basal planes. A histogram of the fracture toughness values is given in Figure 4a. The average fracture toughness is

$$\bar{K}_{IC}(S_1^v) = 240 \pm 79 \text{ kPa} \cdot \sqrt{\text{m}},$$

where the range indicates one standard deviation. The lowest value is 120 kPa·$\sqrt{\text{m}}$ and the highest is 400 kPa·$\sqrt{\text{m}}$.

In beams with the $S_1^h$ orientation, the crack plane parallels the basal planes of the ice. A histogram of the fracture toughness values is shown in Figure 4b. The average value is

$$\bar{K}_{IC}(S_1^h) = 186 \pm 82 \text{ kPa} \cdot \sqrt{\text{m}},$$

The values range from a minimum of 120 kPa·$\sqrt{\text{m}}$ to a maximum of 380 kPa·$\sqrt{\text{m}}$.

It is reasonable to assume that these fracture toughness values are normally distributed about their means. The difference between the two means is 54 kPa·$\sqrt{\text{m}}$. The variance of this difference may be estimated from

$$\sigma^2 = \frac{s_v^2}{n_1} + \frac{s_h^2}{n_2},$$

where $s_v$ is the standard deviation of $K_{IC}(S_1^v)$ for $n_1 = 28$ samples and $s_h$ is the standard deviation in $K_{IC}(S_1^h)$ for $n_2 = 18$ samples [8]. Then, $\sigma = 24 \text{ kPa} \cdot \sqrt{\text{m}}$, indicating that the difference in fracture toughness determinations is significant.
A number of factors may contribute to the large scatter in fracture toughness values. These include variation in crystal orientation, variable strain rates and variations in the sharpness and geometry of the initial crack. The large variation in c-axis orientation implies that in some $S_1^v$ tests the crystals at the crack tip are in orientations corresponding to $S_1^h$ tests and vice versa.

Our observations indicate that the fracture toughness of ice is lower for cracks in the basal plane than for those cutting across basal planes. This result is consistent with the observation by Michel [9] that fractures in pseudo-monocrystalline $S_1$ ice occur preferentially along the basal plane.

The solid-vapor interface energy in ice is anisotropic, with the basal plane having the lowest value [5]. The magnitude of this energy is approximately $0.1 \text{ J/m}^2$ at $0^\circ\text{C}$. The values of fracture toughness obtained in this study, however, correspond to critical strain energy release rates of 3 to 6 J/m$^2$. It is therefore unlikely that surface energy controls fracture in ice.
The observed anisotropy in fracture toughness is probably explained by the Irwin-Orowan theory. Dislocation mobility in ice is strongly dependent on the relative orientation of crystal structure to applied stress. The orientation of the ice crystal near the crack tip will thus determine its ability to absorb energy.

Our values for fracture toughness of freshwater columnar ice at a strain rate of $3 \times 10^{-6}$ $s^{-1}$ are considerably higher than those obtained by experiments at higher strain rates. Goodman [10] used strain rates in excess of $3 \times 10^{-4}$ $s^{-1}$ to obtain a fracture toughness value $K_{IC}$ of 115 kPa·$\sqrt{m}$. Hamza and Muggeridge [11] applied constant displacement rates corresponding to strain rates from $10^{-4}$ to $10^{-2}$ $s^{-1}$ on similar notched-beam specimens. They obtained fracture toughness values ranging from 100 to 200 kPa·$\sqrt{m}$. Other experiments at high strain rates have shown similar results [12-14].

Miller [15] has shown that strain rate has a significant effect on ice fracture toughness. At lower strain rates, the fracture toughness increases dramatically. This is inconsistent with the Griffith fracture theory, in which surface energy production controls crack propagation, since surface energy is a constant.

Our results, at a relatively low strain rate, are consistent with Miller's results. The theory of fracture as modified for metals [2,3] accounts for this strain rate effect by postulating that cracks absorb energy in a deformation zone near the crack tip. In ice, deformation is rate dependent: at low strain rates, a greater crack tip deformation is possible, increasing the work done at the crack tip and thus increasing fracture toughness. At the temperature and strain rate used in our experiments, ice can undergo a significant creep strain [16]. This can account for the increased fracture toughness observed relative to experiments at high strain rates.

**Conclusions**

Our tests on freshwater ice with a weakly preferred c-axis orientation indicate anisotropy in fracture toughness. The fracture toughness is higher for cracks which cut across basal planes than for cracks parallel to basal planes. The surface energy of ice is not large enough to account for our observed fracture toughness values. An alternate theory of fracture accounts for the higher values by postulating a zone of energy dissipation in a small, highly deformed region near the crack tip. This theory also accounts for the increase in fracture toughness at low strain rates. Anisotropy in ice fracture toughness may be a consequence of variation of dislocation mobility with crystal orientation. A detailed examination of creep structures in fractured ice should resolve this question.
Acknowledgement
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References


TRANSVERSE PRESSURE EFFECTS
ON AN EMBEDDED ICE PRESSURE SENSOR

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ABSTRACT

Ice sheet pressure is recognized as an important factor in the design of offshore drilling and production structures. Field measurements of ice pressure are used to monitor ice activity and to provide estimates of the magnitudes of natural ice pressure.

When an ice pressure sensor embedded in an ice sheet is exposed to a far-field transverse ice pressure (ice pressure in the direction normal to the sensing direction), the pressure sensor can respond to this transverse ice pressure in two ways. First, transverse pressures at the sensor may be sensed directly, as a result of cross sensitivity of the sensor itself. Second, the far-field transverse ice pressures may give rise to sensing direction pressures at the sensor location; these would be measured in normal fashion. Both mechanisms must be minimized in order that the far-field ice pressures can be accurately measured.

This paper presents the analytical studies on the transverse pressure effects on an embedded ice pressure sensor. Solutions to a thin rectangular inclusion embedded in an ice block are obtained by the finite element method. Therein, transverse pressure effects on the EPR ice pressure sensor corresponding to various sea ice/pressure sensor stiffnesses ratios are obtained.
INTRODUCTION

Ice sheet pressure is recognized as an important factor in the design of offshore drilling and production structures in the arctic. Several pressure sensors have been designed and built to measure the pressure in an ice sheet [1]. Field measurements of ice pressure can be used to monitor the ice activity and to support the estimates of the magnitude of natural ice pressure [2, 3].

The inclusion of a pressure sensor in an ice sheet will disturb the pressure distribution in the ice sheet because of the difference between the stiffnesses of pressure sensor and ice sheet. Therefore, the pressure felt by the sensor differs from the undisturbed ice sheet pressure which is parallel to the sensing direction of sensor (normal pressure). Furthermore, the transverse ice pressure can cause the sensor to respond in two ways. First, transverse pressure at the sensor may be sensed directly, as a result of cross sensitivity of the sensor itself. Second, the far-field transverse ice pressure may give rise to sensing-direction pressure at the sensor location; these would be measured in normal fashion.

As an ice pressure sensor is primarily designed to measure the normal pressure, many investigators have studied the inclusion effect and proposed approximate inclusion equations to relate the undisturbed pressure in the ice sheet to the pressure felt by the sensor [4, 5, 6]. However, most studies ignore the effects of transverse pressure. Templeton, by assuming that sea ice is a linear elastic material, proposed a closed-form solution applicable to an embedded ice pressure sensor of rectangular cross section having relatively rigid sensing face cover plates [5].

This paper presents analytical studies on the transverse pressure effects on an embedded ice pressure sensor. Elastic-plastic solutions are obtained by the finite element method to study the interaction of EPR ice pressure sensors with the surrounding sea ice.

DEFINITION OF THE PROBLEM

The problem considered here is to study the behavior of sea ice around the pressure sensor when the ice sheet is subjected to transverse ice pressure (Fig. 1.a). The mathematical model used for the solution of the problem is as shown in Fig. 1.b. The derivation of this model from the physical problem has already been discussed in [4] and will not be repeated.

Herein, the sea ice is assumed to be a linear elastic-perfectly plastic material. Furthermore, the ice is assumed to be isotropic in the horizontal plane. Thus, any
anisotropy due to preferred crystal orientation is ignored. The yield function used for sea ice is the function presented by Pariseau [7], reduced to plane stress condition:

\[ a_1\sigma_y^2 + a_3\sigma_x^2 + 2(a_1 + 2a_3)\tau_{xy}^2 + \alpha(\sigma_x + \sigma_y) = \sigma_0^2 \]  

(1)

The constants \(a_1, a_3, \alpha, \) and \(\sigma_0\) in Equation (1) can be evaluated from ice strength values; i.e., horizontal unconfined compressive strength, \(C_X\); horizontal unconfined tensile strength, \(T_X\); and the biaxial strength corresponding to a two-dimensional horizontal pressure, \(\sigma_A\).

The graphical presentation of this yield function is as shown in Fig. 2. In the linear elastic range, the sea ice under consideration is assumed to have Young’s modulus of 400,000 psi and Poisson’s Ratio, 0.3.

A number of different ice pressure sensor designs have been developed by EPR. Widths
from 16.67 in. to 18.0 in. and sensing-direction design stiffnesses from 40,000 psi to 267,000 psi have been used, but all of the designs have incorporated essentially the same cross-sectional relative geometry. Two specific designs were analyzed in this study. Characteristics of the designs analyzed are presented in Table 1. The same geometrical configuration is used for the finite element meshes employed for both pressure sensors, and the only difference between the finite element models is the stiffness values of linear springs.
TABLE 1 - Characteristics of Analyzed Ice Pressure Sensors

<table>
<thead>
<tr>
<th></th>
<th>40,000 psi sensor</th>
<th>250,000 psi sensor</th>
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</thead>
<tbody>
<tr>
<td>Thickness (in.)</td>
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<td>0.44</td>
</tr>
<tr>
<td>Width (in.)</td>
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<tr>
<td>Stiffness in Y-direction (psi)</td>
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<td>250,000</td>
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<tr>
<td>Stiffness in X-direction (psi)</td>
<td>7,200,000</td>
<td>7,200,000</td>
</tr>
</tbody>
</table>

(Details presented in Reference 8)

MATHEMATICAL SOLUTIONS

As a closed-form analytical solution to an elastic rectangular inclusion in a stressed medium is untractable, the finite element method is used. The numerical solutions are obtained from MARC Finite Element Program [9].

Plane stress, 8-node distorted quadrilateral elements are used to simulate the ice sheet. Since there is a stress concentration region near the edges of the pressure sensor, the elements used there have to be made relatively small compared to the elements used elsewhere. (The area ratio of the largest element to the smallest one is approximately 10,000 : 1.) Furthermore, the transition of the smaller size elements to the larger size elements is gradual in order to obtain a reasonable solution.

Linear spring elements are used to simulate the pressure sensor for the following reasons: the aspect ratio of the cross section of the pressure sensor is extreme; an excessive number of plane stress elements would be required for the representation; and the stress distribution in the interior of the pressure sensor is not of interest.

BEHAVIOR OF THE SEA ICE AROUND THE PRESSURE SENSOR

Although the numerical solutions for the two pressure sensors differ from each other, the general results for following transverse pressure effects are similar. The discussion, however, is specifically on the pressure sensor with 250,000 psi stiffness.

The initial yield occurs in the sea ice adjacent to the edges of the pressure sensor when the applied transverse ice pressure is only 18.87 psi (7.86% of the unconfined ice strength, 240 psi). This pressure will be termed the "initial yield pressure". As the applied transverse ice pressure increases, the plastic zone grows; however, because of the high stress concentration, the growth of the plastic zone is very slow; consequently, except for applied...
pressures near yield, the plastic zone remains confined to the vicinity of the edges of the pressure sensor.

**APPLIED TRANSVERSE ICE PRESSURE, p (psi)**

- 18.87
- 56.60
- 75.47
- 169.80
- 235.83

**FIGURE 3. NORMAL STRESS DISTRIBUTION IN THE Y-DIRECTION ALONG THE X-AXIS**

Fig. 3 shows the normal stress distributions of the sea ice in the Y-direction along the X-axis at different load increments. The normal stress components shown here have been normalized by the applied ice pressure of each corresponding load increment. At the initial yield pressure, there is a sharp stress concentration in the sea ice adjacent to the edges of the sensor. Examining the stress distribution of the sea ice farther away from the sensor, one can see that the stress concentration changes very rapidly from a compression stress state to a tension stress state and then gradually tapers off to zero. At a distance approximately 10 inches from the sensor, the disturbance of the stress distribution caused by the inclusion of the sensor has virtually vanished. As the applied transverse ice pressure increases, both stress concentrations in tension and compression keep on decreasing as the plastic zone grows. Immediately before gross yielding of the ice block, the stress
distribution in the sea ice, though maintaining the same pattern as that at initial yield pressure, becomes much more uniform.

**Figure 4. Normal Stress Distributions in the Y-direction Along the Sensor**

The response of the ice pressure sensor to the applied transverse ice pressure is mainly due to the pressure sensor restraining the sea ice's movement in the transverse direction. When the pressure in the ice sheet increases, the ice near the edges of the pressure sensor yields and deforms plastically before the main body of the ice block does; consequently, the effect of restraining of sea ice movement by the pressure sensor in this area is lessened, and the force which the pressure sensor has to absorb is decreased. Shown in Fig. 4 are the normal stress distributions in the Y-direction along the pressure sensor at different load increments. Again the normal stress components shown here have been normalized by the applied ice pressure of each corresponding load increment. At the initial yield pressure, the stress felt by the sensor at the center of the sensor is approximately 0.05 of the applied transverse ice pressure. The magnitude of this stress increases gradually toward the edges of the sensor; wherein the magnitude of the stress is approximately 0.19 of the applied transverse ice pressure. When the sea ice adjacent to the edge of the sensor becomes plasticized, the stress at the edge of the sensor becomes tension while the stress at the center of the sensor decreases. As the applied transverse ice pressure increases, the tensile
stress at the edge becomes more prominent while the stress at the center keeps on decreasing. Just before the gross yielding of the ice block, the tensile stress at the edge of the sensor sharply peaks to 46% of the applied transverse ice pressure, and the compressive stress at the center of the sensor is only approximately 2% of the applied transverse ice pressure.

EFFECTS OF TRANSVERSE PRESSURE ON EPR ICE PRESSURE SENSORS

As stated previously, a pressure sensor will respond to transverse pressures in two ways: (1) the pressure sensor will deform in the transverse direction; (2) the pressure sensor will sense the pressure exerted by the sea ice in its sensing direction. EPR ice pressure sensors have been designed so that the deformation of the sensor in the transverse direction will not cause significant output from the pressure sensors. Therefore, the discussion herein will be limited to the second effect.

<table>
<thead>
<tr>
<th>40,000 psi Sensor</th>
<th>250,000 psi Sensor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied Transverse Ice Pressure (psi)</td>
<td>Average Sensing Pressure (psi)</td>
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<tr>
<td>19.61</td>
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<td>39.22</td>
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<tr>
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<td>225.49</td>
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<tr>
<td>235.29</td>
<td>-0.94</td>
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<tr>
<td>235.83</td>
<td>-1.06</td>
</tr>
</tbody>
</table>
Table 2 shows the values of transverse loading effect of the sensors at various load increments. At the initial yield pressure, the value of transverse loading effect is 1 or 5%. As the applied transverse ice pressure increases, the values of transverse loading continually decrease. Immediately before the gross yielding of the ice block, the sensing face pressure is approximately -1.00% of the applied transverse ice pressure; i.e., when the applied transverse ice pressure is compressing the sensor in the transverse direction, the sea ice will tend to pull open the pressure sensor in the normal direction. As can be seen, the sensing face pressure achieves a maximum value of 2.51 psi. The magnitude of this value is an important limitation on transverse pressure effect errors for applications of EPR ice pressure sensors.

SUMMARY AND CONCLUSIONS

Elastic-plastic solutions are obtained for the interaction of EPR ice pressure sensors with the surrounding sea ice. The formation of the plastic zones enables the sea ice around the edges of the pressure sensor to flow plastically, and the restraint from the pressure sensor to the transverse movement of the sea ice in these regions is reduced. Therefore, the effect of transverse loading on the pressure sensor decreases as the applied transverse ice pressure increases. The sensing face pressure achieves a maximum value of 2.51 psi. This value is an important limitation on transverse pressure effect errors for applications of EPR ice pressure sensors.

REFERENCES


ON THE ACOUSTIC EMISSION AND DEFORMATION RESPONSE
OF FINITE ICE PLATES

ABSTRACT

A procedure is described for monitoring the microfracturing of ice plates subjected to constant loads. Sample time records of fresh water ice plate deflections as well as corresponding total acoustic emission activities are presented.

The linear elastic as well as viscoelastic response for a simply supported rectangular floating ice plate subjected to a central distributed loading is obtained. Stresses and deformations are calculated for a variable Young's modulus with the ice plate depth and a constant Poisson's ratio. The Rayleigh-Ritz procedure is used to obtain the elastic solution for the transverse plate deflection, while the viscoelastic solution is obtained from the elastic response using the correspondence principle. It is shown that the elastic solution depends on five nondimensional parameters: the plate aspect ratio, two parameters characterizing the size of the rectangular loaded area, the ratio of applied forces to the flexural resisting forces and the ratio of the elastic foundation reaction forces to the flexural resisting forces. The viscoelastic solution depends also on the nondimensional time.

In the present investigation acoustic emission methods are used to study the microfracturing activity in polycrystalline ice subjected to flexural loads. Experimental results obtained in the laboratory indicate that the acoustic emissions recorded from ice are important in describing the deformation and fracture of ice.

INTRODUCTION

The relationship between acoustic emission and time dependent inelastic deformations in the field of rock mechanics has been investigated by a number of authors for both uniaxial and triaxial loading conditions. The results of these investigations suggest a strong correlation between creep strain and acoustic emission. By way of contrast a limited number of studies have been carried out on the acoustic emission...
characteristics of ice during time dependent deformations.

Gold [1] has studied the microcracking activity in ice during creep under constant compressive loadings. He reports two stages of cracking. The sudden transition reported by Brown [2] in the character of the creep-time curve was related to the internal cracking activity of ice deformations during creep.

Zaretsky et al [3] have reported results from uniaxial compression creep tests on columnar-grained ice using acoustic emission techniques.

Recently some experimental work [4] on the acoustic emission behavior of equiaxed polycrystalline ice in uniaxial compression, has been reported. The dependence of the axial strain and total number of acoustic events on time is illustrated in Fig. 1 [4]. The nominal applied stress was 2.0 MPa and the sample was strained to over 5%. The full load was applied at a uniform rate over a time period of two seconds.

![Fig. 1 Strain and Total Acoustic Emission Events versus Time - Uniaxial Compression](image_url)

The dependence of the rate of acoustic activity with respect to the axial strain is shown in Fig. 2 [4].

A discussion of the acoustic emission source mechanisms in ice was included [4].

In the present study a procedure is described for monitoring the micro-fracturing of ice plates subjected to constant loads.

†In a qualitative sense, the acoustic emission response was found to be similar for both columnar grained and fine grained equiaxed ice.
EXPERIMENTAL PROCEDURE

In this section a brief description is given of the procedure used in growing and testing the ice plates and of the acoustic emission and loading system. The detailed description of the experimental procedure which includes the following considerations:

1. growth of the ice plate.
2. support of the ice plate.
3. acoustic emission monitoring system.
4. displacement transducers and data recording.
5. mechanical loading system.
6. thin section analysis.

is given in reference [5].

Relatively thin ice plates were grown in a stress free state. A square plywood box with internal dimensions of 1.02m x 1.02m and 0.6m deep was used. The room temperature was kept at a nominal temperature of -10°C. The ice plate was supported on a platform made up of four iron bars arranged in a square. An illustration of the support structure with an ice plate in position is shown in Fig. 3. An acoustic emission monitoring system was used to monitor the acoustic emissions due to ice plate deformations. The total number of acoustic events was recorded over the test duration. The amplifiers used in these experiments had a bandpass between -3db points of 100 to 300 kHz. The load was applied at the center of the ice plate. Direct current deformation transducers (DCDT) as well as linear variable differential transducers (LVDT) were used to measure ice plate vertical deflections.
Constant load tests were conducted for all ice plates. Weights were applied at the center of the ice plate. Samples from each ice plate were collected at the completion of each test. Thin sections were subsequently prepared and the average grain diameter was determined for each plate. Single crystals extended across the full depth of the ice plate.

**ICE PLATE RESPONSE**

Viscoelastic plate theoretical formulations to describe floating ice time dependent deformations were presented in References [6,7 etc.]. Large plate deflections and shear deformations were retained. Furthermore the presence of a non-uniform temperature distribution across the thickness of the plate was taken into account [6,7]. The ice plate was treated as a viscoelastic plate on an elastic foundation.

Ice plates were modeled by thin isotropic plates with spring-dashpot constitutive relations in References [8,9 etc.]. Time dependent floating ice plate deflections were calculated in Reference [8] using the bilateral Laplace Transform and considering that ice response is incompressible under hydrostatic stress and obeys Maxwell's model for deviatoric stress and strain. A Maxwell-Kelvin three element model was employed in Reference [9] to study the influence of loading time on the bearing capacity of an ice plate. Ice plates with radial or longitudinal cracks were also studied [9]. A similar model was used in Reference [8] that exhibits primary, secondary and tertiary creep response characteristics. At the center of the load, deflections were found to increase with time while stresses to decrease with time [8]. A Maxwell-Kelvin four element model was employed in Reference [9] together with a numerical integration technique to study the time dependent response of an infinite floating ice plate loaded with a circular load. The theoretical results obtained were fitted to field data and the ice material constants were
found to be functions of time and location [9].

The finite element method (FEM) of analysis was used by various investigators [10-14] to study the time dependent response of floating ice plates. A linear viscoelastic FEM analysis of an infinite sea ice plate on a fluid foundation was reported in Reference [10]. Numerical results were compared with test data for an Arctic sea-ice sheet [10]. A similar approach was employed in Reference [11] to study the influence of reinforcement in an artificially thickened ice plate (W. Hecla N-52 offshore platform). A few case histories were analyzed in Reference [12] using finite element techniques developed for long term analysis of floating ice platforms. The power creep law was taken to govern ice time dependent deformations [12]. A similar approach was employed in Reference [13] to study the creep response of floating sea ice sheets. A creep bending finite element model has been developed in Reference [14] using the initial strain approach. Thick plate theory and a power creep law were assumed for ice plate time dependent deformations [14].

The linear elastic as well as viscoelastic response for a simply supported rectangular ice plate was presented in Reference [5]. The assumption was made that ice is incompressible under hydrostatic stress and obeys Maxwell's model for deviatoric stress and strain. The viscoelastic response was obtained from the corresponding elastic solution using the correspondence principle. An experimental procedure was also developed [5] to monitor the microfracturing activity in ice plates subjected to constant loads. Fresh water ice plate deflections as well as corresponding total acoustic emission activities were recorded [5].

In this article the work reported in Reference [5] is extended to include the important influences on ice plate response of the variation of ice mechanical properties across the thickness and of the presence of the water foundation.

**ICE PLATE LINEAR ELASTIC RESPONSE**

The linear elastic response of a simply supported rectangular plate subjected to a central loading (perpendicular to the plane) of the plate of constant magnitude $p$ per unit area (Fig. 4) is developed. The ice plate is resting on a liquid foundation with specific weight $k_1$. The ice plate Young's modulus $E$ is assumed to depend on the vertical cartesian coordinate $z$ (Fig. 4), [15]. Taking the Poisson ratio $\nu$ to be constant, the solutions obtained for homogenous plates may be used for floating ice plates, if a modified flexural rigidity $D_1$ is used

$$D_1 = \frac{1}{1-\nu^2} \int_{-z_0}^{h-z_0} z^2 E(z)dz$$  \hspace{1cm} (1)

where $h$ is the ice plate thickness.
Fig. 4 Rectangular Plate Subjected to Distributed Loading.

The parameter $z_0$ can be evaluated from the requirement that the resultant normal forces are zero at the neutral plane.

The bending strain energy $U_B$ stored in the plate during deformation is

$$U_B = \frac{1}{2} D_1 \int_0^a \int_0^b \left[ (w_{xx} + w_{yy})^2 - 2(1 - v) [w_{xx} w_{yy} - (w_{xy})^2] \right] \, dx \, dy$$

where $w$ is the plate transverse deflection and $a$, $b$ are the plate dimensions (Fig. 4).

The potential energy $U_F$ stored in the linear liquid foundation is

$$U_F = \frac{1}{2} \int_0^a \int_0^b k_1 w^2 \, dx \, dy$$

The change in potential energy $U_w$ of the applied load per unit area $p$ is

$$U_w = p \int_{\frac{a-c}{2}}^{\frac{a+c}{2}} \int_{\frac{b-d}{2}}^{\frac{b+d}{2}} w \, dx \, dy$$

The total energy $U$ of the plate foundation system is

$$U = U_B + U_F + U_w$$

Since simple support conditions prevail on all four edges of the plate the deflection is expanded in the following series [5]

$$w = \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} w_{mn} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}$$
Minimizing the total energy $U$ with respect to the amplitude term $w_{mn}$ yields

$$
\frac{w_{mn}}{h} = \frac{16}{\pi^2 m^2 n^2} \left( \frac{\sin \frac{m\pi c}{2a} \sin \frac{n\pi d}{2b}}{m^2 + \left( \frac{n\pi}{b} \right)^2 + K_1} \right)
$$

(7a)

where $P = \frac{pa^4}{hD_1}$ and $K_1 = \frac{k,a^4}{D_1}$

(7b, c)

ICE PLATE LINEAR VISCOELASTIC RESPONSE

The assumption is made that ice is incompressible under hydrostatic stress and obeys Maxwell's model (Fig. 5) for deviatoric stress and strain [5,8]. The viscoelastic response can then be obtained from the corresponding elastic solution using the correspondence principle.

$$
\sigma = \frac{q_1}{r_1} - \frac{q_1}{\sigma + r_1\sigma} = q_1 e
$$

Fig. 5 Elements of Maxwell Model

The deflection amplitude term $w_{mn}/h$ is given by

$$
\frac{w_{mn}}{h} = \frac{16}{\pi^2} \frac{1}{k_1 mn} \frac{\sin \frac{m\pi c}{2a} \sin \frac{n\pi d}{2b}}{1 + \left( \frac{K_1 Z_{mn}}{Z_{mn}} \right) \exp \left( - \frac{K_1 t}{r_1 Z_{mn}} \right)} \mu(t)
$$

(8)

where $Z_{mn} = k_1 + \pi^4 \left[ m^2 + \left( \frac{n\pi}{b} \right)^2 \right]^2$

(9)

and $q_1, r_1$, are the Maxwell model parameters (Fig. 5).

EXPERIMENTAL RESULTS

In a previous [5] test sequence nice ice plates were tested. The results of these tests are summarized in Table 1. The values for the ice material constants based on Maxwell's model idealization varied considerably with each test. To achieve a more uniform gain size distribution the last three tests were grown by seeding the water surface (when the water temperature reached the freezing point temperature) with snow grains of predetermined size.

A plot of the normalized ice plate displacement (at the LVDT location) and total acoustic emission event activity as a function of the normalized time is presented in Fig. 6 for tests number seven and eight.
TABLE 1

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Applied Load [N]</th>
<th>Plate Thickness [mm]</th>
<th>$E_0 \times 10^5$ [N/cm$^2$]</th>
<th>$r_1$ [hr]</th>
<th>Grain Size Seeding [mm]</th>
<th>Average Grain size (surface projection) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>196</td>
<td>19</td>
<td>*</td>
<td>*</td>
<td>no</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>49</td>
<td>15</td>
<td>38.283</td>
<td>1.128</td>
<td>no</td>
<td>5.8</td>
</tr>
<tr>
<td>3</td>
<td>157</td>
<td>17</td>
<td>**</td>
<td>**</td>
<td>no</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>49</td>
<td>9</td>
<td>13.929</td>
<td>11.268</td>
<td>no</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>109</td>
<td>13</td>
<td>3.356</td>
<td>47.059</td>
<td>no</td>
<td>11.7</td>
</tr>
<tr>
<td>6</td>
<td>93</td>
<td>15</td>
<td>1.356</td>
<td>28.261</td>
<td>no</td>
<td>6.7</td>
</tr>
<tr>
<td>7</td>
<td>206</td>
<td>28</td>
<td>0.169</td>
<td>28.420</td>
<td>2.38</td>
<td>2.9</td>
</tr>
<tr>
<td>8</td>
<td>615</td>
<td>30</td>
<td>0.094</td>
<td>65.365</td>
<td>2.38</td>
<td>3.3</td>
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<tr>
<td>9</td>
<td>809</td>
<td>30</td>
<td>**</td>
<td>**</td>
<td>2.38</td>
<td>8.6</td>
</tr>
</tbody>
</table>

* Printer failure on data acquisition system does not permit reliable material parameter estimation.

** Instantaneous failure.

Fig. 6 Displacement and Total Acoustic Emission Events versus Time – Finite Ice Plates.

In a subsequent test sequence eight additional ice plates were tested. The results of these tests are summarized in Table 2.
### TABLE II

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Applied Load [N]</th>
<th>Plate Thickness [mm]</th>
<th>Average Grain Size [mm]</th>
<th>Threshold [V]</th>
<th>Test Duration t_F [hours]</th>
<th>Plate at the end of the test</th>
<th>Total Acoustic Emission Events (A.E.)_F</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>703</td>
<td>30.0</td>
<td>2.35</td>
<td>0.80</td>
<td>90</td>
<td>62.017</td>
<td>4.15</td>
</tr>
<tr>
<td>11</td>
<td>412</td>
<td>31.1</td>
<td>2.87</td>
<td>0.80</td>
<td>90</td>
<td>46.025</td>
<td>2.43</td>
</tr>
<tr>
<td>12</td>
<td>609</td>
<td>22.5</td>
<td>3.32</td>
<td>0.80</td>
<td>90</td>
<td>87.921</td>
<td>10.92</td>
</tr>
<tr>
<td>13</td>
<td>713</td>
<td>24.5</td>
<td>3.54</td>
<td>0.80</td>
<td>90</td>
<td>0.058*</td>
<td>0.83</td>
</tr>
<tr>
<td>14</td>
<td>703</td>
<td>29.4</td>
<td>2.75</td>
<td>0.80</td>
<td>90</td>
<td>68.497</td>
<td>6.40</td>
</tr>
<tr>
<td>15</td>
<td>296</td>
<td>27.1</td>
<td>3.53</td>
<td>0.80</td>
<td>90</td>
<td>91.383</td>
<td>2.96</td>
</tr>
<tr>
<td>16</td>
<td>510</td>
<td>34.0</td>
<td>2.97</td>
<td>0.80</td>
<td>90</td>
<td>44.433</td>
<td>3.22</td>
</tr>
<tr>
<td>17</td>
<td>510</td>
<td>29.3</td>
<td>2.43</td>
<td>0.80</td>
<td>90</td>
<td>61.333</td>
<td>3.91</td>
</tr>
</tbody>
</table>

* Plate fractured.

** Total acoustic emission counts.

### CONCLUSIONS

An experimental procedure is described for monitoring the development of acoustic emission activity of finite ice plates subjected to constant loads. Sample time records of fresh water ice plate deflections as well as corresponding total acoustic emission activities are presented. The linear elastic as well as viscoelastic response for a simply supported rectangular ice plate is given. Ice material parameters are estimated using the theoretical solutions described. The following tentative observations can be made:

1. the ice plate creep curve did not display a region of increasing strain rate (tertiary creep), at least for the tests conducted during the present study.

2. the estimated ice material parameters based on Maxwell's linear viscoelastic model varied quite significantly from test to test. This can probably be due to the different structure for each tested plate.

3. a rapid rise in the number of acoustic emission events is observed following the load application.

4. in some cases a relatively constant increase of the number of acoustic events with time is observed as straining proceeds.

5. as the test proceeds often we observe significant increases of the acoustic emission activity over short periods of time. We attribute this to local yielding of the ice.

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MECHANICAL PROPERTIES OF LOW DENSITY ICE UNDER CYCLIC AXIAL LOADING

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Abstract
As part of a long-term study to evaluate dynamic properties of geologic materials under simulated earthquake and low frequency loading conditions, cyclic triaxial tests were performed on laboratory prepared test specimens of low density ice. The test specimens, at a density of 0.77 g/cc, were tested at axial strain amplitudes from $10^{-3}$ to $1.6 \times 10^{-2}\%$, temperatures from -1 to -10°C, frequencies from 0.3 to 5.0 Hz, and confining pressures from 0 to 700 kN/m². The values of dynamic Young's modulus for the test specimens over the range of test conditions were from 1.2 to 4.4 GN/m²; the values of damping ratio were from 0.02 to 0.12. The test results indicate that dynamic Young's modulus of low density ice increases, in general, with increasing confining pressure, frequency, descending temperature, and decreasing axial strain amplitude. The test results indicate that, in general, damping ratio of low density ice decreases as frequency increases from 0.3 to 1.0 cps and increases as frequency increases from 1.0 to 5.0 Hz. The damping ratio tends to decrease with descending temperature. The damping ratio increases slightly with increasing strain amplitude. There appears to be no well-defined relationship between damping ratio of low density ice and confining pressure. A comparison of the dynamic properties of low density to high density ice indicates that dynamic Young's modulus increases significantly as ice density increases. The influence of ice density on damping ratio is not well defined. A comparison of dynamic properties of ice obtained in the present study to those obtained in previous studies indicated values of wave velocities determined in the present study are lower than wave velocities determined in previous laboratory and field studies. It appears, however, the differences can be explained by differences in the test techniques employed in the present study compared to those in previous studies. The values of damping ratio determined in the present study compare favorably to those obtained in previous laboratory studies.
Introduction

Early research to evaluate properties of ice under cyclic loading conditions arose in connection with (1) vibrating machinery placed on or in frozen ground deposits, (2) geophysical exploration of frozen ground deposits, (3) excavation of frozen ground deposits by blasting, and (4) wheel loadings on ice floes or ice bridges. More recently, however, the demand for energy in Alaska and other cold regions of the world has focused attention on the need to evaluate properties of ice under cyclic loading conditions arising in connection with (1) earthquake loadings and (2) ice floe movement. As part of a long-term study to evaluate dynamic properties of ice under low frequency and simulated earthquake loading conditions, cyclic triaxial tests were conducted on laboratory prepared specimens of ice at high strain amplitudes (>10^{-3}%), low frequencies (<5.0 cps), and over a range of confining pressures (0 to 700 kN/m^2). The results of these studies are reported herein.

Mechanical Property Determination and Cyclic Triaxial Test System

The mechanical properties of ice under cyclic loading conditions can be evaluated in a cyclic triaxial test. In the cyclic triaxial test a cylindrical test specimen is placed in a triaxial cell and confined to an initial isotropic stress state, as shown in Figs. 1a and 1b. An axial load is cycled on the specimen causing a reversal of shear stresses which are a maximum on 45 degree planes. During the test, the cyclic axial load and specimen deformation are recorded. The axial (deviator) stress and strain in the specimen are determined with a knowledge of the cross-sectional area and length of the specimen. Typical test results expressed in these terms for one cycle of loading are shown in Fig. 1c. From the results shown in Fig. 1c, dynamic Young's modulus, $E_d$, and damping ratio, $\lambda$, may be calculated as follows:

$$E_d = \frac{\sigma_{\text{max. deviator}}}{\varepsilon_{\text{max. axial}}}$$

(1)

and

$$\lambda = \frac{A_L}{4\pi A_T}$$

(2)

with the terms as defined in Fig. 1c. $A_L$ represents the total dissipated energy per cycle and $A_T$ represents the work capacity per cycle.

The cyclic triaxial test system employed in the test program has been described in detail by Vinson, et al. [1]. The test system represents a coupling of existing equipment to evaluate the dynamic properties of unfrozen soils with existing temperature control equipment to evaluate the static properties of frozen soils. The test system consists of four basic components: (1) a triaxial cell surrounded by a circulating coolant in a cold bath; the triaxial cell contains the test specimen and non-
circulating coolant; (2) an electrohydraulic closed-loop test system which applies a cyclic axial load to the piston loading rod of the triaxial cell, (3) a refrigeration unit that maintains the temperature of the system at a given level; and (4) output recording devices.

Test Specimen Preparation and Description

All tests were conducted on ice specimens prepared in the laboratory as follows:

1. A hollow cylindrical teflon mold, with the specimen base inserted in one end, and the specimen cap, was placed in a large freezer box maintained at a temperature of \(-30 \pm 1^\circ C\) for approximately one hour. Both the cap and base have a "coupling" device consisting of an aluminum plate and four allen head screws to allow the specimen to be subjected to tension.

2. The teflon mold was filled to within 50 mm from the top with loose, dry, clean snow passing the No. 4 sieve.

3. Precooled carbonated water, close to \(0^\circ C\), was poured into the snow from the top, and the top cap was inserted.

4. The mold was placed in a freezer maintained at a temperature of \(-30 \pm 1^\circ C\) for approximately 24 hours; the sample was frozen multidirectionally and then extruded from the mold.

The resulting ice specimens were cloudy and bubbly in appearance with an average density of 0.77 g/cc. The grains comprising the specimen were from less than 0.1 to 0.2 mm in size. They had a random crystallographic orientation. Approximately one out of three specimens were cracked or contained excessive bubbles, voids, or appeared to have large variations in density and were rejected. The specimens used in the test program were, therefore, quite homogeneous. There was a slight radial pattern of ice crystals visible in some specimens when they were broken apart and examined in cross-section.
Test Program and Results

Test Program. Strain-controlled cyclic triaxial tests were conducted on the low density ice specimens. The specimens were tested at frequencies of 0.3, 1.0, and 5.0 Hz, confining pressures of 0, 175, 350, and 700 kN/m², temperatures of -1, -4, and -10°C and axial strain amplitudes between 10^-3 and 1.6 x 10^-2%. Each specimen was generally subjected to 20 cycles of loading at a given axial strain amplitude, confining pressure, and frequency of loading. Dynamic properties were determined at the 10th cycle of loading.

Influence of Axial Strain Amplitude. The values of dynamic Young's modulus and damping ratio were plotted against the log of percent axial strain amplitude. Typical test results are shown in Fig. 2. The solid lines shown in Fig. 2 represent the least squares best fit line of the data set shown. Considering the data shown and similar relationships for all test conditions, the least squares best fit lines vary slightly from one another, but their slopes do not appear to be influenced by frequency, confining pressure, or temperature. Hence, the relationship between dynamic properties and axial strain amplitude was assumed to be independent of these test parameters.

Overall, dynamic Young's modulus decreases of the order of 10 to 15% over the range of axial strain amplitudes considered. Over the range of test conditions considered, the value of dynamic Young's modulus ranged from 1.2 to 4.4 GN/m².
Following a close examination of the data sets for damping ratio for all test conditions it was concluded that there was no identifiable relationship between damping ratio and confining pressure for the majority of the specimens tested. As a consequence of this, the data for all confining pressures were plotted together. Damping ratio, in general, increases slightly with increasing axial strain amplitude. Over the range of test conditions considered, the value of damping ratio ranged from 0.01 to 0.12.

The relationship between dynamic Young's modulus and/or damping ratio and other test variables considered, can be established by interpolation of the results presented in Fig. 2 and similar results for all other test conditions considered in the program, at a specified axial strain amplitude. A strain amplitude of $4.4 \times 10^{-3}\%$ was selected for this purpose. Another strain amplitude could have been selected without significantly changing the conclusions reached in the following paragraphs.

**Influence of Confining Pressure.** The relationship between dynamic Young's modulus and confining pressure is shown in Fig. 3. The results shown indicate dynamic Young's modulus increases with increasing confining pressure. The increase is significant up to 350 kN/m$^2$. For confining pressures greater than 350 kN/m$^2$ the increase is gradual. The frequency and temperature of testing do not appear to have an influence on the relationship. The increase of dynamic Young's modulus of ice with confining pressure might be associated with changes in the microstructure of the ice under confining pressure. Microfissures might close when a specimen is subjected to a high confining pressure. This would lead to a specimen with a higher dynamic modulus. Conversely, they might open when a specimen is subjected to a lower confining pressure which would lead to a lower dynamic modulus. This tendency was exemplified by the fact that when the confining pressure was released from the triaxial cell and the specimen was not allowed to deform there was a gradual increase in load on the specimen. This load might be associated with the microfissures opening. As previously mentioned, there was no identifiable relationship between damping ratio and confining pressure for the majority of the specimens tested.

**Influence of Temperature.** The relationship between dynamic Young's modulus and temperature is shown in Fig. 4a; the relationship between temperature and damping ratio is shown in Fig. 4b. The results shown indicate the dynamic Young's modulus increases with decreasing temperature. The increase is approximately 100% for a temperature decrease from $-1$ to $-10^\circ C$. The rate of increase appears to be slightly greater for the temperature range $-1$ to $-4^\circ C$ than for the temperature range $-4$ to $-10^\circ C$. Density, confining pressure and frequency do not appear to have a significant influence on the relationship. In general, the damping ratio decreases with temperature. The influence of temperature appears greatest in the range $-1$ to $-5^\circ C$. 

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Influence of Frequency. The relationship between dynamic Young's modulus and frequency is shown in Fig. 5a. Dynamic Young's modulus increases only slightly for an increase in frequency from 0.3 to 5.0 Hz. The relationship between damping ratio and frequency is shown in Fig. 5b. Damping ratio decreases as frequency increases from 0.3 to 5.0 Hz at a test temperature of -1°C. Damping ratio decreases as frequency increases from 0.3 to 1.0 Hz and increases as frequency increases from 1.0 to 5.0 Hz for test temperatures of -4 and -10°C.

Influence of Density. The results from cyclic triaxial tests conducted on ice specimens at an average density of 0.90 g/cc have previously been reported [2]. It is instructive to compare the results obtained in the previous study to those obtained
in the present study as shown in Fig. 6. It is apparent dynamic Young's modulus increases significantly with density. For all practical purposes, the rate of increase appears to be independent of confining pressure, temperature and frequency. Based on a comparison of the damping ratios of low and high density ice specimens for all test conditions considered, it may be concluded that no well-defined relationship exists between damping ratio and density.

Comparison of Dynamic Properties of Ice

Basis for Comparison. The longitudinal and compression wave velocity appear to be the most convenient terms to use as a basis for comparison of the results from previous studies to those of the present study. The longitudinal wave velocity, $V_L$, was calculated from the results of the present study using

$$ V_L = \sqrt{\frac{E_d}{\rho}} $$

(3)

in which, $\rho$ = material density.

This equation was used to calculate the longitudinal wave velocity in the cylindrical specimens of the present study because the length of the specimen is much shorter than the wavelength of the compression wave that is propagated from the cap to the
base during cyclic loading. The compression wave velocity, $V_p$, may be computed from the longitudinal wave velocity using:

$$V_p = V_L \sqrt{\frac{1-\mu}{(1+\mu)(1-2\mu)}}$$  \hspace{1cm} (4)$$

in which, $\mu$ = Poisson's ratio.

Poisson's ratio was not determined for the low density ice tested in the research program. However, based on values reported by previous investigators [3,4,5,6] a reasonable value of Poisson's ratio for the ice tested would be $\mu = 0.35$. With this value the relationship between longitudinal and compression wave velocity becomes:

$$V_p = (1.27)V_L$$  \hspace{1cm} (5)$$

Wave Velocities. The results from the present study are plotted together with the results from previous studies in Fig. 7. It can be seen that the longitudinal wave velocity of the present study is between 2.3 to 2.5 km/sec and the compression wave velocity is between 2.9 to 3.1 km/sec. The longitudinal wave velocity from previous laboratory studies [3,5,6] associated with resonant frequency test procedures, is 2.7 to 3.1 km/sec; the compression wave velocity from field test results [7] associated with geophysical test procedures, is 3.2 to 3.8 km/sec. Overall it appears there is a favorable comparison. The difference between the results shown
may be attributed to differences in the test techniques used to measure the dynamic properties. The differences in the test techniques and their influence on the comparison of wave velocities are as follows:

1. Strain amplitude - the strain amplitude of the present study $(4.4 \times 10^{-3}\%)$ is greater than for the resonant frequency procedure (approximately $10^{-5}$ to $10^{-3}\%$) or the geophysical procedure (approximately $10^{-7}$ to $10^{-4}\%$). It was found in the present study that the elastic modulus of ice decreases slightly with increasing strain amplitude. Therefore, the wave velocities of the present study should be lower than the results associated with resonant frequency or geophysical procedures.

2. Frequency - the test frequencies in the present study (0.3 to 5.0 Hz) are much lower than for the resonant frequency procedure (approximately 1 to 10 kHz) or geophysical procedure (approximately 150 Hz). The present study indicates that the dynamic Young's modulus of ice increases with increasing frequency in the range of frequency from 0.3 to 5 Hz. Smith [5] reports that the modulus increases with frequency in a very high range of frequency, 800 to 2800 Hz. Therefore, the lower values of wave velocities obtained in the present study compared to those obtained by previous researchers seem reasonable owing to the lower frequency of testing in the present study.

3. Temperature - the relationships shown are for temperatures of $-9^\circ C$ and colder. In this range the influence of temperature on the wave velocities shown should be small.

Damping Ratios. The results from the present study are plotted together with the results from previous studies in Fig. 8. The comparison appears to be favorable. The damping ratios from the present study at frequencies of 0.3 to 5.0 Hz, are in the range 0.02 to 0.08. The damping ratios from previous resonant frequency tests are in the range 0.01 to 0.03. The relative closeness of the comparison may be explained by a consideration of the strain amplitudes and frequencies of the different test procedures as follows:

1. Strain amplitude - the strain amplitude of the present study $(4.4 \times 10^{-3}\%)$ is greater than that associated with resonant frequency procedures. It was found in the present study that the damping ratio of ice decreases slightly with decreasing strain amplitude. Therefore, the damping ratios obtained in the resonant frequency studies should be smaller than those for the present study.

2. Frequency - the frequencies associated with resonant frequency tests are much greater than for the present study. The present study indicates that the damping ratio of ice increases with increasing frequency in the range.
1.0 to 5.0 cps. If the increase in damping ratio with increasing frequency is valid up to frequencies of the resonant frequency tests, the damping ratio obtained from the resonant frequency tests should be greater than those of the present study. Therefore, the combination of the decrease of damping ratio with decreasing strain amplitude and the increase of damping ratio with increasing frequency has apparently caused the damping ratios from the present study to be close to those of previous studies, i.e., the effects of strain amplitude and frequency tend to cancel each other.

Acknowledgements
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References
ABSTRACT

Surface wind direction is usually the result of a three-way balance within the planetary boundary layer between the Coriolis force, the large-scale pressure gradient and the viscous force. These winds are typically shifted 20° counterclockwise (CCW) from their free stream (geostrophic wind) level above the boundary layer. A five year study of atmospheric environmental data along the Alaskan Beaufort Sea Coast has shown that surface winds can be shifted as much as 180° from their free stream level directions in both the summer and winter seasons. For the summer season, the increased persistence of surface onshore (northeasterly and easterly) winds for this coastal area can be largely attributed to the arctic sea breeze effect which creates a time varying mesoscale pressure field. For the winter season, the abundance of west to southwest winds at Barter Island (64 km north of the Brooks Range) while Pt. Barrow (300 km north of the Brooks Range) is experiencing east to northeast winds is evidence of mountain barrier baroclinity with its associated mesoscale pressure field.

To accurately determine the amount of boundary layer shift in wind direction, both the synoptic wind field and the surface wind directions had to be known. The synoptic wind field determination was enhanced through the deployment of offshore pressure and position data relaying buoys in 1975 and 1976 and again in 1979. The calculated geostrophic winds were confirmed by matching them to above boundary layer winds from rawinsonde and pilot balloon data at experimental sites. The surface wind data, which has an important relationship to nearshore ocean currents and sea ice movement, was collected from a network of towers on islands and coastline sites around Simpson Lagoon, Harrison Bay and Prudhoe Bay.

Introduction

The Alaskan Beaufort Sea coast is typically open to shipping from early July through...
early September, a period during which the edge of the arctic ice pack moves offshore. The discovery and development of the Prudhoe Bay oil fields (sixties) has led to increased ship traffic and increased probability of permanent man-made and natural offshore drilling platforms in the Beaufort Sea. Since nearshore current flow along the Alaskan arctic coast is primarily a function of surface wind direction, the importance of coastal meteorology with direct application to transport of detritus, biota, spilled oil and small ice floes has grown.

Two mesoscale phenomena, the arctic sea breeze (summer) and mountain barrier baroclinity (winter) have been shown to become the overriding meteorological features for a significant percentage (25%) of their season of occurrence and to produce surface wind directions up to $180^\circ$ from those predicted by National Weather Service (NWS) synoptic charts.

The Sea Breeze

The intensity, duration, and extent of the sea breeze circulation (a mesoscale feature) are mainly determined by horizontal gradients in the amount of heat supplied by the earth's surface to the atmosphere (Defant,[1]). Moritz [2] after investigating historical data from Pt.Barrow, Alaska, discovered that calculated geostrophic wind speeds and direction frequencies for July were similar for winds with east and west components, but measured surface east winds had greater magnitude and frequency than west winds. He suggested that the surface temperature contrasts at Alaska's north coast produce an added pressure gradient component that is not recorded by the existing National Weather Service (NWS) synoptic observation network.

Fig.1 shows the positions of pressure stations (P) within the land-based grid for August 1976 and 1977. The primary study area (rectangle A) has surface wind (10 meter) stations offshore on the Jones Islands (Pingok and Cottle), Cross Island and the McClure Islands (Narwhal) and onshore stations at Tolaktovut Point, Oliktok and Deadhorse. These stations exist in an area where the summer tundra-ocean thermal contrast remains positive (land always warmer than water).

Data Analysis

To increase the resolution beyond that of the existing NWS pressure grid (August 1977) the microbarograph network (Fig. 1) was extended to include two pressure and position buoys in the polar ice pack. These data provided the basic input for a two-dimensional least squares fit (TDLSF) to a cubic surface (Krumbein, [3]) which produced a pressure map for the study area and allowed for calculation of geostrophic velocities at field data sites where 10 meter winds were measured. Fig. 2 is an example of a TDLSF plot on a sea breeze day. The surface winds at the indicated stations are shown using standard NWS notation for the wind velocity arrows (→), short flag = 1.5 to 3.5
The calculated geostrophic wind for Cottle Island on August 16, 1976 (0000 GMT) was 0.93 m/s from 188.5° with a measured surface wind of 5.28 m/s from 90°. The land-sea temperature difference over 15 km (Prudhoe Airport to the coast [P-C]) was 7.8°C.

The above TDLSF technique was used for twice daily comparison with NWS surface pressure maps. To compare surface winds on Cottle Island with the geostrophic winds on a 3-hourly basis the technique was used to fit a plane surface to pressure data from Deadhorse, Lonely, Umiat and one offshore buoy (closest to coast). The time segment was from August 13 to 23 and August 30 to September 3, 1976. In Fig. 3 the geostrophic wind direction ($D_G$ ordinate) is plotted as a function of the surface wind direction ($S_{10}$, abscissa). The sea breeze influenced data points (SB) are contained within the dashed lines. The geostrophic winds within a 60° band from 195° -255° (SW quadrant) correspond to surface winds in a 60° band from 90° -150° (SE quadrant). The sea breeze had offset weak geostrophic winds in the boundary layer to produce an average "turning" of 120° CCW from the free stream level to the surface to increase the frequency of surface winds from the east. Planetary boundary layer turning should normally show winds shifted approximately 20° counterclockwise (CCW) from their free stream direction to their 10-meter direction.

Fig. 4 is a histogram of wind speed and direction with the top scales representing direction (from) frequencies and indicated by bars. The bottom scales represent speed frequency and are indicated by printed numbers. Allowing for synoptic effects Fig. 3 above shows that a large percentage of the asymmetry in the wind direction data (Fig. 4) must also be a thermally induced mesoscale effect (sea breeze).

Time series of surface wind data from coastal stations and islands were examined with a rotary spectrum technique (Gonella, [4]), that divides the variance into clockwise (CW) rotating variance (negative frequency) and counterclockwise (CCW) rotating variance (positive frequency). O'Brien and Pillsbury [5] state that absence of significant peaks at the 24-hours period in the CW part of the spectrum for at-sea buoys indicates absence of sea breeze ocean surface circulation. Fig. 5 is a semi-log plot of rotary spectra for August 1976 (Tolaktovut Point, coastal station) with 95% confidence limits (C) and band-widths (B) indicated. The vertical axis has units of spectral density ($[m^2/s^2]/c/3h]$) with the horizontal axis in units of cycles/day. The significant peak occurring near -1 cycle/day (24-hour period) is the CW rotating contribution from the sea breeze. Similar peaks in Deadhorse and Narwhal Island spectra (not shown) are evidence of sea breeze influence in at least a 40 km band centered on the coastline.

Meteorological pilot balloon data (August 1977), was converted to horizontal wind velocity versus height as shown in Fig. 6 for August 15, 1977. Radiosonde data (1526 and 1810 ADST) indicated the inversion layer top (ILT) to be at 200 meters with a
temperature of 17°C and a 5°C surface temperature. The free stream wind remained constant at approximately 220°T. The 1524, 1810, and 1910 profiles show temporal CW rotation of the surface wind vector during times at sea breeze influence and all show large CW rotation of the wind vector with height (greater than 100° to the inversion layer top). The Prudhoe-Coast (P-C) temperature difference (1500 ADST) was 12.8°C.

**Mountain Barrier Baroclinity**

Mountain Barrier Baroclinity (MBB), a predominant wintertime phenomenon, is responsible for up to 180° surface wind shifts along the Alaskan arctic coast between Pt. Barrow and Barter Island (a distance of less than 500 km) during moderate wind conditions. Schwerdtfeger [6,7], states that a stable air mass moving toward a mountain range (Fig. 7) without heating from below induces baroclinity by causing a downward tilting (away from the obstacle) of isobaric and isothermal surfaces. This results in a mesoscale pressure gradient force away from and perpendicular to the mountain range with a maximum value near the inversion layer top (level 1, Fig. 7). Therefore, coastward northerly flow induces a strong westerly component of the wind near the bottom of the cold air layer over the north slope of the Brooks Range, deflecting low level flow (between 1 and 0, Fig. 7) to the east.

A unique data set has been obtained through the implementation of the University of Washington's Arctic Basin Buoy (ABB) array (mid-February 1979). These buoys, which were air dropped on the arctic ice pack, transmit pressure and position data to the TIROS-N satellite and use the ARGOS data collection system. Data from two offshore buoys with comcomitant onshore station data have allowed the calculation of more precise 3-hourly geostrophic winds for the north coast of Alaska.

**Data Analysis**

Fig. 8 is a section of a NWS synoptic chart for 11 March 1979 (1200 GMT) with the Brooks Range cross hatched. F,L,G,H and S are surface wind data locations at Pt. Barrow, Lonely, Prudhoe Bay, Barter Island (from west to east along the Alaskan coast) and at a Russian ice station respectively. The winds are shown again using NWS notation. B_{13} and B_{14} are locations of buoys (ABB) which furnished surface pressure (accuracy ± 1.5 mb) out on the ice pack (no wind direction measurements). The pressure contours are in millibars (mb) and the numbers adjacent to the locations are surface pressure minus 1000 mb. For the stations shown, the NWS analysis used data from S,F and H, only, to construct contours. The surface winds at S and F and the surface pressure at B_{13} (1037.2 mb) are reasonable for the offshore high pressure system (anti-cyclonic flow) shown. However, the winds at G and H (opposite to what the NWS pressure pattern indicate) and the surface pressure at B, show inadequate analysis. The actual pressure change from B_{13} and B_{14} was 15.3 mb (map contours indicate 8 mb).
while the actual change from F to H (a comparable distance) is 5.6 mb. Simultaneous Barter Island(M) rawinsonde (radio tracked meteorologically instrumental balloon) data (Fig. 9) shows a strong temperature inversion up to 400 m and approximately 150° of wind turning within the inversion layer. The computed geostrophic wind for this time using pressure data from buoys B13, B14, and Pt. Barrow was from 36° at 15 m/s. The above figures demonstrate the existence of an additional gradient force away from the Brooks Range axis since frictional wind turning usually accounts for only 20-30° of CCW change from the upper free stream direction to the surface (10-meters).

Fig. 10 is a scattergram showing the 3-hourly surface wind direction at Barter Island (ordinate) versus the surface wind direction at Pt.Barrow (F) (abscissa) for March 1979. Since Pt. Barrow is more than 300 km north of the Brooks Range, the surface wind should vary from the geostrophic wind direction by less than 30° CCW. The data points enclosed by the solid line represent simultaneous surface winds at Barter Island and Pt. Barrow with an average directional difference of 140°.

To compare surface winds at Barter Island with geostrophic winds, the above TDLSF technique [3] was applied to calculate three hourly geostrophic winds for the area enclosed by the B13 - B14 - Pt. Barrow array since it was the only one not influenced by the Brooks Range. This combination was a good fit to the measured surface wind directions at Pt. Barrow (scattergram not shown) and was representative of the 800 m to 2000 m flow (above the inversion layer) in the Barter Island rawinsonde data (Fig. 9 is an example of this). The scattergram (Fig. 11) showing 3-hourly wind directions at Barter Island (ordinate) versus the computed (B13 - B14 - Pt. Barrow) geostrophic winds (abscissa) for March 1979 is similar to Fig. 10. This is no surprise since surface wind directions at Pt. Barrow should be within 30° of the geostrophic wind directions for the same time periods. Since the upper level flow in the Barter Island rawinsonde data also fits the geostrophic winds calculated for B13 - B14 - Pt. Barrow area, the data points enclosed by the solid line (Fig. 11) take on added significance as evidence for MBB averaging 140° of turning from the geostrophic (free stream) wind level to the surface. This is one major reason for the predominance of surface westerly winds at Barter Island in the winter months, while Pt. Barrow has a predominance of easterly winds (Brower, et al., [8]).

Fig. 12 derived from historical compilations [8] of monthly wind direction frequency histograms is a plot of the percentage difference (Δ%) between total surface wind direction frequencies in the east to northeast (E-NE) quadrant and west to southwest (W-SW) quadrant for three coastal stations. These are Lonely, Oliktok and Barter Island at approximately 275 km, 165 km and 64 km respectively from the foothills of the Brooks Range. Positive (Δ%) implies more E-NE winds than W-SW winds while negative (Δ%) implies more W-SW winds than E-NE winds. The October through April months (when
the atmospheric boundary layer is most stable) show a great disparity between Lonely (farthest from the mountains) and Barter Island (closest to the mountains). In the months of May through August, when insolation exceeds 20 hours and the land snow cover is depleted or completely gone, the land boundary layer approaches neutral stability and the three coastal stations become similar. MBB is minimized and the mesoscale thermal effect of the coastline becomes a major influence (Kozo, [9]) on coastal surface winds.

Conclusions

The dominant physical factor causing the abnormal atmosphere boundary layer turning of the wind on the Alaskan Arctic coast in summer is the sea breeze. The large diurnally varying land-sea thermal contrast, clockwise rotation of the surface wind vector, and surface wind flow in opposition to moderate offshore gradient flow are factors and evidence of this mesoscale phenomenon.

MBB is a major physical process responsible for the wintertime abundance of W-SW winds from Barter Island to Prudhoe Bay along the Alaskan arctic coast. It is a seasonal effect related to stability of the atmospheric boundary layer. This stability over land, in turn is affected by surface albedo which when high (winter) promotes a stable layer or when low (summer) establishes a near neutral boundary layer. Previous attempts at relating wintertime ice movement in coastal regions to actual surface winds must be reinvestigated for locations within the MBB "shadow" of the Brooks Range.

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References


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**FIG 2.** TWO-DIMENSIONAL FIT TO SURFACE PRESSURE NETWORK ON AUGUST 16, 1976 (0000 GMT). C, N, T REPRESENT COTTLE ISLAND, NARWHAL ISLAND AND TOLAKTOUVUT POINT, RESPECTIVELY (CONTOURS — MD).
FIG. 3 GEOSTROPHIC WIND DIRECTION ($D_G$) AS A FUNCTION OF SURFACE WIND DIRECTION ($S_{10}$) IN AUGUST 1976

FIG. 4 HISTOGRAM OF SURFACE WIND SPEED AND DIRECTION FOR AUGUST 1976, 1977 AND 1978 COMBINED (FROM COTTLIE ISLAND)

FIG. 5: TIME SERIES ROTARY SPECTRA OF SURFACE WIND VELOCITY DATA FOR AUGUST 1976 COLLECTED FROM TOLAKTOVUT POINT

FIG. 6: PROFILES OF WIND SPEED AND DIRECTION FROM PINGOK ISLAND ON AUGUST 15, 1977 FOR ADST TIMES OF 1524, 1810 AND 1910
FIG 7. CROSS SECTION OF THE FLOW OF A STABLE AIR MASS TOWARD A MOUNTAIN RANGE (SCHWERDTFEGER, 1974)

FIG 8. SECTION OF A NWS SYNOPTIC CHART FOR 11 MARCH 1979 (BROOKS RANGE CROSS-HATCHED)
F, L, G, H, AND S REPRESENT SURFACE WIND DATA LOCATIONS AT PT. BARROW, LONELY, PRUDHOE BAY, BARTER ISLAND AND A RUSSIAN ICE STATION RESPECTIVELY. \( b_{13} \) AND \( b_{14} \) ARE LOCATIONS OF ABB BUOYS (PRESSURE CONTOURS—mb). THERE WERE NO WIND MEASUREMENTS AT \( b_{13} \) AND \( b_{14} \).

Fig 10: Surface wind direction at Barter Island versus surface wind direction at Pt Barrow

Fig 11: Surface wind direction at Barter Island versus computed geostrophic wind direction

Fig 12: Monthly percentage difference (Δ%) between total surface wind direction frequencies in the E–NE quadrant and the W–SW quadrant for Barter Island, Oliktok and Lonely
INFLUENCE OF AN ICE LAYER ON STORM SURGE AMPLITUDES

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Abstract

Hourly observed and predicted sea level variations in the St. Lawrence estuary and in the Gulf of St. Lawrence, for a period of eleven years (1965-1975) were examined to study the influence of ice cover on storm surge amplitudes. The term "storm surge" refers to the variations in sea level due to atmospheric pressure gradients and wind stress. Statistical analysis of the data shows that maximum amplitudes of hourly positive surges of at least 3.0 m occurred only during ice-free months, whereas maximum amplitudes of hourly negative surges occurred during the period of ice cover. In the Gulf of St. Lawrence, monthly mean positive surges occurred in spring and summer and monthly mean negative surges occurred in autumn and winter. However, in the estuary, monthly mean of surges does not appear to be negative in the presence of ice. The development of storm surges as a function of the rate of development of the storm (i.e. as a function of the rate of development of meteorological forcing) is also studied.

1. Introduction

Storm surges fall into the classification of long waves whereas wind waves are referred to as short waves. Here the words long and short refer to period (and wavelength). Wind waves have periods of the order of few seconds, whereas storm surges have periods of the order of several hours. It has been fairly well established that wind waves suffer attenuation in the presence of ice [2, 3, 9, 11, 12]. Sverdrup [10] considered the influence of an ice layer on tides in the North Siberian Shelf. Einarsson [4] appears to be the first to clearly state that the influence of an ice layer on long waves might be different from that on wind waves.

Lisitzin [7] considered the influence of an ice layer on storm surges in the Baltic Sea. Her studies showed that the storm surge amplitudes were smaller when ice was present. Henry [5] found similar results in the Southern Beaufort Sea.
Murty and Polavarapu [8] made a preliminary study of the influence of an ice layer on storm surges in the Gulf of St. Lawrence and the St. Lawrence estuary. Their results agreed with those of Lisitzin's [7] in one case, disagreed in a second case and there was neither agreement nor disagreement in a third case.

The study of Murty and Polavarapu [8] is extended here in several respects. First, the water bodies considered in this study are the Bay of Fundy and part of the Atlantic coast, in addition to the Gulf of St. Lawrence and the St. Lawrence estuary. Second, a distinction has been made between positive and negative surges and this distinction is justified by the results which show somewhat different influences of ice cover on positive and negative surge amplitudes. Third, in addition to the hourly residues, mean monthly residues were also examined for the influence of ice on them.

2. Storm surge amplitudes in Eastern Canadian water bodies

Figure 1 shows the geography of the Gulf of St. Lawrence, the Bay of Fundy and part of the Atlantic coast. Data from the following stations was used in this study: North Sidney, Port-Aux-Basques, Pictou, Charlottetown, Harrington Harbour, Lark Harbour and Sept-Iles.

Statistical analysis of the storm surge data showed that the highest positive surges of amplitudes of about 3.0 m occurred only during the months May to August and December, when no or little ice was present. The highest negative surges occurred during January and February, when ice was present [1].

Figure 2 shows the geography of the St. Lawrence estuary. The stations used in this study are: Sainte-Anne-Des-Monts, Baie-Comeau, Pointe-Au-Père, Tadoussae, Rivière-du-loup, Saint-Joseph-de-la-Rive and Saint-Jean-Port-Joli. In the St. Lawrence estuary, highest positive surges occurred during June, August, September, October and December and extreme negative surges occurred during December and February. Also in the St. Lawrence estuary, the duration of the positive surges is shorter on the north shore and the duration of the negative surges is shorter on the south shore.

The above remarks pertain to hourly values of the storm surges which are calculated as the residue after subtracting the hourly values of the predicted astronomical tides from the observed hourly water level values. However, when we examine the monthly residues, the results are somewhat different. In the Gulf of St. Lawrence, the extreme positive values usually occur during October, December and January, whereas the negative extreme surges occurred during June to September.

Figure 3 shows the hourly residues for January 1972 at Pointe-du-Chêne, as an example. Table 1 lists the month in which maximum surges occurred during the eleven-year period of this study.
Figure 1: Geography of the Gulf of St. Lawrence, the Bay of Fundy and part of the Atlantic coast.
Figure 2: Geography of the St. Lawrence Estuary.
Figure 3: Hourly residues for January 1972 at Pointe-du-Chêne.
Table 1
Occurrence of Maximum Storm Surge Amplitudes

<table>
<thead>
<tr>
<th>Station</th>
<th>Water Body</th>
<th>Year</th>
<th>Month</th>
</tr>
</thead>
<tbody>
<tr>
<td>St. Jean (N.B.)</td>
<td>Bay of Fundy</td>
<td>1975</td>
<td>December</td>
</tr>
<tr>
<td>Port-Aux-Basques</td>
<td>Atlantic coast</td>
<td>1975</td>
<td>May, June</td>
</tr>
<tr>
<td>Pointe-du-Chêne</td>
<td>Gulf of St. Lawrence</td>
<td>1972</td>
<td>January</td>
</tr>
<tr>
<td>Rivière-au-Renard</td>
<td>Gulf of St. Lawrence</td>
<td>1975</td>
<td>August</td>
</tr>
<tr>
<td>Sept-Iles</td>
<td>Gulf of St. Lawrence</td>
<td>1975</td>
<td>June</td>
</tr>
<tr>
<td>Baie-Comeau</td>
<td>St. Lawrence Estuary</td>
<td>1975</td>
<td>August</td>
</tr>
<tr>
<td>Pointe-au-Père</td>
<td>St. Lawrence Estuary</td>
<td>1975</td>
<td>September</td>
</tr>
<tr>
<td>St. Joseph-de-la-Rive</td>
<td>St. Lawrence Estuary</td>
<td>1975</td>
<td>September</td>
</tr>
<tr>
<td>Rivière-du-loup</td>
<td>St. Lawrence Estuary</td>
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<td>June, August</td>
</tr>
<tr>
<td>Québec</td>
<td>St. Lawrence Estuary</td>
<td>1972</td>
<td>December</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1975</td>
<td>August</td>
</tr>
<tr>
<td>Tadoussac</td>
<td>St. Lawrence Estuary</td>
<td>1972</td>
<td>December</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1975</td>
<td>August, October,</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>December</td>
</tr>
</tbody>
</table>

3. Effect of meteorological forcing

It was shown that the rate of growth of the atmospheric forcing terms is important in determining the amplitudes and times of occurrence of the maximum surges [6]. The amplitude of the surge is directly proportional to the rate of growth of the storm whereas the time of occurrence of the maximum surge is inversely proportional to the rate of growth of the storm. One cannot automatically assume that the observed water level due to a stronger weather system will be greater because there is a difference in the time taken to reach the maximum amplitude, for different growth rates. This time difference could be important when the surge is superimposed on the tide to give the total water level. A surge produced by a storm with a slower growth rate could produce greater water level disturbances if it so happens that the time of maximum surge coincides with the high tide.

Figure 4 shows the amplitude of the surge (in dimensionless form) on the ordinate versus a dimensionless time on the abscissa. Here \( f \) is the coriolis parameter, \( h \) is the water depth, \( g \) is gravity, \( \eta \) is the deviation of the water level from its undisturbed position and \( A \) is an amplitude function for the storm. The dimensionless time is given by \( b = f \cdot t \) where \( t \) is physical time. Holland [6] calculated the growth rate for a few cases. These calculations have been extended here to include growth rates of 15, 20, 25 hours etc. It can be seen that as the
Figure 4: Dimensionless storm surge amplitude (ordinate) versus dimensionless time (abcissa) for different growth rates of the storm (in hours denoted on each curve).

storm growth period increases, the amplitude of the surge decreases. However, once the growth period approaches 20 hours, the sensitivity of the surge amplitude (and the time of occurrence of the maximum surge) to the growth period is considerably diminished.

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References


INTERNAL WAVES IN DAVIS STRAIT AND THEIR
MEASUREMENT WITH A REAL-TIME SYSTEM

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ABSTRACT

Large internal waves were observed in Davis Strait during offshore drilling operations in 1979. They were detected by the influence of the wave-induced currents on the drill ship and by the surface rip propagating with the waves. In 1980 a current measuring system consisting of 7 electromagnetic current meters distributed on a 3 mooring array was deployed to intercept the waves before their arrival at the rig. Data were transmitted every 60 s to the ship and displayed on various devices. When currents exceeded preset thresholds audio/visual alarms were sounded and the ship was reoriented bow into the wave until it had passed, seastate and wind permitting. The system proved to be effective giving up to 100 minutes warning of internal waves approaching the site. From the 1980 data we have indentified these waves as internal solitons with periods of 10 to 12 minutes and amplitudes of 20 to 45 m. Phase speeds are typically 0.70 to 0.90 m/s superimposed on tidal currents of the order of 0.30 m/s. The solitons were observed to travel in the same general direction as the ebb currents and their arrival time at the rig was well correlated with the ebb tide. Typically the waves were observed singly or in pairs with the smaller wave leading. At other times rank-ordered trains of waves (4 to 5 peaks) were measured. Wave-induced velocities in the upper layer were typically 0.30 to 0.80 m/s and represented a 100 to 200 percent increase over background current speeds. Solitons were observed regularly at the semidiurnal period (12.5 hours) suggesting that the tides in this area are important in forming these waves. If the solitons are formed from a lee wave produced by the tidal currents flowing over uneven bathymetry, the 1980 data indicate that the mouth of Hudson Strait is a likely source area.
1. INTRODUCTION

Internal waves occur within subsurface layers in the ocean that are stratified because of temperature and salinity variations. They are often observed in two distinct frequency bands: the tidal-inertial band with frequencies of 0.07 to 0.08 h⁻¹ and a much higher band with frequencies of 6 to 10 h⁻¹. It is in this latter range that large nonlinear waves, classed as internal solitons, have been observed in several areas of the world. Such waves were noted by Perry and Schimke[1], in the Andaman Sea near the west coast of Thailand, and discussed in terms of their effect on deepwater drilling in this area by Osborne et al.[2]. The internal wave measurements from the Andaman Sea were found to compare favourably with two-layer soliton theory based on the Korteweg-de Vries equation[3]. Similar waves have also been observed in the Sulu Sea by Apel et al.[4] and their measured properties were used to characterize the wave motion by Holbrook et al.[5].

Large internal waves were first observed in Davis Strait during offshore drilling operations in 1979 in 350 m of water (see Fig. 1.). The increase in near-surface current speed, up to 2 knots in 2 to 3 minutes, was responsible for large deflections of the dynamically positioned drill ship. To better define these waves and to provide a warning of their strength and time of arrival at the ship in 1980, while the HEKJA 0-71 well was drilled to depth and tested, a current metering system was designed and installed to provide real-time measurements to the rig. In this paper the instrumentation is briefly described and some internal wave data collected in 1980 are presented. From these data a number of wave characteristics are derived and discussed.

2. INSTRUMENTATION

Currents were measured at 15, 30 and 110 m (nominal) depth with 7 I0195RX electromagnetic current meters distributed on 3 subsurface moorings as shown in Fig. 2. Each instrument was wired through a 120 m tether to a spar buoy which contained the power supply for the current meters (lead-acid batteries), A-D converters and the VHF transmitters. The spar buoy design is discussed by Hodgins and Lea[6]. Data were transmitted every 60 s from each buoy and displayed on a deck unit in the control room of the drill ship as speed and direction in digital form, both visually and on hardcopy, as current speed on panel meters (which activated the A/V alarms when preset threshold values were exceeded) and as current speed on strip charts. The data were also logged on magnetic cassette tapes for later analysis. Additional data on the thermal structure were collected from a moored Aanderaa thermistor chain sampled every 120 s. The moorings were placed around the drill ship as shown in Fig. 1 (insert).
Fig. 1. Map of Davis Strait showing the observation site at HEKJA 0-71 and the mooring locations (insert).

Fig. 2. Configuration of the current metering system. The electromagnetic current meters are indicated by the circled numbers 1 to 7.
The first warning of the sudden, large currents produced by an internal wave was obtained when the wave passed mooring 1/2 (containing current meters 1 and 2) triggering the alarms on the appropriate panel meters. Current speed estimates were obtained by the operations personnel primarily from the strip chart recording and speed and direction data were taken from the hardcopy output. Confirmation of the wave progress was obtained as it passed the closer moorings, 3/4 and 5/6/7 at 2 and 1 n.m. respectively. Typical warning times were 100, 50 and 25 minutes at 4, 2 and 1 n.m.

The current metering system was deployed on August 1, 1980 and recovered on August 29 following mechanical failure of two tethers. One refurbished mooring was then redeployed on September 5, 1980 at 1 n.m. with two additional meters at 15 and 30 m depth functioning over the bow of the drill ship. All instruments were recovered on October 1, 1980. Data from the meters at 15 and 30 m were vector averaged over 60 s to remove surface wave velocities whereas the observations from 110 m were averaged over a 2 s sampling interval. The current meters, manufactured by InterOcean Systems Inc. of San Diego, have an accuracy of ±2 cm/s for speed and ±2" for direction.

The self-recording thermistor chain returned good data from July 28, 1980 to August 4, 1980 with a measurement accuracy of ±0.15°C. Eleven thermistor probes were spaced uniformly in the chain between 12 and 122 m. Several internal waves were thus measured simultaneously on both the thermistor chain and on the current metering system.

Throughout the first deployment period in early August, CTD data were obtained within 15 km of the site to provide information on the background density structure and to sample the passage of a few internal waves. On two occasions the effects of large internal waves can be seen in the CTD data, and in one case, the same wave can be found in all three data sources.

3. OBSERVATIONS

The strip chart outputs shown in Fig. 3 illustrate the appearance of the internal waves on the drill ship instrumentation. Here 5 waves are discernible above the background (ebb tide) current with the largest current speed peak produced by the second wave in the train, and smaller peaks thereafter. The largest currents, between 1.9 and 2.3 knots, are found at 30 m depth and generally exceed those measured at 15 m on large waves. The wave periods shown here, about 10 to 12 minutes, are typical of other waves observed in 1980, as is the interval of 20 to 40 minutes between waves in the train. The code "259-1" gives the "Julian Date - 1st ebb tide of the day". Similar codes are used below.
Fig. 3. A train of 5 large waves is shown in the strip chart records for a portion of the ebb tide on September 15, 1980. The charts are aligned in time (GMT). The upper chart corresponds to measurements from the mooring at 1 n.m. (uppermost instrument not functioning). The lower chart shows measurements made at the drill ship with current meters 3 and 4. The vertical scale is in knots from 0 to 4 and time is increasing toward the left. Each step in the current speed traces corresponds to an elapsed time of 1 minute.
Trains of waves such as shown in Fig. 3 were not as common as single waves or pairs of waves. In many cases the smaller of two waves would arrive first. An example of this is shown in Fig. 4 where two large amplitude waves are readily seen in the thermocline data. These two waves, marked a and b, can be traced from mooring to mooring in the current speed measurements, although at mooring 5/6/7 (1 n.m.) additional speed peaks are evident before and, perhaps, after the largest pair.

Using the change in the $2^\circ$C isotherm level as a measure of the thermocline (and pycnocline) depression, wave amplitudes of 28 and 43 m are obtained. The thermocline data also provide wave periods of 10 to 15 minutes, consistent with the current speed time-series data.

We believe that wave 216-2b was also sampled at 19:16 h about 10 n.m. WNW of the ship during the CTD survey. Temperature profiles bracketing this period are shown in Fig. 5. The profile at 19:16 h shows a pronounced deepening of the thermocline of about 40 m, consistent with the passage of an internal wave and the subsequent measurements at the thermistor chain. The ship's captain (at the CTD sampling station) also noted the presence of a narrow but long band of choppy surface water moving past his ship at the time and he identified the phenomenon as a "tide rip".

Estimates of wave speed were obtained from events 216-2a and 216-2b (Fig. 4) by noting the travel time, $\Delta t$, between moorings. The background currents along the travel path of the wave, $R_L$, (tidal plus residual) were obtained by low pass filtering the time-series current data from 30 m depth. The direction of wave propagation was calculated from the peak wave-induced velocity components (total current minus low-passed signal) averaged between adjacent moorings. The distance travelled by the wave, $L$, was then found by projecting the line joining the moorings onto the direction of propagation defined by this average direction. The wave speed, $c$, was calculated from

$$c = \frac{L}{\Delta t} - R_L$$  \hspace{1cm} (1)

For wave 216-2a, $c$ averaged 0.75 m/s between the moorings at an average direction of 42$^\circ$. Wave 216-2b had an average speed of 0.87 m/s along a direction of 55$^\circ$. The background currents averaged 0.31 m/s. Knowing $c$ and the wave period, $T$, the wave length, $\lambda$, was estimated from

$$\lambda = (c + R_L) \cdot T$$  \hspace{1cm} (2)

For example, $\lambda$ for wave 216-2b has a value of 788 m, defined here as the distance from shoulder to shoulder of the trough.
Fig. 4. Current speed time-series data on August 3, 1980 (upper panel) and isotherm levels measured at the thermistor chain (lower panel). Time in hours GMT.

Fig. 5. Temperature profiles showing an internal wave at 19:16 h GMT, August 3, 1980 (indicated ▼).
Estimates for the maximum upper layer velocity produced by wave 216-2b were derived from the filtered time-series data and averaged 0.82 m/s (in a range of 0.79 to 0.84 m/s) at 55°T on meters 2, 4 and 6. As shown in Fig. 4 the speed peaks are approximately 15% greater at meter 3 than meter 4 in this record, but virtually the same on meters 5 and 6. Based on the other observations there is, however, a tendency for the speed peaks either to be close the one another in the upper layer, or for the value at 30 m to exceed that at 15 m. Thus the result at mooring 3/4 is somewhat out of character in this respect.

Over the drilling season distinct internal waves or wave trains were observed on more than 90 ebb tides. The times of observation always coincided with late ebb, ranging from 10 to 120 minutes before low water at HEKJA (tidal heights were recorded at this site with a gauge mounted on the blowout preventer stack). The direction of wave propagation, and of the wave-induced velocities, from many observations average 54°T with extreme values ranging from 25°T to 85°T. These directions agree well with the ebb tidal current directions. Thus the internal waves were found to be propagating in basically the same direction as the ebb currents, with a total speed of about 2.4 knots.

4. CONCLUSIONS

The large internal waves observed in Davis Strait are similar in many respects to the internal solitons described by Osborne and Burch [3]. For example, in both areas, the Andaman Sea and Davis Strait, the wave progress can be detected by the surface "rip" of choppy water accompanying the wave. Also from simple two-layer soliton theory, where the upper layer thickness is less than the lower layer thickness, only a wave of depression (downward displacement of the pycnocline) is possible and this is what is observed in both locations.

Since theoretically the phase speed of solitons is proportional to their amplitude, the largest waves travel fastest. In ideal circumstances then ordered groups of waves evolve with the largest wave leading. The Andaman Sea data fitted well this type of model; however, in Davis Strait a consistent picture is not found. In some cases, wave trains (4 or 5 peaks) were so ordered but at other times, smaller waves would be seen leading the larger solitons. This may be a natural variability due to the mechanism by which they are formed and propagate to the site or due to differences in the way they evolve between the source area and the observation point. Unlike the Andaman Sea, where the waves were propagating from deep water (>2000 m) onto the shelf (∼1000 m), in Davis Strait they are travelling from a shallow area (∼300 m) across the shelf out toward the deep water of the Labrador Sea.
As noted earlier the wave length, $\lambda$, is approximately 750 to 800 m. This value, which is roughly twice the scale length, $\xi$, of a soliton, provides a ratio of $h/\xi \sim 0.8 \sim 0.9$ where $h$ is the water depth (350 m). In the Andaman Sea, $h/\xi \sim 0.4 \sim 0.6$ placing the waves there in a shallow water regime governed by the Korteweg-de Vries equation. Osborne and Burch [3] found good agreement between their measurements of the upper and lower layer velocity peaks and the values calculated from such a two-layer shallow water model. Applying a similar model to the Davis Strait data produced poorer agreement. Phase speeds were 0.84 m/s, observed vs 0.99 m/s from the model for the case of wave 216-2b. The maximum wave-induced velocity in the upper layer was observed at 0.42 m/s vs 0.73 m/s from the model and the scale lengths, $\xi$, were 394 m, observed, and 268 m from the model.

Thus the simple theory which does not take the pycnocline thickness into account predicts a faster, narrower and stronger (in terms of induced velocity) waves than observed. An improvement in modelling the Davis Strait solitons may be obtained with a better representation of the density variation, such as that proposed by Lee and Beardsley [7]. Also as indicated by the $h/\xi$ ratio the waves in Davis Strait lie more toward the intermediate regime examined by Joseph [8], Kubota et al. [9] and Chen and Lee [10]. These models may also offer an alternative formulation.

The internal waves were observed to propagate along a direction of about 55°T. Projecting this line back from HEKJA suggests that the mouth of Hudson Strait may be a source area. The correlation of the wave events with ebb tides also suggests that the predominantly semidiurnal tides are instrumental in forming the initial wave. A possible mechanism for the formation of this wave could be the creation of lee waves behind the ridge joining Resolution Island and the Button Islands on flood tides (Maxworthy [11]). The lee wave, on the turn to ebb is swept eastward over the ridge and forms a deep trough of brackish water. The tidal currents are strong, of the order 3 to 5 knots, near Hudson Strait but decrease rapidly away from the coast. At some point the lee wave would propagate radially outward and eventually pass over the HEKJA site.

The straight line distance between HEKJA and Hudson Strait is about 130 Km. An estimate for the mean tidal current between the source area and the site is about 1 m/s and assuming a value of 0.80 m/s for a wave speed, the time required to reach HEKJA is about 15 to 20 hours. This is much greater than the observed period of recurrence of the waves. Consequently waves formed on any one flood tide would not reach the site on the next ebb, and would have to survive a subsequent flood-ebb cycle having propagated only part way to the site. This is also true in the Andaman Sea where a similar generating mechanism has been suggested by Osborne and Burch [3].
5. ACKNOWLEDGEMENTS

This program was part of the 1980 drilling operation at HEKJA 0-71 in Davis Strait carried out by Aquitaine Company of Canada Ltd. as operator for the Baffin-Labrador Group of companies comprised of Aquitaine Company of Canada Ltd., Petro-Canada Exploration Inc., Home Oil Company Ltd., Hudsons Bay Oil and Gas Company Ltd., Murphy Oil Limited, Pancadian Petroleum Ltd., and SOQUIP.

6. REFERENCES

A THREE-DIMENSIONAL MODEL OF
NORTON SOUND UNDER ICE COVER

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For more than six months in a year, Norton Sound, Alaska, is covered with ice. The generation, transport and the presence of ice cover gives rise to a distinctive hydrodynamic characteristic which is drastically different from the ice-free condition. As part of a larger study program in conjunction with petroleum exploration in the Alaskan offshore region, a three-dimensional model has been developed to study the ice's generation, movement, and its modifying effect on the hydrodynamic regime of Norton Sound. It has been found that ice alters the tidal regime, turbulent energy transfer process, residual circulation, and the vertical density structure induced by the salt rejection process during the ice production period. Consequently the direction and magnitude of the ice drift pattern cannot be estimated by such simple rules as using a constant percentage of wind speed and fixed turning angle.

This paper illustrates the effect of tide, depth, boundary and ice/ice interaction on the ice's drift.
Due to the amount of cooling and the orientation of its opening, the formation, distribution and transport of ice in Norton Sound is vastly different from the conditions on the northern shores of Alaska. During the period from November through May, under the prevailing NNW wind, ice produced in the eastern portion of the sound is transported in a southwesterly direction. This process often creates an open area in the mid-eastern sound. By mid-winter, shallow areas on the eastern and southern shores are usually covered with shorefast ice. The size of ice floes in Norton Sound, based on observations, ranges from a few meters to a few thousand meters.

Tides in Norton Sound are of mixed types. Tides not only produce a counterclockwise residual circulation in Norton Sound, they also maintain a high turbulence level in the water column. When this interacts with wind and ice through vertical turbulent shear coupling, a complicated mechanism results. In other words, under identical wind stress, the direction and magnitude of ice drift in the tidal area are different from those in the area where tide is absent. Furthermore, the movement of ice floes is influenced by the amount of ice/ice interaction, particularly in the vicinity of the shorefast ice zone where momentum transfer between ice is extremely efficient. Due to these nonlinear components the direction and magnitude of the ice drift pattern cannot be estimated by such simple rules as using a fixed percentage of wind speed and constant turning angle. To study the ice movement, a three-dimensional hydrodynamic model has been coupled to an ice model by means of a turbulent closure technique which involves the computation of turbulent (subgrid-scale) energy densities in the flow field and at the ice/water interface. In addition, the temperature of ice and the amount of salt rejection during ice formation are also considered in the computation of the vertical density gradient of the water column, which in turn alters the rate of vertical exchange of momentum, mass, heat and energy.

Considering the length limitation, we can only give a brief description of the model used in our study. The model is based on the finite difference solution of a set of partial differential equations governing the momentum, mass, salt, heat, and turbulent energy balance in the water column and ice field. We present them directly in the finite difference form, adapting standard notation for summing, differencing and shifting. Using $i, j, k, n$ for the discrete representation in the $x, y, z, t$ domain, the finite difference formulation for water takes the following form:

$$\frac{\partial \zeta}{\partial t} \bigg|_{i,j,n} = - \sum_k \left[ \delta_x (h^x u) + \delta_y (h^y v) \right] \quad \text{at } i, j, n$$

(1)

where the variation of the water level $\zeta$ is derived from the continuity equation by vertical integration and $h$ is the layer thickness.
The momentum equation in the x-direction is

\[
\delta_t (h_x u_x) = - \delta_x (h_x u_x u_x) - \delta_y (h_x v_x u_x) - \delta_x (h_z w_x) + fh_{xxy} - \frac{1}{\rho_x} h_x \delta_x p
\]

\[
+ \frac{1}{\rho_x} \left[ h_x \left( E_x \delta_x z^2 t \right) + \delta_x \left( h_A x_\delta u_x \right) - \delta_y \left( h_A y_\delta u_x \right) \right]
\]

at \( i + \frac{1}{2}, j, k, n \)

where \( E_x \) is the vertical momentum exchange coefficient and \( A_x, A_y \) are horizontal exchange coefficients in x and y direction, respectively.

The momentum equation in the y-direction is:

\[
\delta_t (h_y v_y) = - \delta_x (h_x u_y v_y) - \delta_y (h_y v_y v_y) - \delta_x (h_z w_y) - fh_{xyy} - \frac{1}{\rho_y} h_y \delta_y p
\]

\[
+ \frac{1}{\rho_y} \left[ h_y \left( E_y \delta_z z^2 t \right) + \delta_x \left( h_A y_\delta v_y \right) - \delta_y \left( h_A x_\delta v_y \right) \right]
\]

at \( i, j + \frac{1}{2}, k, n \)

The mass-balance equation for salt is:

\[
\delta_t (\rho_s) = - \delta_x (\rho_x \delta_x) - \delta_y (\rho_y \delta_y) - \delta_z (\rho_z \delta_z)
\]

\[
+ \delta_x \left( h_x D_x \delta_x \right) - \delta_y \left( h_y D_y \delta_y \right) - \delta_z \left( h_z D_z \delta_z \right)
\]

at \( i, j, k, n \)

where \( D_x \) and \( D_y \) are the horizontal diffusion coefficients and \( k \) is the vertical mass exchange coefficient. For temperature we have:

\[
\delta_t (T) = - \delta_x (h_x u_x T) - \delta_y (h_y v_y T) - \delta_z (h_z T)
\]

\[
+ \delta_x \left( h_x D_x \delta_x \right) - \delta_y \left( h_y D_y \delta_y \right) + \delta_z \left( h_z \kappa' \delta_z \right)
\]

at \( i, j, k, n \)

where \( \kappa' \) is the vertical thermodiffusion coefficient.

For the turbulent energy density in the system we use:

\[
\delta_t (e) = - \delta_x (h_x u_x e_x) - \delta_y (h_y v_y e_y) - \delta_z (h_z e_z)
\]
Similar equations for velocity components $u$ and $v$ can be written for the top and bottom layers, except that the effects of ice and bottom friction are added. The vertical momentum exchange coefficients $E_x$, $E_y$, and constituent exchange coefficients $k$, $k'$, $E_e$ are derived from the subgrid-scale turbulent energy density of the general form

$$E = a \frac{z}{L}$$

where $a$ is the turbulent closure constant, and $L$ is the length scale and is taken as a function of the distance from the bottom and surface boundaries as follows:

$$L = k'z(1 - z/d)^{1/2}$$

where $k'$ is the von Karman constant, $z$ represents the vertical distance from the bottom to the point considered, and $d$ is the vertical distance from surface to bottom. In the computation, if the computed $L$ is greater than half of the layer thickness, the latter is used as $L$.

In the horizontal direction, the exchange coefficient is computed as a function of the local vorticity gradient and the local grid dimension:

$$A = \gamma |(\delta \omega_x + \delta \omega_y)| (\Delta \xi)^3$$

where $\omega$ is the vorticity, $\lambda$ is a coefficient and $\Delta \xi$ represents the grid length. From Eq. (8) we obtain the horizontal exchange coefficient

$$D_x = A \frac{\omega_x}{\omega_y} = \frac{A^x}{A^y}$$

In the interior, in addition to the convection, diffusion processes, the generation of subgrid-scale energy or the rate at which mean kinetic energy is converted into turbulence energy by means of the turbulent stress working against the mean velocity gradient is:

$$S_e = a_3 \frac{z}{L} \left[ (\delta u_x)^2 + (\delta v_y)^2 \right]$$

The rate of turbulent energy dissipation is determined from the concept that the dissipation rate depends on the transfer process from larger eddies to smaller eddies according to
The rate of turbulent energy dissipation from the increase in potential energy due to mixing the heavier layers with lighter layers is

\[ D_{pe} = \frac{-a_2 \delta N^2/ \rho_k}{v} \]  

(12)

where \( \rho_k \) = reference density
\( a_2, a_3 \) = coefficients

The mathematical model used to compute the response of the ice cover to driving forces is based upon consideration of the change in momentum in the horizontal plane by the wind stress at the upper surface, the stress at the ice-water interface, Coriolis force, internal ice stresses and the sea surface tilt.

The momentum equation for the ice can then be written in finite difference form:

\[ \frac{\delta (H^x u^t)}{\delta t} = - \delta_x \left( \frac{H^x u^x}{u^x} \right) - \delta_y \left( \frac{H^x v^x}{v^x} \right) + f H^x v^y - \frac{1}{\rho^x} H^x \delta_x p \]

\[ + \frac{1}{\rho^x} \left[ C^x \rho_a W_a^2 \sin \psi - \left( E_3 \frac{u}{u^t} \right)_{k=3/2} + \delta_x \left( H^x A^x \delta_x u \right)_\gamma + \delta_y \left( H^x A^x \delta_y u \right)_\gamma \right] \]

at \( i+\xi, j, l, n \)

\[ \frac{\delta (H^x v^t)}{\delta t} = - \delta_x \left( \frac{H^x v^x}{v^x} \right) - \delta_y \left( \frac{H^x v^y}{v^y} \right) - f H^y v^x - \frac{1}{\rho^y} H^y \delta_x p \]

\[ + \frac{1}{\rho^y} \left[ C^y \rho_a W_a^2 \cos \psi - \left( E_3 \frac{v}{v^t} \right)_{k=3/2} + \delta_x \left( H^y A^y \delta_x v \right)_\gamma + \delta_y \left( H^y A^y \delta_y v \right)_\gamma \right] \]

at \( i, j+\xi, l, n \)

where \( H = \) local ice thickness
\( u = \) ice velocity in \( x \) direction
\( v = \) ice velocity in \( y \) direction
\( C^* = \) wind stress coefficient (a value of 0.0025 is used)
\( \rho_a = \) density of air
\( \rho_I = \) density of ice
\( W_a = \) wind speed
\( \psi = \) wind angle from the \( y \) coordinate
\( f = \) Coriolis force term

In these momentum equations we have on the left side of the equal sign the change in momentum. The first two terms on the right are the advection terms, the
third term is the Coriolis force term, the fourth term the pressure term, and the fifth the wind stress term. The sixth term represents the momentum transfer to the flowing water underneath the ice, with $E_x$, $E_y$ representing the exchange coefficients derived from the SGS energy density locally, and the last two terms give a rough approximation of shear.

With the last two terms we are able to couple part of the ice field to land by use of very high horizontal momentum exchange terms or represent a certain area with unbroken thick ice coverage.

Conform the bottom stress of a fluid flow model, and the momentum exchange coefficient at the bottom of the ice can be expressed:

\[
E_x = \rho g \left( \frac{h_z}{2} \right)^2 \left[ (\delta_z u)^2 + (\delta_z \theta)_{ij} \right] / (C_s)^2
\]

\[\text{at } i, j, 1, n\]

\[
E_y = \rho g \left( \frac{h_z}{2} \right)^2 \left[ (\delta_z u)_{ij}^2 + (\delta_z v)^2 \right] / (C_s)^2
\]

\[\text{at } i, j+1, 1, n\]

where the density ($\rho$) and the thickness of the ice layer are computed locally by the parametric relationship presented later.

The Chezy coefficient ($C_s$) between the ice water interlayers is used not only for the momentum transfer computation, but also for the computation of turbulence energy generation with respect to the transport of subgridscale energy in the surface layer of water. In the computation a value of 420 cm$^2$/sec is used, which is equivalent to a shear stress coefficient of 0.0055. This value was estimated from several velocity profiles measured in the field.

In the presence of ice, the local top layer thickness for the water computation is adjusted according to the ice displacement, which is also a function of local ice thickness and the ice water density differences.

If ice is present at the model's open boundaries, then the nonlinear advection and diffusion terms are neglected in the momentum equation near the boundary in the same manner as is done in the flow computations.

Before a simulation is made, we have to estimate the initial ice coverage and ice thickness. Since there existed only very limited information about the ice thickness during the winter period, an initial ice thickness model was designed to estimate the ice thickness at the start of a simulation. Critical information to determine the ice thickness is the average value of the total degree-days (below zero) at the starting time of the simulation and the spatial salinity distribution at the beginning of the winter season.

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To initiate the ice simulation at each grid location, local water salinity is used to estimate the freezing point. This value is then stored for later use. The freezing point of sea water for various salinities can be estimated by

\[ T_f = -0.003 - 0.527S_w - 0.00004S_w^2 \]  

(17)

where \( S_w \) is the local sea water salinity.

In the modeled area, from weather statistics, an averaged value of total degree-days (below zero) can be obtained that corresponds to the starting time of the simulation. From this information, and the freezing point of sea water, local total degree-days below sea water freezing point can be computed. Once the freezing point has been reached, the salt is being rejected from the ice, thus it is no longer a function of local sea water, but of the ambient freezing temperature. The relationship between the salt content and the ice temperature can be expressed as

\[ S_i = 2.3 - 0.1883T_a \]  

(18)

\( T_a \) represents the ice temperature, which may be assumed to be the same as the ambient air temperature.

From the salinity of ice, the density of young ice is approximately

\[ \rho_i \approx 0.918 + 0.0008S_i \]  

(19)

To estimate the local ice thickness, the local latent heat of fusion of ice, \( \lambda_i \), can be computed also from the salinity of ice mentioned earlier:

\[ \lambda_i = 80.0 = 4.267S_i \]  

(20)

The initial local ice thickness at each grid location can be computed (in cgs units) according to

\[ H_1 = \left[ \frac{2 \Delta t}{\lambda_i \rho_i} (D - T_f) \times 24 \times 3600 \right]^{\frac{1}{2}} \]  

(21)

where \( \lambda \) denotes the coefficient of thermoconductivity, which is approximately 0.0055, and \( D \) represents the total degree-days below the freezing point locally.

In the subsequent computation, the amount of salt rejection or formation is computed as the source or sink terms in the salt balance equation for the top layer water simulation.

\[ S = \left( H_{1}^{t+\Delta t} - H_{1}^{t-\Delta t} \right) \cdot \frac{\rho_i \Delta t S_i^{t} \times 1000}{2 \Delta t \rho_w H_w^{t}} \]  

(22)
where \( \rho_w \) and \( H_w \) represent the density and surface layer thickness of water.

To illustrate the ice's response to wind and tidal forces, we present certain computational results using the adjusted model. Figures 1 and 2 give the bathymetric distribution of Norton Sound and the vertical layout of the model through a cross-section. Figures 3 and 4 show the distribution of ice thickness at the end of February. Areas in the eastern and southern shallow portions of the sound are covered with shorefast ice. The movement of ice under the combined effects of tidal current and a 10-knot wind from NNE is shown in Fig. 4. Negligible movement occurs in the shorefast ice zone. In the meantime, the water 1.5 meters beneath the ice (Fig. 5) shows the combined effect of tidal forces and wind stress transmitted through the ice.

To study the inertia component of the ice's movement, the wind stress is exerted only in the first 12 hours of the day simulated. The 24-hour displacement of ice at selected locations is presented in Fig. 6. The trajectory of ice floes under the northeasterly wind interacts with the underlying water (Fig. 7), and with tidal excursion produces a complicated transport pattern. When the 24-hour net displacement is plotted, the resultant movement (Fig. 8) indicates that there is no definite relationship between wind and ice movement in both direction and magnitude. They vary according to factors such as local depth, local tidal regime, vertical turbulent shear coupling (induced by wind, ice, tide and bottom stresses), and horizontal boundary effects.

**Acknowledgment**

This study was supported by the Bureau of Land Management through interagency agreement with the National Oceanic and Atmospheric Administration (NOAA), under which a multi-year program responding to needs of petroleum development of the Alaskan Continental Shelf is managed by the Outer Continental Shelf Environmental Assessment Program (OCSEAP) Office.
A risk assessment of offshore structures in northern waters is essential because of the particularly severe environmental conditions and the size of the installations.

Statistics of fatal accidents in Norwegian oil activities are summarized and show that structural failures in floating structures and helicopters dominate.

Statistics for structure-induced accidents in world-wide operation are presented with reference to a compilation from Lloyds' list by Det norske Veritas. An important conclusion is that the accident rate for mobile platforms is much greater than for fixed platforms.

Systematic risk analyses are necessary as a basis for a reliable risk assessment. Methods of risk analyses and the application of the results are outlined.

In the design against catastrophes, the notions of accidental loads and barriers are essential.

Finally, safety management and the role of government control are discussed, and some conclusions concerning safety and economy are quoted.
RISKS IN NORTHERN OFFSHORE ACTIVITIES - A MAJOR ISSUE

There are many reasons why particular attention should be paid to risks in offshore activities in northern waters.

The hazards are numerous:

- the environment is severe, especially with relation to wind, waves, ice and low temperatures
- structures in fixed position are often large and untraditional because of great water depths and hard weather conditions
- large and untraditional structures imply severe man-made hazards like fires, explosions and collisions
- mobile structures, although more conventional, are used in rough weather and for untraditional purposes such as production
- quantified observations of the environmental conditions, e.g. wave heights and shapes as well as spectral distributions, are incomplete
- the transfer from environmental parameters to environmental loads, e.g. wave and ice forces, is not very well understood
- long time deterioration under the environmental conditions in question is uncertain
- underwater inspection is difficult, particularly under ice

The consequences may be catastrophic:

- the structures are large
- a large number of people stay day and night on the installations, and the rough climate coupled with the long distances would make evacuation and rescue operations difficult
- low water temperatures allow only a short time for rescuing people in the water
- low temperatures delay the natural destruction of oil spills

The acceptance of risk is delicate:

- society is less inclined to accept risks in new activities, than the traditional risks that we are accustomed to
- we are much more scared by an accident with large-scale consequences, e.g. a multiple-death accident, than by a number of single-death accidents with the same total number of fatalities
- modern mass media support these attitudes by concentrating on large-scale dramatic accidents and more or less ignoring accidents which occur everyday e.g. on the roads
ACCIDENT EXPERIENCE IN NORWEGIAN OFFSHORE ACTIVITIES

In 1978 the Norwegian government launched a major research programme for safety and contingency planning offshore, partly managed by the Norwegian Petroleum Directorate and partly by the Royal Norwegian Council for Scientific and Industrial Research (NTNF). Accident statistics mentioned below have mainly been obtained from the NTNF part, denoted the "Safety Offshore" programme [1]. Unfortunately, neither unified official statistics nor single definitions of the offshore activities exist. The statistical data presented in [1] have been collected from various sources. Activities from the drilling of exploration wells to the loading of oil or gas into a tanker or through a pipeline to an onshore terminal have been included, see Fig 1. The figure shows for example that whereas onshore safety training and onshore fabrication of structures and equipment have been excluded in these accident statistics, the transport of people from the onshore terminal to the offshore installations has been included. This choice has resulted in some debate concerning comparisons with accident statistics for industry onshore, where transport from home

Table 1  Fatal accidents in Norwegian offshore activities as per 1st March 1981

<table>
<thead>
<tr>
<th>ACCIDENT</th>
<th>YEAR</th>
<th>DEATHS</th>
<th>INITIATING FAULT IN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alexander L. Kielland</td>
<td>1980</td>
<td>123</td>
<td>Structure</td>
</tr>
<tr>
<td>Helicopter S61 Crash</td>
<td>1978</td>
<td>18</td>
<td>Structure</td>
</tr>
<tr>
<td>Helicopter S61 Crash</td>
<td>1977</td>
<td>12</td>
<td>Structure ( ? )</td>
</tr>
<tr>
<td>Deep Sea Driller Gounding</td>
<td>1974</td>
<td>6</td>
<td>Operation</td>
</tr>
<tr>
<td>Statfjord A Fire</td>
<td>1978</td>
<td>5</td>
<td>Operation</td>
</tr>
<tr>
<td>Helicopter S61 Ditching</td>
<td>1973</td>
<td>4</td>
<td>Structure</td>
</tr>
<tr>
<td>Alpha Rescue Capsule</td>
<td>1975</td>
<td>3</td>
<td>Structure</td>
</tr>
<tr>
<td>Diving Bell</td>
<td>1974</td>
<td>2</td>
<td>Equipment</td>
</tr>
<tr>
<td>Micoperi 26 Barge, suffocation</td>
<td>1977</td>
<td>2</td>
<td>Procedure</td>
</tr>
<tr>
<td>Single Death Accidents</td>
<td>67-81</td>
<td>32</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>207</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 1 Sketch defining boundaries of offshore accident statistics
to the working site is normally excluded. The argument for including offshore transportation is that it can be compared to transportation within the working place, for instance in mines and large constructions sites onshore. Nevertheless, no choice is definitely right or wrong. The most important requirement is that the statistics presented must be unambiguously defined.

The consequences of accidents relate to man, environment and economy. Here, the consequences have only been measured in terms of loss of lives. Based on these definitions of risk arena and measure of consequences, the accidents in the Norwegian sector up to 1st March 1981 are listed according to their severity in Table 1.

The limitations in the consequence measure should be borne in mind. Thus, the Bravo blowout in 1977, from an ecological and economic point of view a major accident, is not included in the table because no lives were lost.

The table demonstrates that the multi-death accidents dominate. The column to the right shows that structural failure was the initiating event in a majority of these cases. The 32 single death accidents are mainly working accidents. An analysis of 21 working accidents [2] shows that falling or a falling object is a major cause of such accidents.

Table 2 Comparison of fatality rates in Norwegian activities

<table>
<thead>
<tr>
<th>ACTIVITY</th>
<th>FATALITIES PER 1000 MAN-YEARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shipping</td>
<td>2.1</td>
</tr>
<tr>
<td>Mining</td>
<td>0.9 - 1.4</td>
</tr>
<tr>
<td>Construction</td>
<td>0.3</td>
</tr>
<tr>
<td>Industry onshore</td>
<td>0.15</td>
</tr>
<tr>
<td>Offshore before ALK</td>
<td>1.5 - 2.0</td>
</tr>
<tr>
<td>Offshore after ALK</td>
<td>3.5 - 4.7</td>
</tr>
<tr>
<td>Offshore as per March 1981</td>
<td>3.1 - 4.1</td>
</tr>
</tbody>
</table>

Risk levels should be compared on the basis of fatality rates, e.g., the number of deaths divided by the total working time. Because the personnel stay offshore day and night, the total working time has been measured in terms of man-years. This
Table 3  Fatalities in Norwegian offshore activities by location and activity

<table>
<thead>
<tr>
<th>Location</th>
<th>Activity</th>
<th>Fixed platform</th>
<th>Mobile platform</th>
<th>Supply ships</th>
<th>Crane vessels/ barges</th>
<th>Pipe-laying vessels</th>
<th>Helicopters</th>
<th>Other</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maintenance/ testing</td>
<td>-</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Construction</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>11</td>
</tr>
<tr>
<td>Drilling</td>
<td>-</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>Production process</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Hotel platforms</td>
<td>-</td>
<td>123</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>123</td>
</tr>
<tr>
<td>Diving</td>
<td>-</td>
<td>5</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td>Crane operations</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Anchor handling</td>
<td>-</td>
<td>-</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Transp to/from / between locations</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>34</td>
<td>-</td>
<td>-</td>
<td>36</td>
</tr>
<tr>
<td>Emergency evacuation</td>
<td>3</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>9</td>
</tr>
<tr>
<td>Others</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>-</td>
<td>3</td>
<td>-</td>
<td>9</td>
</tr>
<tr>
<td>TOTAL</td>
<td>18</td>
<td>139</td>
<td>4</td>
<td>5</td>
<td>3</td>
<td>34</td>
<td>4</td>
<td>-</td>
<td>207</td>
</tr>
</tbody>
</table>

The number can, however, only be specified between upper and lower limits obtained using different sources for man-hours. As a consequence, upper and lower limits are given also in Table 2, where fatality rates in Norwegian offshore activities are compared with other Norwegian industries. Numbers for onshore activities have been adjusted so as to exclude administrative personnel, who are not included in the statistics for shipping and offshore activities. The numbers "before ALK" and "after ALK" relate to the situation immediately before and after the Alexander L. Kielland accident.

The 207 fatalities are distributed between the locations and activities shown in Table 3. The table demonstrates that so far no deaths can be related to the production process, in spite of the vast amounts of energy that are involved. Unfortunately, lack of readily available data has prevented a distribution of fatality rates by locations and activities.
ACCIDENT EXPERIENCES WITH STRUCTURES IN WORLD-WIDE OPERATION

Data of structure-induced accidents for the period January 1970 to January 1981, as compiled from Lloyds' list by det norske Veritas, are used. While the data for accidents resulting in major structural damage are extensive, the statistics for smaller accidents are incomplete because of insufficient reporting. The statistics for mobile platforms are believed to be more complete than those for fixed structures. Furthermore, the quality of the reporting system for different geographical locations varies. For these reasons the statistics should be interpreted with caution.

Table 4 shows the distribution of accidents for all platforms and for mobile ones, according to severity of the structural damage and immediate cause. In Table 5 only accidents involving fatalities are categorized according to the above pattern. Table 6 shows the number of fatalities.

The statistics quoted have several implications. It is for instance observed that about 10-15% of all reported structural accidents involve fatalities. About 25% of all reported accidents resulting in severe or total structural loss are fatal.

There have clearly been more major structural accidents with mobile units than with fixed platforms. Because the number of fixed platforms is roughly five times greater than the number of mobile ones, the accident rate for mobile platforms is over five times that of fixed platforms. The number of lives lost and the fatality rate have also been greater for mobile rigs than for fixed platforms.

It should be observed that mobile units constitute a variety of structural types. The accident and fatality rates differ among these structures. The accident rate relating to severe or total structural losses is, for instance, about 2-4 times greater for jack-ups than for semi-submersibles. However, fatality rates do not differ significantly between these two types of platforms.

As far as the initiation of accidents is concerned, Table 4 indicates that weather - and, as it turns out to be in many cases, maloperation - are important factors for mobile units. Collisions, blow-outs, fire and inadequate structural strength also frequently result in accidents. Most accidents to fixed platforms are caused by blow-outs, fires, collisions and weather.

Tables 5 and 6 show that most fatal accidents pertaining to mobile platforms are caused by blow-outs, explosions, capsizing and inadequate strength. It is noted that two accidents - caused by capsizing and inadequate structure respectively -
Table 4 Number of accidents for platforms in world-wide operation during 70.01.01. - 80.12.31. according to initiating event and extent of structural damage

Source: Lloyds' list

<table>
<thead>
<tr>
<th>Initiating event</th>
<th>Total</th>
<th>Severe</th>
<th>Damage</th>
<th>Minor</th>
<th>No</th>
<th>SUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather</td>
<td>7 (3)</td>
<td>12 (10)</td>
<td>30 (22)</td>
<td>21 (17)</td>
<td>9 (8)</td>
<td>79 (60)</td>
</tr>
<tr>
<td>Collision</td>
<td>4 (2)</td>
<td>5 (2)</td>
<td>17 (11)</td>
<td>21 (18)</td>
<td>23 (12)</td>
<td>70 (45)</td>
</tr>
<tr>
<td>Blow - out</td>
<td>15 (5)</td>
<td>13 (7)</td>
<td>15 (9)</td>
<td>14 (7)</td>
<td>13 (6)</td>
<td>70 (34)</td>
</tr>
<tr>
<td>Leakage</td>
<td>-</td>
<td>2 (2)</td>
<td>3 (3)</td>
<td>-</td>
<td>3 (2)</td>
<td>8 (7)</td>
</tr>
<tr>
<td>Machine etc</td>
<td>1</td>
<td>2 (1)</td>
<td>5 (4)</td>
<td>5 (6)</td>
<td>-</td>
<td>13 (11)</td>
</tr>
<tr>
<td>Fire 1)</td>
<td>3 (1)</td>
<td>6 (2)</td>
<td>20 (12)</td>
<td>19 (12)</td>
<td>-</td>
<td>48 (27)</td>
</tr>
<tr>
<td>Explosion 1)</td>
<td>2 (0)</td>
<td>3 (2)</td>
<td>10 (4)</td>
<td>9 (6)</td>
<td>1 (0)</td>
<td>25 (12)</td>
</tr>
<tr>
<td>Out-of-pos</td>
<td>-</td>
<td>-</td>
<td>3 (2)</td>
<td>-</td>
<td>6 (4)</td>
<td>9 (6)</td>
</tr>
<tr>
<td>Foundering</td>
<td>4 (1)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4 (1)</td>
</tr>
<tr>
<td>Grounding</td>
<td>2 (1)</td>
<td>6 (6)</td>
<td>3 (2)</td>
<td>5 (2)</td>
<td>1 (1)</td>
<td>17 (12)</td>
</tr>
<tr>
<td>Capsizing</td>
<td>11 (11)</td>
<td>4 (4)</td>
<td>3 (1)</td>
<td>1 (1)</td>
<td>-</td>
<td>19 (17)</td>
</tr>
<tr>
<td>Structural strength 2)</td>
<td>1 (1)</td>
<td>6 (4)</td>
<td>20 (14)</td>
<td>25 (20)</td>
<td>2 (2)</td>
<td>54 (41)</td>
</tr>
<tr>
<td>Other</td>
<td>2 (0)</td>
<td>3 (0)</td>
<td>1 (0)</td>
<td>12 (8)</td>
<td>15 (10)</td>
<td>33 (18)</td>
</tr>
<tr>
<td>SUM</td>
<td>52 (25)</td>
<td>62 (40)</td>
<td>130 (84)</td>
<td>132 (97)</td>
<td>73 (45)</td>
<td>449 (291)</td>
</tr>
</tbody>
</table>

1) Fires and explosions occurring in connection with blow-outs do not belong to this category as the initiating event in this case is the blow-out

2) This category includes structural failures that are not apparently included by rough weather or accidental loads. Hence, accidents caused by a deficient structure belong to this category

accounted for 60% of the fatalities. Most fatal accidents in connection with fixed platforms have been induced by blow-outs and explosions. The accidents which have caused most fatalities are due to blow-outs, explosions and collisions.
### Table 5
Number of fatal accidents occurring in connection with platforms in world-wide operation during 70.01.01. - 80.12.31. according to initiating event and extent of structural damage

Source: Lloyds' list

<table>
<thead>
<tr>
<th>Initiating event</th>
<th>Total</th>
<th>Severe</th>
<th>Damage</th>
<th>Minor</th>
<th>No</th>
<th>SUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather</td>
<td>1 (1)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1 (1)</td>
</tr>
<tr>
<td>Collision</td>
<td>1 (1)</td>
<td>1 (1)</td>
<td>-</td>
<td>1 (1)</td>
<td>1 (0)</td>
<td>4 (3)</td>
</tr>
<tr>
<td>Blow-out</td>
<td>4 (1)</td>
<td>6 (4)</td>
<td>2 (2)</td>
<td>1 (0)</td>
<td>-</td>
<td>13 (7)</td>
</tr>
<tr>
<td>Leakage</td>
<td>-</td>
<td>1 (1)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1 (1)</td>
</tr>
<tr>
<td>Machine etc</td>
<td>-</td>
<td>-</td>
<td>1 (1)</td>
<td>-</td>
<td>-</td>
<td>1 (1)</td>
</tr>
<tr>
<td>Fire 1)</td>
<td>1 (0)</td>
<td>1 (0)</td>
<td>1 (0)</td>
<td>2 (0)</td>
<td>-</td>
<td>4 (0)</td>
</tr>
<tr>
<td>Explosion 1)</td>
<td>1 (0)</td>
<td>4 (2)</td>
<td>5 (2)</td>
<td>5 (4)</td>
<td>-</td>
<td>15 (8)</td>
</tr>
<tr>
<td>Out-of-pos</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Foundering</td>
<td>1 (1)</td>
<td>1 (0)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2 (1)</td>
</tr>
<tr>
<td>Grounding</td>
<td>-</td>
<td>1 (1)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1 (1)</td>
</tr>
<tr>
<td>Capsizing</td>
<td>4 (4)</td>
<td>3 (3)</td>
<td>1 (1)</td>
<td>-</td>
<td>-</td>
<td>8 (8)</td>
</tr>
<tr>
<td>Structural strength 2)</td>
<td>1 (1)</td>
<td>-</td>
<td>1 (0)</td>
<td>6 (4)</td>
<td>1 (1)</td>
<td>9 (6)</td>
</tr>
<tr>
<td>Other</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4 (2)</td>
<td>-</td>
<td>4 (2)</td>
</tr>
<tr>
<td>SUM</td>
<td>12 (8)</td>
<td>18 (13)</td>
<td>12 (6)</td>
<td>19 (11)</td>
<td>2 (1)</td>
<td>63 (39)</td>
</tr>
</tbody>
</table>

1) and 2) See Table 4

**RISK ANALYSIS OF MAJOR ACCIDENTS**

A risk assessment can be based entirely on experience, e.g., as expressed by Tables 1 - 6, or on system analysis, or on both.

Methods based on experience alone have limited applicability due to the following reasons:
Table 6  Number of lives lost in structural accidents for platforms in worldwide operation during 70.01.01 - 80.12.31 according to initiating event and extent of structural damage

Source: Lloyd's list

<table>
<thead>
<tr>
<th>Initiating event</th>
<th>Structural loss</th>
<th>SUM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total</td>
<td>Severe</td>
</tr>
<tr>
<td>Weather</td>
<td>13 (13)</td>
<td>-</td>
</tr>
<tr>
<td>Collision</td>
<td>1 (1)</td>
<td>8 (8)</td>
</tr>
<tr>
<td>Blow - out</td>
<td>12 (5)</td>
<td>35 (26)</td>
</tr>
<tr>
<td>Leakage</td>
<td>-</td>
<td>1 (1)</td>
</tr>
<tr>
<td>Machine etc</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Fire 1)</td>
<td>-</td>
<td>7 (0)</td>
</tr>
<tr>
<td>Explosion 1)</td>
<td>4 (0)</td>
<td>8 (2)</td>
</tr>
<tr>
<td>Out - of - pos</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Foundering</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Grounding</td>
<td>-</td>
<td>6 (6)</td>
</tr>
<tr>
<td>Capsizing</td>
<td>93 (93)</td>
<td>6 (6)</td>
</tr>
<tr>
<td>Structural strength 2)</td>
<td>123 (123)</td>
<td>3 (0)</td>
</tr>
<tr>
<td>Other</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SUM</td>
<td>246 (235)</td>
<td>71 (49)</td>
</tr>
</tbody>
</table>

1) and 2) See Table 4

- with only one or a few accidents of a certain type there will be considerable uncertainty associated with the estimated probability of recurrence for purely statistical reasons

- once an accident has occurred, changes will be introduced to the system and probably also to other similar systems, e.g. by changes in codes. These changes will almost certainly have impact on the probability or the consequence of future similar accidents
- some types of accident have not yet materialized and hence no statistics exist

It is for example evident that a single accident with widespread consequences such as in the case of the "Alexander L. Kielland", does not give an accurate expression of the risk level of mobile units. Even the direct statistics presented are subject to large uncertainties.

To obtain a better understanding of the risk level, analytical approaches must be used together with available experience. A rational risk analysis is based on the following observations:

- almost every major accident has originated from a small fault and gradually developed through a long sequence (or several parallel sequences) of steadily more serious events culminating in the final event

- each single event in the chain occurs at a rate which allows the use of statistics, such as reliability data for components

- it is often reasonably well known how a system will respond to a certain event

The "Alexander L. Kielland" accident is a typical example. The physical accident chain was initiated by a failure of a single bracing, which was followed by the failure of five other bracings connecting a column to the rest of the platform. The loss of the column, implying loss of buoyancy, resulted in heeling, which immediately made the ordinary and emergency generators stop. In addition loose equipment started to move. The moving objects, and increasing hydro-static and -dynamic loads may have caused additional structural damage. The heeling also increased mooring-line tension, so that eventually the cables broke. More important, the heeling of the platform submerged openings, which resulted in progressive flooding. This influx of water made the rig steadily sink and heel with the potential of causing further structural damage, stopping the machinery functioning and thus impairing the operability of life-saving equipment. Finally, the rig capsized. The influence of factors of human management on the cause of the accident and the consequences in terms of loss of lives is fully described in [3].

By combining knowledge of system build up with knowledge of failure rates for the system elements, it is possible to achieve an indication of the risks in the system.

Thus, there is a need for failure/accident and background data in the case of a systems analysis approach. Background data are particularly required to describe the
system and its environment and normalize the risks. To normalize the risks it is necessary to know the length of time people and equipment are exposed to the hazards and the quantities exposed (compare Table 2). Up to now, knowledge about human reliability has been particularly scarce and has contributed the greatest uncertainties to the risk analysis. Evidently, the results of such analyses may be uncertain, and the conclusions may be difficult to draw. Nevertheless, a systematic analysis based on the best data available is the only rational approach to this intricate problem.

The results from a risk analysis may be used in different ways with varying benefit:

- **as the basis for accepting a complete system.** This is probably the most controversial use of the results from risk analyses. The uncertainties involved in the calculations are significant. For a mass-produced technical system, such as electrical equipment, the frequency of a certain failure may be predicted fairly well. For complex systems, involving people, the uncertainties are far greater. However, even if the basic data are dubious, the risk analysis should be formulated in numerical terms. On the other hand, a public discussion of such numbers may often be a serious problem.

- **as the basis of improving a system.** It is normally considered that the determination of the relative importance of each separate risk element is more accurate than that of the complete system. Through an identification of the most important risk elements new barriers may be introduced into the system more efficiently.

- **as the basis of alertness in monitoring or surveying.** The risk analysis may reveal major potential accidents which may be caused by failure of a single or double barrier. In such cases intensified control of the barrier during operation may be implemented as a measure to reduce the risk.

- **as the basis of an adequate accident preparedness system.** Contingency equipment can be tailored to the expected risk situations and accident combating can be more efficient.

In the offshore industry there has been a steadily increasing use of risk analysis. A risk analysis of a Condeep platform made by Statoil A/S is perhaps the most comprehensive one done to date in Norwegian oil activities. However, Statoil indicates a factor of uncertainties of 20 to each side in the estimated number of fatalities per platform year.
Risk analysis is today required by the Norwegian Petroleum Directorate for the evaluation of new concepts.

The final risk level is frequently defined by the principal layout. Thus, the risk assessment should be carried out at a very early stage in the design.

DESIGN PHILOSOPHY

The aim of the structural design is, apart from satisfying the functional requirements, to minimize the risk of structural failure, and above all a failure with catastrophic consequences. Thus two approaches are evident [4]

- to reduce the probability of a hazardous event
- to reduce the consequences of a hazardous event

The initiating hazardous events can be:

- abnormal (low) strength
- abnormal loads (accidental loads)

Abnormal strength should be controlled at the design and fabrication stage, reducing the probability of hazard.

The probability of abnormal loads could also be reduced by a proper design of equipment and control during operation, in particular of equipment with potential of high energy release. However, risks related to abnormal loads are mainly reduced by a proper layout.

Low probability events with catastrophic potential should be identified through a searching process and analysed by system analysis.

Draft guidelines from the Norwegian Petroleum Directorate on safety assessment of fixed platforms [5] emphasize accidental loads and specify that design basic accidents should be defined in quantitative terms, and comprise for instance:

- fire and explosions
- collisions (supply boats, helicopters, external shipping)
- dropped objects and flying fragments
- rare earthquakes
- environmental loads not included in the design loads

The guidelines allow accidental loads with a probability of less than $10^{-4}$ per year
to be ignored. The corresponding number for design environmental loads is $10^{-2}$.

A leading design principle is that if an undesired event occurs, the ensuing chain of hazardous events shall be stopped by new barriers (fail-safe design).

For mobile structures a fail-safe design philosophy has already been applied in connection with the floatability and stability of the rig. The design strategy, however, could be extended to cover structural integrity and the possible interaction of different failure modes (strength, stability etc) that may cause progressive failure.

A safety measure combatting abnormal loads and abnormal strength as well - performance monitoring - is being used for instance for the registrations of rigid body motions of floating rigs, stresses in mooring lines, and displacements and selected critical stresses in fixed structures. However, so far such systems will give appropriate warnings only in special cases and cannot replace traditional inspections. Thus, easy inspection design is urgent.

The design of the Statfjord "B" platform to reduce the consequences of accidental loads has been discussed by Dier [6]. Figure 2 shows how the layout of the deck has been chosen to locate a safe haven on the left side, as far away as possible from the main hazard area on the right, and with buffer zones imbetween. In this case fire and explosion hazards have been considered. For other parts of the structure collision and other hazards may be of interest.

SAFETY MANAGEMENT AND GOVERNMENT CONTROL

In the major accidents in Norwegian offshore activities such as the Ekofisk blow-out and the capsizing of Alexander L. Kielland, there has been no indication that the incidents were caused by new and unknown risk factors in the technical systems. In both cases the wisdom of hindsight tells us that with proper management of the known technologies these accidents might have been avoided.

This observation is valid for most accidents, and thus proper safety management is essential, and here, the exertion of government control plays an important part.

The government control of mobile platforms is exercised in a similar way in most countries. The control is based on the certification (in UK) or classification (to a certain extent in Norway) made by third parties.

The government control of fixed installations is in UK and USA also exercised by a third party, whereas Norwegian law requires an internal control by each operator.
Fig. 2  Risk zoning on the Statfjord "B" platform (from [6])

under the supervision of the Norwegian Petroleum Directorate, in some cases assisted by consultants.

Both systems have strong and weak sides. In the Norwegian internal control system there is never any doubt that the responsibility for safety is placed with the operator, and that this applies to the integrated system of technology, operation and management. Also, lessons learned will often be applied much faster when only one company is involved. This being due to the long and time-consuming process to change the rules and regulations of the third party. On the other hand, this may also be one of the weak points of this system, as experience gained in one company is not as easily transferred to others as in a system with a third party. If the operators use a third party organization as a consultant in their internal control system, this problem is reduced.
The Norwegian system requires new types of experts, who are able to check that the companies have established proper internal control systems, and by spot-checks, that the systems function as intended. This is quite different from controlling the results of the work done.

In addition, it will still be necessary to perform random control of the content of the decisions made by those responsible for the internal control in the company. Availability of personnel with sufficient insight in the area of safety management may become a serious bottleneck in the implementation of the internal control system.

Use of an internal control system would ultimately require few specific government rules as to design of structure, equipment etc. Most requirements could be directed towards the function and less towards specification of special methods and design concepts.

From a theoretical point of view a lot can be gained through functional rules as they open the way for new and better solutions. However, they also present the possibility of making the same mistake over and over again. Functional rules therefore need to be accompanied by an outline of acceptable solutions in the form of guidance notes. Specificational rules tend to conserve existing technology, and mistakes may occur due to faulty rules or poor applicability.

The third party system emerged from the system used in international shipping and is still quite similar to that system. The system requires the operating company to have the most important parts of the hardware system checked by a third party. A safety assurance system is much more extensive, as human beings operating the equipment and maintaining it, introduce risks which are beyond the control of the third party. Conventional third party systems could be extended to cover procedures and personnel better, and to ensure the operational fitness of the hardware. In no case can the third party system allow the company to operate without a proper safety management system.

It has been stated by a representative from Phillips Petroleum Company Norway [7] that the best solution is an internal control system combined with the use of a third party type of organization as consultant. The role of an independent organization as consultant is, however, considerably different from that of a third party. Even if the rules of the Norwegian Petroleum Directorate must be complied with all the way through, it will be necessary with more specific acceptance criteria for the different pieces of equipment and different operations. In the
The role of a consultant an independent organization can recommend, but not require, that certain criteria are met. As third party they are in a position to make requirements.

The Burgoyne committee have, however, concluded by recommending third party control [8].

SAFETY AND ECONOMY

There has been an almost explosive increase in costs for the production plants in the North Sea. To investigate the reasons for this increase, the Norwegian government appointed a special committee (Styringsgruppen for kostnadsanalysen - norsk kontinentalsøkkel) by Royal Decree of 16th March 1979. Among other tasks this group should clarify the influence of Norwegian regulations and safety requirements on the increase of costs, particularly as it had been claimed that the increase was caused by extreme safety requirements.

The following sections regarding costs caused by safety requirements are quoted from the conclusions of the committee [9]:

- New safety specifications from government agencies have caused noticeable cost increases for some projects, but not to such an extent that they can be said to be decisive for the costs.

- Laws and regulations regarding work environment, workers' protection, protection of ocean environments etc. have also caused considerable costs without being decisive for the cost situation. Hook-up work is an exception. New requirements in laws and regulations, a stricter enforcement of older laws and requirements regarding work permits, working hours, location and standards for living quarters etc. have made this work more expensive by an estimated 15 per cent.

It should, however, also be borne in mind that increased safety is accompanied by increased reliability and regularity. In particular, a reduction of production down time counts heavily on the positive side in the cost/benefit analyses.
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To make sure a safety performance of ice breakers or structures in cold regions, usual deterministic way to select steels is no longer applicable, since environmental conditions such as ice-structure interaction forces, and temperature, and even mechanical properties of steels show considerable scatter. Therefore, those variables must be considered as statistical manners.

In this paper we describe a method based on reliability analysis to select steels which should be used for the ice breakers and structures in the cold sea area. The method was also applied to analyse the safety assessment of the structures and to design a structure possessing both of fail security and economy.

Distribution functions of fracture toughness value of steels and imperfections inherently existing in the structures were surveyed by means of fracture toughness tests and non-destructive investigations, respectively. Uncertainties involved in each processes were cleared through probabilistic manipulations.

Finally, the probability of failure was quantified for the structures under given conditions. Imperfections which have not been detected by the NDI grow under fluctuation of the stresses, then the probability of the failure increased when service period was prolonged.

Reliability level for steel selection and for structure designing criterion was, then, determined by reflecting on the economy due to non-destructive investigation details, inspection interval and so on.

INTRODUCTION

Many ferritic steels behave unfortunately in a brittle manner when cooled down to
low temperatures. Some grades of steels, of course, are available with large fracture
toughness values even in extremely low temperature use. But cost becomes substantially
higher due to the grade that is otherwise highly satisfactory for offshore structure
construction or ship building purposes in the polar sea area. Because current systems
grading steel are satisfactory for semi-polar region operations, and moreover they are
based on the experiences on the brittle fracture of ships, or on a deterministic
approach using transition temperature criterion[1]. In the bases, the extreme case of
the brittle fracture is merely considered under combined conditions of the lowest
temperature and the largest stress acting on the structural members. Therefore, there
are difficulties in applying the current systems to the conditions of the colder
regions without imposing a cost burden which may not be necessary. All structural
members may not be exposed to the lowest temperature, and all structural components
may not be equally lowered down to the extreme ambient temperature, because of warming
influences from heat sources within the structures. All structural members also may
not be suffered from the largest stresses due to wave and/or ice interaction forces,
because the stresses have certain distributions by their nature.

In welded structures, welded joints as structural components sometimes seem to be
located at the positions where stress or strain concentrates. If some defects exist
in the welded parts, they will give a great influence on the strength properties, and
moreover they will cause the whole of structure to fail. Detectability of the defects
has been improved, and detectable minimum size of the cracks has been also smaller due
to recent improvement in the technique of non-destructive investigation. However, we
may introduce uncertainties, if we suppose the structure being safe and sound after
having been repaired all the defects which had been detected by the NDI.

On the other hand, a concept of fail tolerance design has been developed in order
to avoid usage of higher grade steels nor usage of the complicated NDI technique but
also to construct secure structures even though some defects exist. Therefore, we must
accept fracture mechanical approach instead of transition temperature approach to
recognize the low temperature properties of steels.

The brittle fracture of the structural members in the polar regions, thus, should
be considered as a statistical manner under the combined influences of probability that
fracture toughness as capability exists at some domain and probability that fracture
toughness as demand exceeds the domain at low temperatures. Therefore, it can be ex­
pected that all of the structural steels used in the extremely low temperatures will
need not to be of a grade that remains ductile at the lowest polar ambient temperature.

Under these concepts, it could be ideally possible to construct a structure having
a uniform reliability for all the components against the brittle fracture or having a
reliability being enough higher than a required value for the polar region use.

The overall objective of this study is to enable to define safety requirement with respect to the low temperature property of steel and its use in the polar class ships and steel structures, and to provide a system designing structures unifying the economy and the safety including a method for effectively assessing the suitability of a given low temperature range.

RELIABILITY ASSESSMENT AGAINST BRITTLE FRACTURE

A flow diagram for the reliability assessment is given in Fig. 1. This is composed of six blocks which are enclosed by broken lines. Those details are described as follows.

(i) Estimation of statistical frequency function for fracture toughness value as capability; Scatter in the fracture toughness is mainly caused by its nature of the fracture phenomena and by the inequality of manufacturing and so on. Then the most important thing is how many specimens are needed to grasp the scatter, which of course depends on the level of probability to be discussed.

(ii) Estimation of statistical frequency function for crack size; The cracks in the structure show various shapes. The current fracture mechanics can not treat such complicated crack shapes, because the fracture mechanics is not almighty. Therefore, we must substitute the actual cracks for simple shape cracks. The simplification methods have been already proposed, i.e., overestimating the crack size better than the actual crack shape in order to take safer side [2],[3],[4].

(iii) Estimation of statistical frequency function for stresses at hot spot; The true stress spectrum at the hot spot will be obtained by the various approach suggested in literature, such as cycle by cycle analysis or root mean square analysis and so on.

(iv) Estimation of statistical frequency function for fracture toughness value as demand; Crack tip opening displacement \( \delta \), one of fracture toughness values, can be obtained by a product of stress and crack size when the stress acts on the cracks [3]. If both of them are distributed obeying certain statistical manner, the \( \delta \) can be obtained by Cartesian Product based on probability theory. Namely, the statistical frequency function of \( \delta_{ij} \) is obtained using \( \delta_{ij} \) value given by the product of the probability of \( a_i \), and \( \sigma_j \) which gives certain domain of \( \delta_{ij} \).

\[
\delta_{ij} = a_i \sigma_j / E
\]

where \( E \) is Young's modulus and \( a \) is a constant and taken to be 3.5 [4].

Fig. 2 shows a schematical drawing of this process.

(v) Probability of failure; The probability of failure is given under the condi-
Fig. 1 Reliability Assessment against Brittle Fracture
tion that event A and event B are simultaneously occurred. Namely, the simultaneous probability is that the probability of fracture toughness as demand exceeds a certain value \( x \) (event B), while the probability of fracture toughness as capability being between \( x \) and \( x + dx \) (event A). This calculation is given in equation (2), and shown in Fig. 3 schematically.

\[
P_f = P(B|A)P(A) = P(B)P(A) = \int_0^\infty \left(1 - \int_0^x g(x)dx\right) f(x)dx
\]  

(2)

where \( f(x) \) and \( g(x) \) are frequency function of fracture toughness as capability and demand, respectively.

\[P_f = 1 - \int_0^\infty g(x)dx - \int_0^\infty f(x)dx
\]

\[P_f = \int_0^\infty \left(1 - \int_0^x g(x)dx\right) f(x)dx
\]

where \( f(x) \) and \( g(x) \) are frequency function of fracture toughness as capability and demand, respectively.

Fig. 2 Fracture toughness as demand

Fig. 3 Probability of failure

(vi) Estimation of reliability value and judgement for safety assessment of structure; The reliability against brittle fracture is finally given in equation (3).

\[
R = 1 - P_f
\]

(3)

If the reliability is greater than a required value, the steel or structure will be safe under the given condition. Examples for this analysis are given later.

STEEL SELECTION SYSTEM OF SHIP STRUCTURAL STEEL FOR LOW TEMPERATURE USE

Mechanical properties of steels examined here are listed in Table I. Fracture toughness tests on base metal, weld metal and weld fusion boundary of the steels were carried out in conformity with the method described in British Standard BS 5762 [5]. The critical crack tip opening displacement \( \delta_c \) versus temperature curves were preliminarily determined. Referring this relationship, test temperatures were selected. Then, the fracture toughness tests were carried out on thirty specimens at each test temperature. In order to survey the scatter in \( \delta_c \) values, they were plotted on Normal, Log-normal and Weibull probability chart. For example, the \( \delta_c \) plots on the Log-normal chart is given in Fig. 4. The best fit probability function was determined by means of Pearson's \( \chi^2 \) method. As the result, the distribution of the \( \delta_c \) seems to obey both Log-normal and Weibull distribution.

According to equation (1), the distribution of fracture toughness as demand was calculated using the distribution of defect sizes and the distribution of stress ampli-
tudes. In equation (1), the value of $a$ is taken to be equal to 3.5 4. The best fit probability function was again determined by means of Pearson's $x^2$ method. The $\delta$ values as demand seem to obey the Log-normal distribution. The distribution of defects (crack like-flaw) was investigated by x-ray technique on a certain ship. And the distribution of stress amplitudes on the ship due to wave was measured by means of strain gauges which were attached on the deck plates.

Then, the probability of brittle fracture under the given condition was calculated by manipulating equation (2), and the reliability against brittle fracture was finally obtained by equation (3). Those results were plotted in Fig. 5 as a function of temperature.

![Fig. 5](image-url) Relationship between reliability and temperature for steels.

![Fig. 4](image-url) $\delta_c$ plots on log-normal probability chart

![Fig. 6](image-url) Charpy absorbed energy vs temperature

**Table I** Mechanical properties of steels

<table>
<thead>
<tr>
<th>Steel</th>
<th>Thick. (mm)</th>
<th>Y.S. (kg/mm$^2$)</th>
<th>U.T.S. (kg/mm$^2$)</th>
<th>Elong. (%)</th>
<th>vEo (kgm)</th>
<th>vTrs (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>24.0</td>
<td>27.9</td>
<td>48.5</td>
<td>38.1</td>
<td>8.2</td>
<td>-7.0</td>
</tr>
<tr>
<td>B</td>
<td>25.4</td>
<td>33.0</td>
<td>50.6</td>
<td>35.6</td>
<td>11.2</td>
<td>-11.8</td>
</tr>
<tr>
<td>C</td>
<td>40.0</td>
<td>32.5</td>
<td>50.0</td>
<td>38.9</td>
<td>23.0</td>
<td>-36.0</td>
</tr>
<tr>
<td>D</td>
<td>45.0</td>
<td>38.0</td>
<td>57.1</td>
<td>39.7</td>
<td>12.4</td>
<td>-16.0</td>
</tr>
</tbody>
</table>
According to Det Norske Veritas [6], the weld metal can be used safely at 0°C, if Charpy absorbed energy is greater than 4.8 kgm when the Charpy impact tests are conducted at -20°C. As can be seen in Fig.6, where Charpy absorbed energy in weld metal of steel C was plotted, the absorbed energy shows almost 4.8 kgm at -20°C. And the reliability in the weld metal shows 99.912% at -20°C (see Fig. 4). Therefore, this reliability could be a requisition.

As the conclusion, for example, the base metal of steel D can be used safely down to -125°C, and the welded fusion boundary of steel D can be safe for -70°C use.

ESTIMATION OF RESIDUAL CRACK DISTRIBUTION AT WELDED PORTION AND RELIABILITY OF WELDED STRUCTURES AGAINST BRITTLE FRACTURE

There are many uncertainties which are involved for crack size measurement. Uncertainty due to accuracy, detectability, and detectable minimum crack size of non-destructive investigation was surveyed.

Surface cracks at the welded portion in steel D were detected by Magnetic Particle Investigation. The length of the crack was measured on the surface and the depth was measured by grinding off them carefully.

There are great discrepancy between actual crack length and measured crack length by the magnetic particle investigation [7]. Referring Fig. 7, the real number of the cracks whose sizes exist actually in a certain crack length interval can be calculated by following equation.

\[ N_i = \sum_{j=1}^{n} N_j \int_{x_i}^{x_{i+1}} f_j(x) \, dx \]  

(4)

where \( N_i \) is corrected number of cracks within a crack length interval, \( N_j \) is number of measured cracks in jth interval, and \( f_j(x) \) is a frequency function of crack length detected by NDI within the jth interval.

Detectability of the non-destructive investigation must be taken into consideration. The detectability decreases with decreasing crack size. Therefore, the number of cracks which are missed by the NDI increases with decrease in crack length. The number of actually existing cracks can be obtained by equation (5), and the number of residual cracks can be also estimated by equation (6).

\[ \text{Detectability} = \frac{N_i}{N_{Ai}} = 1 - \exp \left[ -0.8(2a-0.5) \right] \]  

(5)

\[ N_{Ri} = N_{Ai} + N_i \]  

(6)

where \( N_i \) is number of cracks in the ith interval according to the measurement, \( N_{Ai} \) is
number of cracks actually existing in the ith interval, and \( N_Ri \) is number of cracks existing in the same interval. The frequency functions for the each cracks in the weld metal and in the weld fusion boundary were determined by the Pearson's \( \chi^2 \) method. The results were plotted in Fig. 8, where \( f_{MPI} \), \( f_A \) and \( f_R \) are the frequency function for measured, actual and residual cracks, respectively.

![Fig. 7 Correction method to get actual number of cracks](image)

![Fig. 8 Frequency function for each cracks in weld metal and in weld fusion boundary](image)

Uncertainty due to selection of the investigating area is zero, since whole welded portion was investigated.

The probability of brittle fracture at the instance of construction was calculated according to equation (2), and the results are given in Table II. In Table II, the cracks distributing such as \( f_{MPI} \), \( f_A \) and \( f_R \) are supposed to be suffered from constant stress level of 25 kg/mm\(^2\). For the cracks in weld metal, residual stress due to welding was superimposed to the constant stress, since the cracks exist normal to the welding line.

<table>
<thead>
<tr>
<th>Steel</th>
<th>Temp. (°C)</th>
<th>( f_{MPI} ) (%)</th>
<th>( f_A ) (%)</th>
<th>( f_R ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld Fusion</td>
<td>-30</td>
<td>1.23x10(^{-6})</td>
<td>1.80x10(^{-5})</td>
<td>1.46x10(^{-5})</td>
</tr>
<tr>
<td>Boundary</td>
<td>-50</td>
<td>1.95x10(^{-8})</td>
<td>6.37x10(^{-8})</td>
<td>2.48x10(^{-3})</td>
</tr>
<tr>
<td>Weld Metal</td>
<td>0</td>
<td>1.29x10(^{-1})</td>
<td>1.54x10(^{-1})</td>
<td>6.97x10(^{-8})</td>
</tr>
<tr>
<td></td>
<td>-30</td>
<td>4.49</td>
<td>3.36</td>
<td>4.73x10(^{-1})</td>
</tr>
</tbody>
</table>

The brittle fracture probability for the welded structure could be reduced drastically, if the cracks detected by the NDI will be completely repaired. But we can not
ignore the influence of the residual cracks, when the structure would be used at sev­
erer conditions such as colder temperature.

Crack extension due to fluctuation of stress was estimated by Paris's formula[7].

\[
\frac{da}{dN} = C (\Delta K)^m
\]

(7)

where a is crack length, N is number of stress cycle, \(\Delta K\) is stress intensity factor range, and C and m are material constants. Stress function was assumed to have 5kg/mm\(^2\) of stress range. The result for the brittle fracture probability is given in Fig. 9. The probability of failure increases sharply if N exceeds \(10^6\).

Fig. 9 Relationship between probability of failure and stress cycles

Assuming that the crack distribution in the structure can not be changed, if the dimension of the structural components is changed, the probability of failure will be reduced by using thicker steel plate, since the working stress level becomes smaller under the same load level. Then total cost for three kinds of structures were calculated as a trial, assuming each of them was made by 30, 35 and 40mm thick plate. On the other hand, defining damage cost being a product of the probability of failure and sum of building cost and losing cost.

\[
\text{Damage cost} = (\text{Building cost} + \text{Losing cost}) \times P_f
\]

(8)

where losing cost = (a hundred thousand dollars a day) \(\times\) (Break down period).
The results are given in Fig. 10. Consequently, there exists a minimal in the total cost and the corresponding reliability is about 99.63%. This optimum reliability could be a design criterion.
The cracks less than the minimum detectable size $a_{cr}$ grow due to the fluctuation of stress. Assuming that the inspection was made after some stress cycles had been occurred, the $a_{cr}$ at $N=N_1$ grows to $a_{cr}^*$ at $N=N_2$, thus the probability of brittle fracture increases. This situation is given in Fig. 11, and the effect of inspection interval on the probability of failure is given in Fig. 12. The shorter the inspection interval, the more improvement in the $P_f$ is expected, though, the $P_f$ increases due to the growth of the cracks existing in the weld portion. This increases in $P_f$ depends strongly on the inspection interval, therefore, the maximum in $P_f$ during the whole life of the structure can be determined by the fixed inspection interval. And the optimum interval can be obtained in the same way as described previously.

![Fig. 11](image1.png) ![Fig. 12](image2.png)

**Fig. 11** Effect of minimum detectable crack size  
**Fig. 12** Relationship between probability of failure and inspection interval

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DYNAMIC ICE-STRUCTURE INTERACTION ANALYSIS FOR
NARROW VERTICAL STRUCTURES

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ABSTRACT
When an icefield crushes against a narrow structure, the ice force has a highly
dynamic nature that depends on the properties of both the ice and the structure. The
analysis of the dynamic ice-structure interaction is important not only in
order to estimate the dynamic magnification effects associated with the structure, but also in establishing a basis for fatigue and serviceability design. This paper
describes a method of computing the ice force and response of the structure on the
basis of information given for ice velocity and properties of ice and the structure.
The method is a step by step procedure using mode shape analysis involving two
basic phases. During the first phase the structure penetrates into the ice sheet
until a random loading rate dependent ice strength is reached. The ice sheet then
fails within an area with finite length. Both the penetration and the failed zone
are assumed to depend linearly on force. The ice forces and structural responses
have been computed for a test structure at the U.S. Army Cold Regions Research and
Engineering Laboratory in Hanover, New Hampshire and the results are found to be
consistent with those actually measured in laboratory experiments.
1. Introduction
When an icesheet crushes against a narrow vertical structure the ice force has an uneven, dynamic nature. Traditional massive hydraulic structures subjected to dynamic ice forces such as bridge piers and concrete caissons have been successfully designed using static equivalent force method. However, in order to meet the new engineering challenges in the most economical way, new light and slender structural solutions are needed. These kind of structures may be subjected to severe dynamic magnification effects under ice-structure interaction. The serviceability and fatigue criteria may also become important in the design process. Better understanding is needed not only on the magnitude of ice forces but also on the ice-structure interaction process in order to provide a rational design procedure for this class of structures.

The nature of the ice-structure interaction process depends on the properties of the structure and as well as on the properties and velocity of ice. Matlock et al [5] recognized this and used a simple one dimensional model to analyze the interaction process. Määtännäinen [6] added a nonlinear term to the equation of motion of the structure corresponding to the dependence of ice strength on loading rate. He also described a vibration isolation system to reduce the vibration level.

The idea for the present analysis was created during the laboratory indentation tests with natural ice plates performed by Eranti [2]. The observations and results indicated that the ice-structure interaction process can be divided into two phases. The structure first penetrates into ice in indentation phase during which the ice force increases as a function of penetration until a critical value is reached. The ice then fails suddenly within a finite length that depends on the critical force. The structure meets only minor resistance until a good new contact is developed.

2. Analysis
For the purpose of this analysis we define the critical ice force using Afanasev's [1] formula

\[ P_{cr} = mI \sigma_0 hD \]  

(1)

where \( m \) is the shape factor (\( m = 0.9 \) for cylindrical structure), \( h \) is the ice thickness and \( D \) is the diameter of the structure. The critical ice strength \( \sigma_0 \) is generally regarded as a random loading rate dependent variable and the indentation factor \( I \) is given by
\[ I = \sqrt{\frac{5h}{D} + 1} \quad h/D < 1 \]
\[ I = 4 - 1.55 D/h \quad h/D \geq 1 \]  

Assuming that the penetration of the structure into the ice during indentation phase depends linearly on the nominal stress we get

\[
\begin{align*}
[m]\ddot{x} + [c]\dot{x} + [k]x &= \{P(t)\} \\
x_c(t) &= \int_0^t v_{ice}(t)dt - \sum_j (i_j + f_j) - \frac{A}{mhD} P_c(t)
\end{align*}
\]

where the first equation is the general equation of motion of the structure. \([m]\) is the mass matrix including hydraulic effects using approximate added mass concept. \([c]\) is the damping matrix, \([k]\) the stiffness matrix and \(\{P(t)\}\) the external force vector. \(\{x\}\) is the modal point displacement vector and dot represents derivative with respect to time \(t\). The second equation states that the penetration of the structure into ice is proportional to stress. \(x_c\) is the lateral displacement of the contact point, \(v_{ice}\) is the velocity of ice, \(i_j\) and \(f_j\) are, respectively, the lengths of \(j\):th indentation and failed zones and \(A\) is a constant that gives the relation between nominal stress and penetration. Solving \(P_c(t)\) from the second equation and substituting it to the first we have

\[
[m]\ddot{x} + [c]\dot{x} + ([k] + [k_{cc}])x = \{P_o(t)\} + \{P_c(t)\}
\]

In vector \(\{P_o(t)\}\), the component with index \(c\) corresponds to the lateral degree of freedom at contact point and is equal to zero. In vector \(\{P_c(t)\}\), this component is the only nonzero component given by

\[
P_c(t) = \left[ \int_0^t v_{ice}(t)dt - \sum_j (i_j + f_j) \right] \frac{mhD}{A}
\]

Similarly, only the cc-component in the matrix \([k_{cc}]\) is nonzero with

\[
k_{cc} = \frac{mhD}{A}
\]

When the critical ice force \(P_{cr}\) is reached, the ice is assumed to fail very rapidly in front of the structure within an area with finite length given by
\[ f = \frac{P_{cr}}{m} B \]  

(7)

where \( B \) is a constant giving the relationship between nominal stress and failed zone length. The equation of motion is now given simply by

\[
[m] \ddot{x} + [c] \dot{x} + [k] x = \{P_0(t)\}
\]

(8)

as the ice resistance in the force vector is assumed to be zero.

By eliminating the rotational degrees of freedom using static condensation, equations (4) and (8) can be solved conventionally using mode shape analysis with numerical evaluation of the Duhamel integral involved. Two sets of eigenvalues and eigenvectors are needed and a switch from one to another takes place at each change of phase.

3. Laboratory Measurements

The ice-structure interaction process was analyzed using the proposed procedure for a test structure at U.S. Army Cold Regions Research Center, Hanover, New Hampshire. The test arrangement is shown in figure 1 and described more thoroughly by Maattanen [7]. The computer program with the parameter values used in the analysis is given in Eranti and Lee [3]. The computed support deformation reflecting the reaction is shown in figure 2, the computed ice force in figure 3 and the deflection of the structure at ice level in figure 4.

The corresponding laboratory measurements were pursued later for the situation analyzed. The measured support reaction is shown in figure 5 and the ice force as computed from the support signal (inertia effects included only partially, see ref. [7]) in figure 6. The computed results are found to be in relatively good agreement with those actually observed. Had we used a larger value for constant \( B \), the agreement would have been even better.
Figure 1. Test arrangement

Figure 2. Computed support deformation
Figure 3. Computed ice force

Figure 4. Deflection of the structure at ice level

Figure 5. Measured support reaction versus time
Figure 6. Ice force plot as computed from experimental data

4. Conclusions
A method for analyzing dynamic ice-structure interaction for narrow structures is presented and is compared with experimental findings. Close agreements between computed and observed results suggest that the proposed procedure provides a rational approach for relatively cold ice and narrow flexible vertical structures. Based on this procedure, an overall picture of the dynamic behavior of a structure can be constructed by letting the ice velocity increase slowly over the whole velocity range. For wider structures, the ice-structure interaction process may require the use of independent failure zones along the structural perimeter as suggested by Kry [4].

A number of approximations have been made in this development. For example, sudden ice failure is assumed although some ice resistance to the structural motion generally continues to be present in the failed zone. The assumed linear relationship between ice force and penetration length is also a simplification of the real situation. Furthermore, the use of the proposed method requires the proper selection of ice strength parameters and the constants $A$ and $B$ in equations
Ice strength seems to be lognormally distributed and somewhat loading rate dependent (ref [2] and [7]). A proper mean value for relatively cold good quality ice strength $\sigma_0$ corresponding to the critical loading rate might be of order 1.5 MN/m$^2$. In laboratory tests by Eranti [2] the variance of lognormal ice strength distribution was found to be of the order of $\sigma^2 = 0.06$. Constant $A$ was estimated to be about $0.03h^3/[MN]$ (larger than elastic estimate because of local crushing and plastic effects) and constant $B$ of the order of $0.06[m^3/MN]$. The ice temperature in these tests was close to $-10^\circ C$ and the aspect ratio $h/D$ in the range of $0.7 < h/D < 1.3$. 

Further development of the proposed method is continuing with additional analytical work and laboratory test data analysis. However, the analysis of actual field data for existing structures is even more important at this point and this will be undertaken as data of this kind become available from the Bay of Bothnia experiments.

5. Acknowledgements
The authors wish to thank the Foundation of Technical Promotion of Finland for the financial support of the initial stages of this study and Dr. Andrei Reinhorn and Dr. Dev Sodhi for valuable comments and help.

REFERENCES
RESPONSE OF OFFSHORE TOWERS TO NONSTATIONARY ICE FORCES

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M. Arockiasamy, Assoc. Prof. Engg. & App. Science, Memorial Univ. Canada

ABSTRACT
A dynamic nonstationary response analysis is presented for a three-dimensional framed offshore tower subjected to nonstationary random ice forces. Ice force records exhibit marked changes over periods of time with a random variation of the amplitude and frequency contents. These records are treated as piecewise-separable and the frequency-independent modulating function is estimated by mean square minimization. The evolutionary power spectral density is obtained as a product of the modulating function and the power spectral density of the stationary component. The response variance of the multi-degree-of-freedom system to the nonstationary excitation is determined.

INTRODUCTION
Nonstationary loading is characterised by a finite build-up time, a period of uniform intensity, and a period of decay. Blenkarn's ice force records [1] obtained in Cook Inlet, Alaska, exhibit marked changes over periods of time with a random variation of the amplitude and frequency. The segments of the record are piecewise-separable and the frequency-independent modulating function is determined for each individual segment using mean square minimization. The evolutionary power spectral density is obtained as a product of the modulating function and the power spectral density function of the stationary component. The response variance is determined for a three-dimensional framed offshore tower subjected to the nonstationary random ice forces.
REVIEW OF LITERATURE


PROCEDURE

The structure analyzed is a fixed offshore tower shown in Fig. 1. The members are assumed to be rigidly connected and the added water mass assumed equal to the mass of the water displaced. The Nyquist frequency criterion is used to determine the digitization interval for the ice force record of Ref. 1 divided into 20 unequal segments. Based on the Kolmogorov-Smirnov test, the nonstationary force segments are identified and a typical member of the set, Fig. 2, is chosen for the analysis. This segment is assumed to be piecewise-separable and divided into three subsegments of equal duration. In estimating the stationary component, $n(t)$, overlapped data points for consecutive record segments are used, i.e., the data points in the second half of the first subsegment are carried over and included in the computation of $n(t)$ for the second subsegment, and those in the second half of the second subsegment are included in the third subsegment. For each of the overlapped segments, the frequency-independent modulating function, $c(t)$, is estimated by applying the mean square minimization criterion as

$$c(t) = a_1 + a_2 (t-t_0)$$

(1)

where

$$a_1 = \frac{1}{n} \sum_{i=1}^{n} z_i^2(t)$$

and

$$a_2 = \frac{1}{n} \sum_{i=1}^{n} (z_i^2 - a_1 z_i) i/ \sum_{i=1}^{n} z_i^2$$
zi(t) = digitized ice-force data.

The covariance stationary component, n(t), of zero mean and unity variance is determined from

\[ z(t) = \sqrt{c(t)} \cdot n(t) \]  \hspace{1cm} (2)

The covariance function, \( R(t,s) \), and the evolutionary power spectral density of the separable process, \( f(t,w) \), are respectively

\[ R_z(t,s) = \sqrt{c(t)} \cdot c(s) \cdot R_n(t-s) \]  \hspace{1cm} (3)

and

\[ f(t,w) = c(t) \cdot f_n(w) \]  \hspace{1cm} (4)

where

\( R_n \) and \( f_n \) are the stationary covariance function and spectral density respectively of the component n(t).

From the well-known stationary relation for a single-degree-of-freedom system

\[ x(\omega) = |H(\omega)|^2 f(\omega) \]  \hspace{1cm} (5)

the corresponding nonstationary relation can be written as

\[ x(t,\omega) = |H(\omega)|^2 f(t,\omega) \]  \hspace{1cm} (6)

where

\[ x(\omega) = \text{stationary power spectral density of the response}, \]
\[ x(t,\omega) = \text{nonstationary power spectral density of the response}, \]
\[ f(t,\omega) = \text{nonstationary power spectral density of the excitation}, \]

and

\[ |H(\omega)|^2 = \frac{1}{(\omega_0^2 - \omega^2)^2 + 4\xi^2 \omega_0^2 \omega^2} \]

The evolutionary mean square response, \( \sigma_x^2(t) \), can be obtained from

\[ \sigma_x^2(t) = \int_0^\infty f(t,\omega) d\omega \]  \hspace{1cm} (7)

For a multi-degree-of-freedom system, the mean square response relation for stationary excitation [18],

\[ \sigma_x^2 = \sum_{r=1}^{n} \phi_r^2(x) \frac{\Gamma_r^2 f(\omega_r)}{8 \omega_r^3 \xi} \]  \hspace{1cm} (8)

can be extended to nonstationary excitation as

\[ \sigma_x^2(t) = \sum_{r=1}^{n} \phi_r^2(x) \frac{\Gamma_r^2 f(t,\omega_r)}{8 \omega_r^3 \xi} \]  \hspace{1cm} (9)

where

\[ \phi_r(x) = \text{normalized mode shape of the rth mode}, \]
\[ \Gamma_r, \omega_r, M_r, \text{ and } \xi = \text{rth participation factor, natural frequency, generalized mass, and a fraction of critical damping respectively.} \]
EXAMPLE

The structure analysed is a fixed offshore concept, Fig. 1, considered by Corotis and Martin [18] and Reddy, Cheema, Swamidas, and Haldar [19]. The analysis is restricted to the first three modes.

Frequencies:

\[
\{f\} = \begin{bmatrix} 1.02 \\ 1.75 \\ 1.99 \end{bmatrix} \text{ Hz}
\]

Damping:

\[ \xi = 0.06 \]

Generalized Mass Matrix:

\[
[M] = \begin{bmatrix} 3.612 & 16.315 \\ 16.315 & 1519.344 \end{bmatrix} \times 10^2 \text{ kg}
\]

Generalized Stiffness Matrix:

\[
[K] = \begin{bmatrix} 1.491 & 19.698 \\ 19.698 & 2371.273 \end{bmatrix} \times 10^4 \text{ N/m}
\]

Force Distribution Vector:

\[
\{q(y)\} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 1 \\ 0 \\ 0 \end{bmatrix}
\]

Participation Factors:

\[ \Gamma_1 = 0.0765, \Gamma_2 = -0.0747, \Gamma_3 = -0.0099 \times \frac{1}{\sqrt{\text{kg}}} \]

The evolutionary spectral densities of the ice forces obtained are shown in Fig 3; some typical values are listed in Table I. The evolutionary mean square responses of the structure at \( t = 2.792, 4.212, \) and \( 5.616 \) sec. are given in Table II.

DISCUSSION

Most of the physical phenomena are nonstationary in nature but there are very few analytical solutions of the structural response to nonstationary excitation. In the present paper an attempt has been made to determine the nonstationary response using minimization criteria. The study should be extended to obtain evolutionary response to nonstationary excitation using rigorous mathematical formulations.
ACKNOWLEDGEMENT

The authors are grateful to Dr. I.E. Rusted, Vice-President of Professional Schools and Prof. C.D. diCenzo, Dean of Engineering and Applied Science, Memorial University of Newfoundland, and Mr. O.S. Toope, Head of Applied Arts, College of Trades and Technology, St. John's, for their keen interest and encouragement. The support of this investigation by the National Sciences and Engineering Research Council of Canada, Operating Grant No. A-8119 and Strategic Grant No. G-0561 is gratefully acknowledged.

REFERENCES


<table>
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<th>Time (sec)</th>
<th>Frequency (rad)</th>
<th>Evolutionary Spectral Density (kip^2/Hz)</th>
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<tr>
<td>2.792</td>
<td>6.41</td>
<td>0.2159</td>
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<tr>
<td></td>
<td>11.00</td>
<td>0.1064</td>
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<tr>
<td></td>
<td>12.50</td>
<td>0.1064</td>
</tr>
<tr>
<td>4.212</td>
<td>6.41</td>
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</tr>
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<td></td>
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TABLE II

Evolutionary Mean Square Response

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<thead>
<tr>
<th>Time (sec)</th>
<th>Mean Square Response (m²)</th>
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<tbody>
<tr>
<td>2.792</td>
<td>0.0204 x 10⁻⁴</td>
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<tr>
<td>4.212</td>
<td>0.0414 x 10⁻⁴</td>
</tr>
<tr>
<td>5.616</td>
<td>0.2094 x 10⁻⁴</td>
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FIG. 1. THREE-DIMENSIONAL OFFSHORE TOWER AND LUMPED MASS MODEL

<table>
<thead>
<tr>
<th>MEMBER</th>
<th>CROSS-SECTIONAL AREA (m²)</th>
<th>MOMENT OF INERTIA (m⁴)</th>
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</thead>
<tbody>
<tr>
<td>A</td>
<td>0.0208</td>
<td>0.0071</td>
</tr>
<tr>
<td>B</td>
<td>0.1540</td>
<td>0.0179</td>
</tr>
<tr>
<td>C</td>
<td>0.0465</td>
<td>0.0094</td>
</tr>
<tr>
<td>D</td>
<td>0.1205</td>
<td>0.0085</td>
</tr>
<tr>
<td>E</td>
<td>0.0188</td>
<td>0.0005</td>
</tr>
<tr>
<td>F</td>
<td>0.0750</td>
<td>0.0008</td>
</tr>
<tr>
<td>G</td>
<td>0.0167</td>
<td>0.0004</td>
</tr>
</tbody>
</table>

MASS 11 (206.547 N·sec²/m)  
MASS 10 (88.964)  
MASS 9 (5.458)  
MASS 8 (137.650)  
MASS 7 (152.856)  
MASS 6 (168.151)  
MASS 5 (188.756)  
MASS 4 (256.298)  
MASS 3 (268.980)  
MASS 2 (324.510)  
MASS 1 (335.587)
FIG. 2. TYPICAL SEGMENT OF ICE FORCE RECORD - REF. 1.
FIG. 3. EVOLUTIONARY SPECTRAL DENSITIES OF THE ICE FORCE RECORD.
EXPERIENCES WITH VIBRATION ISOLATED LIGHTHOUSES

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Finland
Professor

Abstract
In order to avoid adverse ice-induced vibration effects in lighthouse superstructures a special vibration isolation system was developed. Two test lighthouses have been in operation at the Gulf of Bothnia, the first one since summer 1977 and the second one since fall 1978. Ice force and structural response measurement system is described. Measurement results are compared to design criteria and to predicted dynamic performance. Vibration isolation appears to be efficient in suppressing superstructure vibrations. The steel lighthouse with the vibration isolation system offers an economical solution for fixed offshore aids to navigation.

1. Introduction
Shallow waters along the coast of Finland with winding shipping passages require closely spaced reliable aids to navigation. Ice is present from three to seven months each winter. Moving ice with thicknesses up to 1 m create severe loadings against bottom-founded offshore structures. Single pile steel structures were introduced in 1973 to withstand ice forces and to be economical as well. At first there were failures due to initial underdimensioning and due to omitting the dynamic nature of ice forces, /1/. However after these problems were attacked and new design criteria presented no more structural failures have occurred. This far 93 fixed bottom-founded steel aids to navigation and two lighthouses with an vibration isolation have been constructed.

An objection against steel lighthouses was their susceptibility for ice-induced vibrations. To reduce ice forces and subsequently construction costs structures can be made narrow by utilizing the high strength of steel. With a conventional superstructure the increasing flexibility together with fluctuating ice forces in crushing
induce intolerably high vibration amplitudes for lantern equipment or for occupation. With other fixed aids to navigation - channel edge marks, leading lights, etc. - which only have a simple electrical lantern high accelerations are not a severe problem. Even for these a vibration isolation was developed to extend the life of the incandescent lamp.

To overcome the problem of ice-induced vibrations a simple vibration isolation system for the superstructure of a steel lighthouse was developed. It consists of flexible, highly damped beams that maintain the superstructure in upright position but allow the lateral displacements of foundation to any direction. Thus the rapid deflections of the foundation during ice crushing cannot contribute high accelerations to the superstructure. The first application of the vibration isolated lighthouse was at Kokkola in 1977 and the second at Kemi in 1978, both at the Gulf of Bothnia. Now during seven years of operation the vibration isolation system has proved its expected performance.

2. The Kokkola test lighthouses

Design requirements for the Kokkola test lighthouse were to withstand 1.0 m thick moving ice with never exceeding 0.5 g lateral acceleration in the superstructure. Design ice force was 4.4 MN in dynamic case and 9.0 MN in the case of a pressure ridge loading for the high water depth of 9.7 m. The foundation was a steel cylinder 4 m diameter at the bottom and narrowing conically to 0.64 m at the waterline. The 4 m wide 35 tons weighing superstructure with a helicopter landing platform was suspended above flexible beams that allowed ±165 mm lateral movement to any direction. The theory of self-excited ice-induced vibrations was used to trim the dynamic behaviour of the superstructure not to be excited by those natural modes that would induce significant vibration amplitudes to the superstructure, /2/. Internal damping in beams and in shock absorbers provided a safety over 5 against these vibrations.

Right from the beginning an telemetry system /3/ has been operational to monitor and record ice induced vibrations. First signals had to be recorded under manual control which yielded a meager amount of data because it was difficult to predict whether ice was moving at sea 200 km away. Also when ice moved its duration was short. Later an automatic triggering was furnished to turn on the tape recorded whenever signals exceeded a present value. The instrumentation includes two full strain gauge bridges at two levels in the foundation pile to measure the ice force and its action level, the relative displacement between the foundation and the superstructure, the superstructure acceleration, the wind velocity and direction.
Ice movements at this site have been less frequent than anticipated. Recorded dis- placements due to ice action at the vibration isolation part have been less than 20 mm which is of same order as displacements due to a 25 m/s wind.

Crushing tracks in ice fields have indicated that ice movements have occurred with ice at least up to 50 cm thick while telemetry system has not been on. To cover these events a fool proof plotter was installed also right from the beginning. This plotter is a pencil attached to the foundation and it plots on a piece of plywood that is attached to the superstructure. These records verify that there has never been any significant continuous ice induced vibrations. A typical plot is presented in fig. 1 which shows displacements of the order of wind allowance.

![Image 1](image1.png)  ![Image 2](image2.png)

Figure 1. Displacement plot.  Figure 2. Displacement plot.

The only significant occurrence is presented in fig. 2. There is a relative displacement of 89 mm from peak to peak or about 50 mm from the origin. Unfortunately there is no information about environmental data during this occurrence. It is most probable that the maximum displacement in fig. 2 is a result of initial ice movement after ice had adfrozen to the foundation pile. Due to better thermal conductivity of the steel pile there might have been also a thicker adfreeze collar at the ice bottom.
Comparisons to computer simulations for the response of a single sudden ice failure event suggest that a 3.3 MN ice force has been needed to cause the 89 mm pp. displacement. It is about 75% from the design dynamic ice force but only 32% from the displacement allowance. Thus results this far verify that there is a good margin of safety against single ice force fluctuations and that, as predicted, there has never been resonant vibrations.

3. The Kemi II lighthouse

The second application for the vibration isolated superstructure was on the foundation pile of Kemi II lighthouse, which had been unused since the winter 1973/74. This foundation pile was identical to that of Kemi I steel lighthouse which failed during the first winter of operation only 10 miles away. Also the original superstructure of Kemi II suffered damage during the same winter due to violent ice-induced vibrations and due to gas explosion that blasted off the top of the superstructure. In order to save the foundation the rest of the superstructure was removed in March 1974.

As after five winters in 1978 the foundation of Kemi II was still unharmed it was decided to be furnished by a new superstructure, fig. 3. It was anticipated that

Figure 3. Superstructure of Kemi II lighthouse with vibration isolation system.
the foundation might fail any winter but as the site was an important navigation fix
and as it would provide a stringent test site for a vibration isolated superstructure
the risk was taken. The foundation was designed for a 3.0 MN ice force and it was
about 4.4 times so compliant as the foundation of the first application at Kokkola.
The vibration isolation part was designed to withstand random 3.0 MN ice force fluc-
tuations. The lateral displacement between the foundation and superstructure was
limited to ±350 mm. No shock absorbers were used as was done at Kokkola and damping
was provided by the flexible beams only. According to the theory of self-excited
ice-induced vibrations the margin of safety against vibrations with superstructure
modes was about 3. At Kokkola it was more than 5.

Eye witnesses at the beginning of winter 1978/79 verified the expected behaviour of
the vibration isolation system while about 30 cm thick ice was crushing against the
foundation pile. The superstructure remained practically stationary regardless of
continuous ratcheting deflections of the foundation. The telemetry system was not
yet installed. It is noteworthy that with the original superstructure in 1973 already
10 cm thick ice induced violent vibrations.

At the end of February 1979 high winds made the 80 cm thick and partially rafted ice
field to move about 0.5 km. This ice action induced so large displacements both to
foundation and to the superstructure that the lateral movement limits in the vibration
isolation part were reached. As hit to limit stop occurred it caused high accelerations
in the superstructure. The lantern was damaged as it fell away with its platform
and the superstructure got also some scratches.

The analysis of this event indicated that the design for 3.0 MN random ice force
fluctuations was not adequate. The ultimate load capacity of the foundation is more
than 4.5 MN. A 80 cm thick ice, 2.5 MPa crushing strength and 1.1 m pile diameter
will result in a 4.7 MN crushing force. Already a single 4.4 MN sudden ice force
release during crushing would have caused the hit to lateral displacement limits.
Or two subsequent 3.2 MN ice force cycles would have caused the same thing. In
resonance with the first natural mode 2.2 MN ice force amplitude would have been
sufficient. For the next winter the lantern platform was re-installed and a new
electrical lantern installed. To match with the ultimate strength of the foundation
pilc the lateral displacement allowance for the superstructure was increased to
±550 mm during summer 1979.
Almost an identical ice movement that caused the damage in 1979 occurred again on 28 February 1980. This time the lateral free displacements between the foundation and superstructure was adequate and acceleration levels remained low in the superstructure. There is no measurement data to support this because the batteries of the telemetry system were drained out. However, there were loose objects left in the superstructure when the battery change started. The ice movement occurred before the new set of batteries were installed four days later. The loose objects remained stationary and did not fall down. This proves that acceleration level must have been less than 0.3 - 0.4 g.

After the telemetry system was equipped with an automatic on/off triggering unit for the tape recorder the amount of measurement data started to increase. It was learnt that ice movements are less frequent and of shorter duration than anticipated. This explains why it was difficult to catch an ice action event by telemetry monitoring. During a two month period in spring 1980 there was only 15 minutes of significant ice action.

In fig. 4 there are samples of relative movements in the vibration isolation part.
as well as corresponding superstructure accelerations. It appears that the frequency of ice crushing is inversely proportional to the displacement amplitude. The frequency decreases from 1 Hz to 0.38 Hz while the amplitude increases from 12 to 48 cm (peak to peak). Similar behaviour has been observed also in model tests. There do not occur any continuous vibrations with the lowest superstructure natural frequency which was calculated to be 0.42 Hz. Acceleration levels are low, less than 0.1 g, elsewhere but at the onset of crushing where accelerations of short duration are less than 0.3 g. Thus the data verifies the expected behaviour of the vibration isolation system.

The pitfall in the telemetery system is that it does not measure the ice thickness and velocity. Average information about ice thicknesses is available but not how local variations are related to the measurement data. Evidently significant variations in ice quality have been present which reflects in varying amplitudes and frequencies. In some cases, especially with low amplitudes, ice induced vibration patterns repeat themselves unchanged for up to minutes. This indicates that limit cycles have stabilized for those particular ice conditions.

Late in the spring, between 10 - 12 May 1980 the 80 cm thick landfast ice started to move driven by high north-eastern winds. This warm ice then did what the cold and strong winter ice was not able to accomplish, it overstressed the foundation pile which got a 5° permanent inclination. It is not known what were the ice conditions exactly, whether it was the landfast ice itself, a rafted or hummocked ice field or a pressure ridge. Anyhow an ice force to make the foundation pile to buckle in bending must have been more than 4.5 MN.

The telemetery system was operational during and after this event. There is data prior the failure of the foundation but the reel change interrupts and in the following reel there was no ice action any more. In fig. 5 there are samples of the overloading event and they present the highest recorded ice action response. In the first sample the amplitude is increasing up to 95 cm p.p., and frequency is decreasing to 0.45 Hz. Accelerations are less than 0.25 g elsewhere but at the onset of crushing which causes short duration accelerations up to 0.5 g. In the second sample the amplitude increases until hit to displacement limits occur, (110 cm p.p.). Hit occurs 4 or 5 times and it is visible in the sharpening of the upper part of displacement curve where now also the second superstructure mode becomes visible. With these high displacement amplitudes the crushing frequency has decreased to 0.34 Hz. Accelerations in the superstructure remain below 0.6 g and even when combined to cross axis acceleration the resulting maximum is less than 0.7 g. Loose objects in the superstructure,
e.g. batteries three times taller than wide, that did not fall also verify, that acceleration levels in the superstructure have not been able to be higher and that the number of high acceleration cycles has been low. Also the marks in the displacement limit ring were so slight that it also supports the count of only few hits. Thus it appears that the vibration isolation system performed excellently even during the overloading of the foundation and saved the superstructure from damage.

During summer 1980 the superstructure of Kemi II lighthouse was removed and re-installed to a new location at Välikivikko. The tilted foundation was equipped with a radar reflector. On 1 December about 10 cm thick ice was moving from the opposite direction of the tilt. It caused ice forces and bending moments high enough to tear the buckles open and to totally collapse the Kemi II foundation pile.

4. Välikivikko channel edge mark

The superstructure of Kemi II was re-installed on a channel edge mark foundation at Välikivikko which is also near Kemi. This foundation is of new design and it is 3.4 times as stiff as the foundation of Kemi II and more than two times so strong. The superstructure natural frequencies remain unchanged, 0.42 and 1.4 Hz but the first significant foundation natural frequency is now 7.5 Hz instead of 3.5 Hz in Kemi II. Safety against ice-induced vibrations is calculated to be 6 times better.

Telemetery measurements have indicated that ice with thickness less than 30 cm does not induce any detectable vibrations to the superstructure. This far the most
significant data is from 30 December 1980 while about 40 - 50 cm thick partially rafted and ridged ice was moving, fig. 6. The signal to noise ratio in this data is poor due to almost drained out batteries.

Figure 6. Displacement and acceleration samples on 30 December 1980. The maximum relative movement in the vibration isolation part was 24 cm from peak to peak. Acceleration levels in the superstructure were less than 0.1 g. Sharp acceleration peaks in the foundation are buried in the noise. By comparing to computer simulations an ice force of 2.3 MN is needed to cause 24 cm relative movement to the vibration isolation part. Ice crushing strength would then be 2.3 MPa for 50 cm thick ice or 3.1 MPa for 40 cm thick ice. A later analysis will be conducted to solve the ice force history from the relative displacement data by using transfer function and Fourier-transform methods. The extrapolation of measurement data for thicker ice indicates that at Välikivikko the design will be adequate for all anticipated ice forces and that superstructure accelerations will stay well below 0.4 g.

5. Resonant vibrations and damping

The safety factor against ice-induced self-excited vibration was calculated according to the steepest negative slope of ice crushing strength vs. loading rate curve. The used model indicated that 0.25 % modal damping would have been sufficient to prevent ice-induced resonant vibrations in the superstructure at Välikivikko. The first (and also the second) mode should have been dynamically stable because its damping
factor was about 5%. However, the data in fig. 6 clearly indicates resonant vibrations at 0.42 Hz. There were indications of resonant vibrations also in fig. 4 and 5 but there the stability margin was lower and the foundation pile of Kemi II was basically too flexible for such high ice forces which made conclusions premature there.

Välikivikko data proves that the efficient ice induced negative damping coefficient is higher than 5%. At Kokkola shock absorbers provided 15% damping for the superstructure modes and there has never been resonant vibrations. This suggests that for the Gulf of Bothnia ice the effective negative damping coefficient is between 5 and 15% for narrow structures. In scale-model tests in laboratory /4/ efficient ice induced negative damping coefficients were measured to be from -5 to -7%. Locally during the crushing negative damping may be overcritical yielding to aperiodic divergence which explains why the crushing phase is so fast.

As the positive structural damping has been less than ice-induced negative damping effect resonant vibrations have appeared. The superstructure supported on the vibration isolation system can be regarded as a structure on springs with foundation movement excitation. The dynamic amplification of movement will then occur. A safe requirement for design would be to apply the magnification factor at resonant frequency with no reduction due to counter-balancing positive and negative damping effects in ice induced vibrations. The superstructure at Välikivikko meets this requirement and regardless of resonance the acceleration level in the superstructure is low. However, also displacements could be reduced and evidently the resonance avoided by increasing internal damping. The developed vibration isolation system offers easy means to increase internal damping.

6. Conclusions

Vibration isolation system in slender fixed bottom-founded offshore structures efficiently prevents high accelerations from the foundation to be contributed to the superstructure during ice crushing. 15% internal damping has proved to be sufficient to prevent ice-induced resonant vibrations. With 5% internal damping resonant vibrations have occurred but vibration isolation has kept acceleration levels low in the superstructure. Even during an overloading event of foundation the vibration isolation system saved the superstructure from high accelerations and from damage. The vibration isolation system appears to be a cheap solution against ice-induced vibrations in economical fixed offshore steel aids to navigation.
References


DYNAMIC RESPONSE OF A JACKET PLATFORM
SUBJECTED TO ICE FLOE LOADS

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Abstract
The ice-induced vibration of a jacket platform located in the Po-hai Gulf is investigated based on both field measurements and computer analyses by Matlock's ice-structure interaction model. The implication of the results on the design parameters and application of the model is discussed.

1. Introduction
The sea ice problem in offshore engineering is getting more important for the designers as the exploration for petroleum has progressed into the Arctic and other frozen continental shelf regions in the world. The ice load being the dominant environmental force on the structure gives rise to apparent changes in some aspects of the design of offshore structures as compared to wave-dominated areas, for example, investigation requirements, structural analysis and design criteria.

In North China, the Po-hai Gulf has a potential oil and gas prospect, and the offshore engineering activities started in the late 1960's. Experience has made it evident that in this non-artic, frozen area, among the environmental loads ice is the most dangerous one, especially for those multi-piled (jacket) platforms which so far have been commonly used in the Po-hai Sea due to the soft sea bottom. Two platforms of this kind (one of them is a small flare jacket) have collapsed during the last 10 years due to the ice forces. This might be the southernmost area (37.5°-41.0°N, Fig.1) in the world having a sea ice problem. After these accidents, some of the design considerations of ice loads were revised and some experiments and field measurements were performed, but a more
thorough insight of the sea ice engineering problem and more in-situ and laboratory investigations are still needed.

As the loads of wind and waves, ice loads are forces of nature. However, since the material characteristics of ice itself are more sensitive to external environmental conditions, it seems even more difficult to give a realistic ice load estimation than to estimate wind and wave loads. A feasible mathematical model of ice-structure interaction is perhaps one of the most important matters in this connection, and also for structural response calculations. Two different models presented by Matlock [1] and Blenkarn-Määtätänen [2], [3], [4], [5], have been considered to be perhaps the most promising ones for describing the dynamic interaction between the ice and the pile structure. They are based on different concepts of physical effects, but each of them throws some light on the origin of the ice-induced vibration, of which our present knowledge is still too meagre to reach a final conclusion. An advisable way to carry out our research would be to work in parallel lines based on the different
models, and make more experiments and field measurements for comparison. This article is the first step in an attempt for a comparison study between the different models.

2. Basic parameters for the design ice loads

The basic design parameters of ice loads in Matlock's model are the ice floe velocity, \( V_{ice} \), the crushing length, \( p \), the maximum elastic deformation of the ice floe before its fracture, \( \delta_{max} \), and the effective ice pressure, \( p_c^* \). All these parameters are dominated by or closely related to the environmental conditions.

Generally, the drifting velocity of ice floe \( V_{ice} \) differs from the velocity of its driving source (current or wind). After the start of crushing, the floe will more or less be decelerated due to the contact force with the structure. The ice floe velocity is thus determined by the contact force as well as the current driving force acting on the floe, and can be approximately estimated by energy equilibrium considerations when certain environmental data are given. In some geographical areas, even for fairly strong floes, the extent of a floe may be limited, and then the ice floe velocity and the velocity of current may deviate significantly. In the Po-hai Sea, values around 0.25-0.5 of this velocity ratio have been observed. The ice floe velocity is an important parameter both in Matlock's and Blenkarn-Määtänen's models. From field observations a more evident tendency is that the ice velocity depends on the contact force, rather than the inverse. That is, a stronger, thicker ice floe always has a lower crushing speed than a weaker, thinner one even for almost the same current condition.

The crushing length \( p \), which together with the ice velocity \( V_{ice} \) determines the frequency of the ice 'impulse', depends mainly on the thickness and strength characteristics of the ice floe. In the Po-hai Sea, observations revealed that for strong floes, the broken fragments are always much greater than the displacement of the structure, and no flaking has been observed. The observations also showed that the bigger broken fragments always appeared in connection with the thicker, lower speed sheets and were accompanied by the lower frequencies of vibration. This tendency seems to be in accordance with the statement presented by Kivisild [6] that a thicker ice floe has a lower natural frequency than a

* The term 'effective ice pressure' (KN/m²) is used for considering certain field conditions on the basis of ice strength \( \sigma_c \) which is regarded as stress at failure in a standard test procedure.
thinner floe when the floe is regarded as a vibrating membrane supported on the surface of water.

The deformation behavior of the floe before fracture has a considerable influence upon the amplitude of the structural vibration by this model. By using elasticity theory with the elastic modulus of ice obtained from the usual uniaxial tests, the elastic deformation of ice resulting from the load by a circular pile in the middle of an ice field can be calculated to be around 1-2 mm for some ordinary situations [5]*. It is this relatively small ice deformation which causes a typical 'impulse' ice load by this interaction model. During the interaction some local elastic deformation of the structural member (column) at the point exerted by ice should be taken into account.

It has been realized that the failure mode of a sufficiently strong ice floe moving against a vertical pile is a repeated edge crushing. The ice crushing strength $\sigma_c$ (or the appropriate effective pressure $p_c$) determines the maximum value of the dynamic ice force on the structure. The determination of $\sigma_c$ (and $p_c$) values is a classical problem in the ice engineering practice, and more literature and information on this topic could be found than on the parameters discussed previously. In Matlock's model, the loading rate influence on the $\sigma_c$ value can also be considered if the experimental data are available. In the Po-hai Gulf, an effective ice pressure of 980 KN/m$^2$ was used for the jacket platform designs. This is fairly close to that which has been suggested for South Alaska [2]. This value was assigned and gained more confidence afterwards according to information from the following sources: (1) Laboratory tests of the ice samples taken from different locations of the Po-hai Gulf; (2) The comprehensive investigations of platforms which have experienced ice attack. Both the collapsed and the less impaired platforms are included. (3) Pertinent design codes.

For a certain location, it is believed that the critical values of such environmental factors as the strength, thickness and extent of the ice floe as well as the pertinent current velocity are determined by the regional geographical and climatic conditions. Hence, it seems reasonable to regard the environmental dependent parameters described above as the design parameters for a structure, and choose their design values from the field data and statistical procedures for a particular

* From the records of some full scale in-situ experiments [7], which seem to deviate considerably from this value, and considering the low value of the elastic limit of ice [8], further investigations of the deformation characteristics of the ice floe during crushing are still needed.
engineering project.

Considering the random nature of the ice properties and other environmental factors, it seems more rational to resort to the stochastic method for dynamic ice-structure interaction analysis. However, the limitation of application of this method for the present platform designs in the Po-hai Gulf is due to the extreme lack of ice force measurements.

3. Field measurements

Background information

The platform investigated is a 6-pile jacket used for supporting some equipment like boilers and combustion engines as well as for the accommodations (Fig. 2 a). This is one of the 3 adjacent jackets constituting an integrated oil production unit. The other two jackets are a production platform with a 60 pile foundation and a big deck area on one side, and a small gas flare jacket, 4 piles, on the other side. There are two 50 m simply supported bridges connecting the three jackets together (Fig. 2 b).
In the beginning of February, 1977, this unit was damaged by ice which covered a rather large area of these shallow waters (6-10M), drifting to and fro twice a day driven by currents and crushing against the platforms most of the time. The small flare jacket was pushed over by the floes about 2 days after the ice drifting arose. The 6-pile jacket was shaken violently, but the biggest one was quite stable and almost no vibration on it was felt.

Measurements

During the ice crushing recounted above, the dynamic response of the 6-pile platform was measured together with the collection of data on ice floes and currents [9]. Table 1 shows a set of measured values which were obtained simultaneously during a representative big crushing. Because of the sheltering by the adjacent structures, only 2 or 3 legs of the 6-pile platform were simultaneously in contact with the floes most of the time. There was no ice crushing or any other type of ice force exerting on the legs in the backrow.

Table 1

<table>
<thead>
<tr>
<th>Items</th>
<th>Ice thickness h (CM)</th>
<th>Ice velocity V_{ice} (CM/S)</th>
<th>Current velocity Vc (CM/S)</th>
<th>Amplitude at deck level x_m (CM)</th>
<th>Frequency of ice induced vibration f_c (HZ)</th>
<th>Ice crushing length p (CM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured values</td>
<td>60.0</td>
<td>10-15</td>
<td>40.0</td>
<td>0.41</td>
<td>2.1</td>
<td>5-8</td>
</tr>
<tr>
<td>Reference remarks</td>
<td>visual observation</td>
<td>x-direction</td>
<td>x-direction</td>
<td>visual observation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Remarks</td>
<td>ice force</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

Later that same year the dynamic behavior of the 6-pile platform was investigated. From field measurements the fundamental frequencies of the platform were obtained to be 2.1 Hz along the x-direction and 2.3 Hz along the y-direction.

4. Computer analysis

Structural modeling and soil contribution

The 6-pile platform is idealized as a 3-dimensional frame system shown in Fig. 3 a. The soil stiffness is simulated by a series of springs at each 'pile head' node.
at the mud line level (Fig. 3b). The mass distribution at the nodes is assigned from the data of the platform including the added mass effects for the submerged members. The damping is represented by introducing local dashpots and the damping coefficients are specified for the different nodes.

![Diagram](image)

Figure 3

The stiffness of the foundation soil has a great influence on the dynamic behavior and response of the structure. Table 2 shows the average values of parameters in the soil profile.

<table>
<thead>
<tr>
<th>Sea bottom No.</th>
<th>Layer</th>
<th>Soil type</th>
<th>Water content (%)</th>
<th>Bulk density (KN/m³)</th>
<th>Void ratio</th>
<th>Undrained shear strength (KN/m²)</th>
<th>Calculated G max (KN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0M</td>
<td>I</td>
<td>MUD</td>
<td>62.0</td>
<td>16.1</td>
<td>1.70</td>
<td>5.0</td>
<td>1500.0</td>
</tr>
<tr>
<td>-6.0M</td>
<td>II</td>
<td>MUDDY CLAY</td>
<td>47.6</td>
<td>16.9</td>
<td>1.34</td>
<td>24.3</td>
<td>7290.0</td>
</tr>
<tr>
<td>-9.0M</td>
<td>III</td>
<td>SANDY CLAY</td>
<td>25.5</td>
<td>19.5</td>
<td>0.74</td>
<td>85.0</td>
<td>83486.3</td>
</tr>
<tr>
<td>-14.0M</td>
<td>IV</td>
<td>CLAY</td>
<td>32.4</td>
<td>19.2</td>
<td>0.83</td>
<td>120.0</td>
<td>91620.7</td>
</tr>
<tr>
<td>Pile</td>
<td></td>
<td>and SANDY CLAY</td>
<td>32.4</td>
<td>19.2</td>
<td>0.83</td>
<td>120.0</td>
<td>91620.7</td>
</tr>
</tbody>
</table>
The following procedure was used to approximately estimate the dynamic lateral stiffness of the pile-soil system:

1. Estimation of dynamic shear modulus $G_{\text{max}}$. $G_{\text{max}}$ is a main parameter in the dynamic stress-strain relation of soils. It depends on a number of geotechnical variables [10]. For the III and IV layers shown in Table 2, the $G_{\text{max}}$ values are calculated by the Hardin and Drnevich formula as follows [11]:

$$G_{\text{max}} = 1230 \left( \frac{(2.973-e)^2}{1+e} \right) \sigma_0$$

where $e$ and $\sigma_0$ are void ratio and effective ambient stress of the soil layer, and both $G_{\text{max}}$ and $\sigma_0$ are in pounds per square inch. For the first two layers (mud and muddy clay), however, from the concept of a linear relationship between deformation modulus and effective ambient stress for a normally consolidated clay mentioned by Janbu [12], and the information provided by Seed and Idriss [13], the following approximate relation was used:

$$G_{\text{max}} = 300 S_u$$

where $S_u$ is the undrained shear strength of the soil. The $G_{\text{max}}$ value of each layer is thus calculated and listed in the side column of Table 2.

2. Construction of the force-deflection curve at the pile head level by a quasi-dynamic pile-soil interaction analysis. Considering the situation here that the inertia effect of the soil mass is not so great on the soil pile interaction, the program SPJ [14] is used to obtain the force-deflection curve for the top level of the pile, taking the dynamic shear moduli as input.

Finally, according to the realistic stress level in the pile and soil, the lateral stiffness coefficients of the foundation soil are assigned to each foundation node as follows:

<table>
<thead>
<tr>
<th>Translation stiffness</th>
<th>Rotation stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>(KN/m)</td>
<td>(KN.m/rad)</td>
</tr>
<tr>
<td>$1.84 \times 10^6$</td>
<td>$5.98 \times 10^3$</td>
</tr>
</tbody>
</table>

This set of coefficients has been used for the natural frequency analysis of the platform, and the results are quite in agreement with the measured values.
Computer program

The dynamic ice load response analysis is carried out by ICEFED, a modified version of the original finite element program FEDA [15]. Basically, the modifications consist in the introduction of a nonlinear load vector in the time-stepping integration scheme according to Matlock's model. The step-by-step integration is started by the initial conditions for the displacement and velocity vectors $\mathbf{r}_0$ and $\mathbf{r}_0'$. Then, for each time increment the dynamic equation of motion

$$M\ddot{\mathbf{r}} + C\dot{\mathbf{r}} + K\mathbf{r} = \mathbf{R}(t, \mathbf{r})$$

(3)

is solved where $\mathbf{R}(t, \mathbf{r})$ is the nonlinear load vector, and $M$, $C$ and $K$ are the mass, damping and stiffness matrices of the system, respectively.

Assuming the load vector to be known at time $t_K$, the solution for the response vector $\mathbf{r}(t_K)$ is provided by a chosen direct integration method within the Newmark-Wilson family implemented in the program [15].

In the present case a Newmark constant average acceleration method was found to yield the best results.

![Diagram](Figure 4)

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Figure 4

- Response at time $t_{k-1}$
- Compute approximate response at time $t_k$
- Evaluate load vector
- Solve equilibrium equations to get new response at time $t_k$
- Convergence?
  - NO
  - YES

$t_k = t_{k-1} + \Delta t$

---

Figure 5

- Ice force
- Relative displacement $\delta_{\max}$
- $\mathbf{Q}_{\max}$
- $\mathbf{r}_{\max}$

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As the load vector \( R(t_K, r(t_K)) \) is unknown, however, an iteration procedure is applied: A first approximation of the response is evaluated on basis of vectors \( r(t_{K-1}) \) and \( \dot{r}(t_{K-1}) \). The corresponding load vector is calculated and the equilibrium equation is solved to give a new response vector. This implies an updated load vector and so on until satisfactory convergence is achieved. This procedure is illustrated in Fig. 4.

Determination of the load vector is based on a decomposition of the response vector at nodes with ice load into longitudinal \( r_L(t_K) \) and transversal \( r_T(t_K) \) components as referred to the ice velocity \((V)\) direction. The possible deviation of \( r(t_K) \) from the \( V \) direction is due to structural coupling of responses. The relative displacements \( \| V \times t_K - r_L(t_K) \| \) and \( \| r_T(t_K) \| \) are then used to determine the ice loads from the relationship shown in Fig. 5.

**Results of analyses**

A typical result of analyses with the input parameters as given in Table 1 is shown in Fig. 6 a. In the calculation, \( \delta_{max} = 3 \, \text{MM} \) is adopted including the consideration of the local elastic deformation of the column. According to the literature [1], [4], the ice-induced vibration is always accompanied by high damping, so the damping ratio \( \xi \) is assigned to be 0.12. The biggest damping coefficients are distributed to the foundation nodes and ice loaded nodes.

As an alternative consideration, a value of \( \delta_{max} = 5 \, \text{MM} \) is employed in the previous analysis with a much higher damping ratio \( (\xi = 0.9) \). This result is illustrated in Fig. 6 b.

Fig. 6 c depicts the influence of the mass of the system on the dynamic response in the case of ice load frequency equal to the undamped resonant frequency of the system and with overcritical damping.

When \( \xi \leq 0.06 \) for the case of \( \delta_{max} = 3 \, \text{MM} \) and when \( \xi \leq 0.5 \) for \( \delta_{max} = 5 \, \text{MM} \), a resonant response was observed.

A simple plane frame structure with similar dynamic behavior and ice loads as those of the real structure has also been used for parameter studies. The results indicate that the critical magnified response occurred at a lower ice 'impulse' frequency than the resonant frequency of the system. From a Fourier series expansion of the ice load history, this can presumably be explained by a redistribution of the dominant amplitudes towards higher frequencies for small \( \delta_{max}/p \) ratios. The parameter studies also indicate that the behavior of the structure (mass, stiffness) has almost no influence on the frequency of the ice breakings.
a) $V_{\text{ice}} = 10.6\, \text{CM/s}$, $\rho = 5\, \text{CM}$, $\delta_{\text{max}} = 0.3\, \text{CM}$, $Q_{\text{max}} = 588.6\, \text{kN}$

b) $V_{\text{ice}} = 10.6\, \text{CM/s}$, $\rho = 5\, \text{CM}$, $\delta_{\text{max}} = 0.5\, \text{CM}$, $Q_{\text{max}} = 588.6\, \text{kN}$
\( \nu_{ice} = 10.6 \text{ CM/s}, \ p = 5 \text{ CM}, \ \delta_{max} = 0.5 \text{ CM}, \ Q_{max(45\degree)} = 588.6 \text{ kN} \)

**Diagram 1:** Displacement of top node 1 (13.5 m above sea bottom)

**Diagram 2:** Displacement of ice load node 5 (8.0 m above sea bottom)

**Diagram 3:** Ice force at node 5 (8.0 m above sea bottom)

1 - Original model
2 - Model with 1/5 mass

*Figure 6*
5. Discussion

1. From the comparison between the analytical results and the measurements, it is indicated that Matlock's model is promising for expressing the dynamic ice-structure interaction in the Po-hai environment.

2. Both from the analyses and the measurements it was shown that the dynamic displacement caused by ice in normal cases is smaller than the corresponding static one. This is mainly due to the inertia effect of the structure under a certain ice velocity, and the usually much higher stiffness of the ice floe than that of the structure. For this 'impulse' type of ice loads, the inertia (mass) of the structure will be the controlling factor of its dynamic behavior.

2. A possible maximum deformation and stress state of the structure, which are not calculated herein, would therefore occur when the ice floe is almost 'stopped' by the structure (low floe velocity). This may result in the first possible failure mode ('static' or 'quasi-static') of a structure loaded by ice floes. In this case, the structural inertia force could be neglected.

4. Even if it has not been confirmed by our measurements, according to the parameter studies amplified resonant response should be expected under a certain combination of ice floe velocity, spatial ice load distribution and damping properties of the system. This will cause another possible failure mode ('dynamic') of the structure. Since the crushing frequency of the ice floe covers a vast range according to the measurements (0.15 - 15 Hz, Määtänen [16]), it is important to evaluate carefully the design parameters - ice velocity and its matching floe thickness on the basis of investigations for a particular area. Which of the two failure modes ('static' and 'dynamic') will be the critical one for a structure being designed, depends on the environmental conditions as well as the dynamic features of the structure.

5. The frequency of the ice load 'impulse' is hardly influenced by the structure. Because of the usually high ice/structure stiffness ratio, the time of occurrence of the local ice failure is mainly governed by the velocity of ice floe rather than that of the structure. Another reason is the much bigger crushing length for the strong ice floes than the deformation amplitude of the structure. These facts imply that the frequency of the ice induced vibration is determined basically by the environmental conditions rather than the structure as observed by Peyton [17] from his measurements and experiments.

6. The connection between the inertia and damping of the system and their effects in an ice-induced vibration is still an interesting problem for the future research, and this may be one of the key points for a better understanding if this type of vibration is a forced or a self-excited vibration.
Acknowledgements

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THE ICE FORCE ACTING ON A CYLINDRICAL PILE

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Abstract

A laboratory ice force tests on cylindrical pile were conducted. The results show that the ice pressure reduces with increase in towing speed and that breaking with buckling mode is dominant in the slower towing speed range. The ice force reduces with increase in the diameter of pile model, but is larger than the empirical formula proposed by Saeki, et al.

Introduction

The northeast coast of Hokkaido island (Japan) is covered by ice-floes from Okhotsk Sea during January and March of each year. Sometimes, these ice floes have damaged fishing resources, especially seaweeds on the seabed near the coast of this area. And hence the demand for anti-icefloe facilities arose, such as ice floe barriers.

Japan also has international joint projects with U.S.S.R. (Sakhalin Project) and China (Poh-Hai Sea Project) to produce crude oil and LNG. Since the production sites of these projects are covered with ice floes during the winter, the offshore structures for these purposes must have enough strength for the ice loads.

Based on the above-mentioned background, we began study on the technology for evaluating the ice forces acting on the offshore structures.

The cylindrical or conical shape have been applied to the offshore structure in ice covered water. To reduce ice force the conical structures are considered to be better in the severe ice conditions, but in the regions where the ice conditions are not so severe, the cylindrical structure have advantages in that it is easy to design and manufacture. Cylindrical structures, therefore, are widely used in such areas. So the force acting on the cylindrical pile is the most fundamental and
important problem, in the design of this type of structure.

Many formulae for predicting ice force acting on the cylinder have been proposed \(^{(1-3)}\). But the designers have found large discrepancies between the estimated values in these proposed formulae. In order to tackle this problem, we have conducted laboratory ice model tests and compared the results with the large scale tests conducted at Lake Saroma in Hokkaido.

The purpose of the laboratory test was to obtain as correct results as possible.

**Testing facilities**

The ice tank had a width of 3m, a length of 6m and depth of 1m, and was made of stiffened steel plate. The inner surface of the tank was insulated with polyurethane panel of 5cm thickness. The rise in water pressure due to the ice forming was relieved by an air pipe attached to the side wall of the tank. A pair of rails were constructed on both sides of the longitudinal walls, and a motor-driven towing carriage rode on the rails. The carriage drove on the rail by Abt system helped by a pair of racks which were attached beside the rails.

The controllable speed of the towing carriage was between 1 cm/sec and 30 cm/sec. The test model was attached to the towing carriage through a 3-component load cell.

Several thermocouples were located near the water surface to measure the temperature of air, middle part of ice, and water.

The ice tank was fabricated in a cold room having length of 16.2m, breadth of 12.6m and height of 5.2m. Two sets of fan units circulated the cold air in the cold room.

**Outline of the experiments**

0.6% saline water was used for the experiments. The salinity was controlled by measuring electric conductivity of the saline water. Wet seeding was applied to make the grain size of the ice small. After 20 to 22 hours of refrigeration, an ice sheet of 4-5cm thickness was formed. During the seeding and the refrigerating, the tank was covered with a perforated vinyl sheet on the top surface, in order to induce the vertical air flow on the water (or ice) surface. This equalized the temperature over the water (or ice) surface and helped to form an ice sheet of uniform thickness.

When the ice thickness became the planned value, the ref-machine was stopped, and ice temperature began to rise. After 2-3 hours of warming up, the temperature of ice sheet became higher and the compressive strength of ice became small enough to conduct ice force test by our towing carriage. The fluctuation of thickness
over the ice sheet was ±1.5mm. The ice force test was conducted when the temperature distribution through the depth of ice sheet became homogeneous, -0.7 ~ -0.8°C.

In order to evaluate the mechanical properties of the tested ice, compression test was conducted after each test.

Since it is very difficult to measure the compressive strength of such thin ice sheets, we tried a method as follows:

1. Made cubic specimens in the cold room (air temperature about -10°C). Their sizes were 5cm x 5cm x ice thickness.

2. Preserved the specimen in the ice tank for 3-4 hours in order to make the temperature of specimen to be same as that of ice sheet.

3. After the ice force test, picked up each specimen and tested them by a simple testing jig proposed by Timco(4). The loading direction was horizontal.

The cylindrical models had diameters of 5, 10, 20, 30cm, and were made by purning epoxy resin reinforced with a steel core on a lathe.

The indentation length of tests were about 10 times the model's diameter. Thus, 2-3 experiments were possible in an ice sheet.

Ice forces were recorded by a magnetic tape recorder and monitored by a pen recorder. A 16mm movie camera was used to record the ice breaking patterns.

The diameter of the models and the towing speed were chosen as experimental parameters. The thickness of ice sheets were kept almost the same, 4-5cm, because the experiments with the thinner ice sheets of 1-2cm thickness were very difficult, and to form thicker ice sheets takes more time.

The characteristics of ice used in the tests

Transverse section of the ice sheet is shown in Photo 1. The upper layer of 5-7mm thickness was the granular ice consisting of small grains of 0.5-1.0mm diameter. Below the granular ice existed the columnar ice whose grain sizes were 4-6mm. The salinity of upper layer and columnar zone were 0.16 and 0.27% respectively. The compressive strengths of ice in horizontal direction had statistic feature. Fig. 1 shows an example of histogram of compressive strength of test specimens sampled from an ice sheet. We will use the mean value.

The flexural strength was also measured by the cantilever test. The length and breadth of cantilever specimen was 30cm and 5cm respectively. A motor driven cantilever testing equipment was used for the test.

The relation between the towing speed and the ice resistance

The measured ice force generally had many peaks.

The examples of peak values' histogram obtained by the tests, in which the ice sheets failed in the crushing mode and in buckling mode, are shown in Figs. 2 and
3, respectively. As shown in these figures, in crushing mode, the ice pressure is stable, and in buckling mode the ice pressure fluctuates in a wide range.

Hereafter, we will use mean values of the peak loads.

The relation between the towing speed and the ice pressure $P=F/(D.t)$ is shown in Fig. 4, where $F$: the ice force acting on the model, $D$: the diameter of the model and $t$: the thickness of ice sheet.

Generally speaking, the ice breaking patterns became crushing mode in the higher towing speed range, and in the lower towing speed range, became buckling mode. In Fig. 4, + marks show the buckling mode.

Further, when the diameter of model became larger in a constant towing speed, the breaking pattern became buckling mode instead of crushing mode.

Especially when the towing speed was slower than 2 cm/sec, almost all breaking patterns were buckling modes. (It is very interesting to note that in some cases, the ice pressures of buckling mode were larger than those of crushing mode.)

The fracture pattern of crushed ice sheets were observed and shear-type fractures as shown in Fig. 5 were found. No cleavage cracks were found.

As shown in Fig. 4, the faster the towing speed of the model, the smaller the ice pressure became. But, to our regret, we could not carry out the experiments in slower towing speed due to the restriction of the capacity of the towing carriage and it has not been made clear whether the ice pressure becomes larger or smaller when towing speed is slower than 2 cm/sec.

There are evident differences between the ice pressures in the group of $-0.7$ to $-0.5^\circ$C ice temperature, but the tendency of the ice pressure reducing with the increase in towing speed, is the same for both groups.

The relation between the ice pressure and the diameter of the model

In actual design, the relation between the ice pressure and the diameter of a pile is very important. Fig. 6 shows this relation obtained from our laboratory tests. In Fig. 6 results of our field tests are also plotted. The field tests were conducted at Lake Saroma in Hokkaido in 1979. In these tests, the cylindrical models of 40 and 80cm diameter were indented into about 40cm thickness ice sheet by a hydraulic jack. The horizontal compressive strength of ice was measured by using cubic specimens (10cm x 10cm x 10cm), which was about $15$ kg/cm$^2$.

Fig. 6 shows that the ice pressure becomes smaller as the pile's diameter increases. In Fig. 6, an empirical formula proposed by Saeki, et al.(1) is also plotted. Our results give higher values than Saeki's formula.

Reason for this is not clear. But we think that one reason may be because of difference in the methods of evaluating the horizontal compressive strength of ice.
Conclusion

The laboratory ice force tests on cylindrical pile were conducted and the result were obtained as follows.

(1) The faster the towing speed, the smaller the ice pressure.
(2) In the low-towing speed range, the fracture in the buckling mode is dominant instead of the crushing mode.
(3) The ice pressure becomes smaller with increase in the pile's diameter.
(4) The ice pressures obtained by us give higher values than those by Saeki, et al.

References
(3) Affanas'yev, V. P., Dologoplov, Iu. V., Shrayshteyn, Z. I.: Ice Pressure on Separate Supporting Structures in the Sea; CRREL translation TL346, 1972
Photo 1 Laboratory Test

Photo 2 Micrographic Structure of Ice Used in the Test

Fig. 1

Compressive strength (kg/cm²)

Mean 1.10
22 samples

Frequency

0.5 1.0 1.5 20 25

Fig. 1
Fig. 5

Fig. 6
Friction Measurements of Sea Ice on Some Plastics and Coatings

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Abstract
Measurements were made of coefficients of kinetic friction between ice and various materials such as iBMA, PMMA, Copolymer, Polymer Sealant, Kanpe-glass, SD₂-X, Vini-Bon-100, Vellox-140, Teflon, Tar Epoxy, inata-160 and stainless steel. Those except stainless steel are among a variety of coatings and paints which have been developed recently to reduce friction between sea ice or freshwater ice and the ship's hull or the marine structure. The normal stress applied between an ice specimen and a test substrate was smaller than 0.25 MPa, which was changed while a test run, the temperature being -5°C and the sliding speed being 0.08 mm/s. Besides, some tests were conducted at a constant normal stress of 0.11 MPa, subjecting a specimen to the sliding speed of 0.44 mm/s at -5°C and 0.08 mm/s at -10°C.

The following results were obtained: The coefficient was almost independent of the normal stress; that is, Amonton's law of friction is also applicable to the friction of sea ice. Most of materials show such low coefficients as range from 0.017 to 0.046. Vellox-140 shows an extremely high coefficient. In case of stainless steel, the coefficient is proportional to the roughness of mean square. Finally, SD₂-X shows a decrease in coefficient with lowering temperature because of its special thermal behaviour.

1. Introduction
Friction between sea ice and the ice breaker or many kinds of marine structures presents a very important problem for their
operation in the arctic condition. The authors reported in POAC, 1979, on the value of coefficient of kinetic friction between sea ice and materials including various metals, some plastics and coatings.

This paper provides the new data of coefficient of kinetic friction between sea ice and newly developed coatings. Measurements were made using the same apparatus as was used in the previous work (Tusima and Tabata, 1979); that is, a cylindrical ice specimen 75 mm in diameter was pressed to the plane of a substrate by the force $W$ ranging from 13 N to 1000 N, normal to the contact plane. The substrate was prepared by painting a test material on a steel plate, 0.4 mm thick, 50 cm long and 12 cm wide. The ice specimen was fixed through an electric load cell and the substrate plate was moved at constant speed under the bottom surface of the ice specimen and a record was made of the shear force $F$ parallel to the surface of contact necessary to keep the ice specimen fixed against the movement of the test substrate.

Figure 1 represents one of the results reported in the previous work. Shear stress is shown against normal stress applied to the contact surface between ice and substrates. The report of POAC, 1977, carries the same figure, in which two mistakes were made. For correction the numerical value attached to the vertical axis should be doubled and the sliding speed $V$ should be $2 \times 10^{-3}$ m/s, as they are seen in the present figure.

The coefficient of kinetic friction $\mu_k$ is defined as

$$F = \mu_k W$$

(1)

It is well-known that, if Amonton's law is satisfied, $\mu_k$ is independent of the apparent contact area and also of the applied load $W$. Dividing equation (1) by the apparent contact area,

$$\tau = \mu_k \sigma$$

(2)

where $\sigma$ is the normal stress to the contact surface and $\tau$ is the shear stress to maintain sliding.

It is clear from this figure that the relation between $\tau$ and $\sigma$ can be represented as

$$\tau = S_0 + \mu_k \sigma$$

(3)

and, as seen in the figure, in most cases $S_0$ is equal or almost equal to zero. That is, for the friction of sea ice, if the normal stress $\sigma$ is smaller than 2.5 bars, $\mu_k$ does not depend on the normal stress. That is, Amonton's law of friction is applicable.
From the previous work, following conclusions were obtained:

a) On the glass plate the coefficient of kinetic friction shows the smallest value of 0.009. The very smooth surface of stainless steel also shows such a fairly small coefficient as 0.02 - 0.06; carbon steel showed such a high value as ranges from 0.05 - 0.15.

b) The coefficient increases with lowering temperature.

c) The coefficient increases with decreasing sliding speed.

d) The existence of sea water at the surface of contact gives little effect on a friction behaviour.

e) Freshwater ice shows a slightly greater coefficient of kinetic friction than sea ice which means that liquid brine in sea ice, which moistens the contact surface, produces little effect as a liquid
lubricant.

2. Test substrates

Table I shows substrates used in the measurement.

<table>
<thead>
<tr>
<th>Dianal BR-101</th>
<th>iBMA Isobutylmeta acrylate (Homopolymer)</th>
<th>Mitsubishi Rayon</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dianal BR-80</td>
<td>PMMA (Polymethlmetaacrylate (Homopolymer)</td>
<td>&quot;</td>
</tr>
<tr>
<td>Copolymer</td>
<td>Polycarbonate + Polysiloxane</td>
<td>Resin, GE USA CRREL</td>
</tr>
<tr>
<td>Polymer Sealant</td>
<td>Amin curable Epoxycoating</td>
<td>Kansai Paint</td>
</tr>
<tr>
<td>Kanpe-glass</td>
<td>Epoxy-Acrylate + Glass Flake</td>
<td>&quot;</td>
</tr>
<tr>
<td>SD2-X</td>
<td>Polydimethylsiloxane</td>
<td>&quot;</td>
</tr>
<tr>
<td>Vini-Bon-100</td>
<td>Vinylchlorid + Vinylacetate (Copolymer)</td>
<td>&quot;</td>
</tr>
<tr>
<td>Vellox-140</td>
<td>Hydrophobic Silica</td>
<td>U.S.A. Bondix</td>
</tr>
<tr>
<td>Teflon</td>
<td>PTFE</td>
<td>Nittö-Denkō</td>
</tr>
<tr>
<td>Tar-epoxy</td>
<td></td>
<td>Kansai Paint</td>
</tr>
<tr>
<td>inata-160</td>
<td></td>
<td>Nippon Paint</td>
</tr>
<tr>
<td>Stainless steel</td>
<td>Smooth surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium rough surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rough surface</td>
<td></td>
</tr>
</tbody>
</table>

Epoxy of special quality was used in case of Epomarine. Meanwhile, SD2-X is a special coating which is being developed; so, it is not available commercially.

3. Experimental Results

a) Kinetic friction coefficient

Measurements were carried out at a room temperature of -5°C and at a sliding speed of 0.08 mm/s, changing the normal force from 13 N to 1000 N as it was applied to the ice specimen. It follows from the apparent contact area of $4.4 \times 10^{-3} \text{m}^2$ that the applied normal stress to the ice-substrate interface ranged from 0.003 MPa to 0.2 MPa. Besides, two other tests were conducted at the condition, that is, the
constant normal stress: 0.1 MPa; the sliding speed: 0.4 mm/s and 0.08 mm/s respectively at -5°C and -10°C.

The ice specimen used was 75 mm in diameter and 100 mm in length, which was cut out from the ice sheet of Lake Saroma, a salt lake with a salinity of water of 30%.

Work of painting or coating to the steel plate was made by Kansai paint Co., Ltd.
Figures 2 to 4 show an obtained relation between normal stress and shear stress. It is clear from the figures, as is also seen in Fig. 1, that the normal stress to the contact surface is proportional to the shear stress, and hence the coefficient of kinetic friction is obtained as the gradient of the line. The numerical value of the coefficient is given in a parenthesis in each line.

Epomarine, iBMA and PMMA show the small coefficient of kinetic friction ranging from 0.017 to 0.020, which is almost the same value as that of the smooth surface of stainless steel. Copolymer of CRREL, Vini-Bon-100, SD$_2$-X, PTFE and inata-160 forms a record group of
coefficients from 0.027 to 0.033. With coefficient values given in the following parentheses, Tar-epoxy (0.046), Polymer Sealant (0.14), Kanpe-glass (0.20) and Vellox-140 (0.43) show a high friction behaviour. Vellox-140 shows the highest coefficient of kinetic friction.

The authors obtained 0.24-0.27 as the kinetic friction coefficient of concrete at -2°C for the sliding speed of 0.08-0.40 mm/s. Such a value as 0.43 seems to be an extraordinary high value. Vellox-140 is well-known as a hydrophobic coating which gives extreme water repellency to any substrate. A contact angle between a water droplet and a coated surface is 130°-150°, which very large. Indeed, such an excellent hydrophobic behaviour is seen at a normal temperature, but
not observed in a cold condition, even at -5°C; besides, it shows a very high friction coefficient.

Copolymer presents much difficulty in coating technique. The main component of Copolymer and Polymer Sealant does not change much; however, the latter has a very high coefficient. It may be due to contained silicon, which is necessary to show a glossy appearance after polishing.

The amount of surface free energy was calculated by Kansai Paint Co., according to which Copolymer, Polymer Sealant, SD₂-X, Kange-glass and Teflon show a small amount of free energy less than 30 m N/m. Simultaneously, the adhesive strength with freshwater ice was also measured, the finding being that with the exception of Teflon, other four materials showed almost no adhesion to ice. Teflon showed a small adhesive strength but other materials, including Vellox-140, showed a high adhesive strength (Unpublished data).

b) Effect of surface roughness

The coefficient of kinetic friction of stainless steel was obtained by three different surface treatments, very smooth, medium rough and rough. The roughness of mean square (RMS) was also measured by a surface roughness meter. Table II shows the results.

<table>
<thead>
<tr>
<th>RMS</th>
<th>₁₄₅</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.07</td>
<td>0.017</td>
</tr>
<tr>
<td>0.26</td>
<td>0.024</td>
</tr>
<tr>
<td>1.79</td>
<td>0.081</td>
</tr>
</tbody>
</table>

The friction coefficient linearly increases with increasing RMS, which suggests the importance of a ploughing effect on the kinetic friction of rough surface.

c) Effects of temperature and sliding speed.

To find effects of temperature and sliding speed on the coefficient of kinetic friction, tests of constant normal stress (0.11 MPa) were made and the results are given in Table III.
Table III  Kinetic Friction Coefficient

<table>
<thead>
<tr>
<th>Sliding speed</th>
<th>0.08 mm/s</th>
<th>0.4 mm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature</td>
<td>-5°C</td>
<td>-10°C</td>
</tr>
<tr>
<td>iBMA</td>
<td>0.017</td>
<td>0.017</td>
</tr>
<tr>
<td>PMMA</td>
<td>0.020</td>
<td>0.017</td>
</tr>
<tr>
<td>Epomarine</td>
<td>0.017</td>
<td>0.017</td>
</tr>
<tr>
<td>Stainless steel (smooth)</td>
<td>0.020</td>
<td>0.025</td>
</tr>
<tr>
<td>Stainless steel (m.rough)</td>
<td>0.024</td>
<td>0.058</td>
</tr>
<tr>
<td>Copolymer</td>
<td>0.027</td>
<td>0.025</td>
</tr>
<tr>
<td>Vini-Bon-100</td>
<td>0.027</td>
<td>—</td>
</tr>
<tr>
<td>SD₂-X</td>
<td>0.028</td>
<td>0.017</td>
</tr>
<tr>
<td>Teflon</td>
<td>0.031</td>
<td>0.025</td>
</tr>
<tr>
<td>inata-160</td>
<td>0.033</td>
<td>0.11</td>
</tr>
<tr>
<td>Tar-epoxy</td>
<td>0.046</td>
<td>0.10</td>
</tr>
<tr>
<td>Stainless steel (rough)</td>
<td>0.081</td>
<td>0.14</td>
</tr>
<tr>
<td>Polymer Sealant</td>
<td>0.14</td>
<td>0.14</td>
</tr>
<tr>
<td>Kanpe-glass</td>
<td>0.20</td>
<td>0.28</td>
</tr>
<tr>
<td>Vellox-140</td>
<td>0.43</td>
<td>0.48</td>
</tr>
</tbody>
</table>

The tendency of increasing coefficient with lowering temperature is noted. The interesting effect is seen in SD₂-X. Materials in this table other than SD₂-X decrease their volume with lowering temperature. However, SD₂-X increases the volume with lowering temperature. Such a special thermal property may cause a decrease in kinetic coefficient with lowering temperature.

In this test, because of the shortage of data, it is difficult to define the effects of temperature and sliding speed.

4. Conclusion

The coefficient of kinetic friction between sea ice and a variety newly developed coatings (Table I) was measured with the normal stress smaller than 0.25 MPa at -5°C and -10°C, the sliding speed being 0.08 mm/s and 0.40 mm/s.

1. The coefficient was almost independent of normal stress. That is,
Amonton's law of friction is also applicable to the friction of sea ice.

2. Most of materials show the low coefficient as ranges from 0.017 - 0.046.

3. Vellox-140 shows an extremly high coefficient.

4. In case of stainless steel, the coefficient is proportional to the roughness of mean square.

5. SD₂-X shows a decrease in coefficient with lowering temperature because of its special thermal behaviour.

Acknowledgements

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Reference

Katutosi TUSIMA and Tadashi TABATA. 1979. Friction measurements of sea ice on flat plates of metals, plastics and coatings. POAC Proceedings
EXPERIMENTAL STUDY ON FLEXURAL STRENGTH AND ELASTIC MODULUS OF SEA ICE

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Akira Ozaki
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ABSTRACT

The purpose of the present investigation is to make clear the difference in flexural strength and elastic modulus of sea ice which is produced by difference in testing method and testing scale. Three kinds of testing methods on flexural strength and elastic modulus were carried out in different scales (small-scale test, intermediate-scale test and large-scale test). The results of this study may be stated as follows:

1. Poisson's ratio of sea ice in southern Okhotsk Sea changed with the stress rate.
2. The deflection curve and bearing capacity of ice sheet calculated from Kubo's theory agreed with the test results before the occurrence of initial cracks.
3. The flexural strength and elastic modulus obtained from small-scale test showed larger values in comparison with the values obtained from another tests at same stress rate.
4. The flexural strength and elastic modulus obtained from insitu cantilever test (intermediate-scale test) agreed well with the values obtained from bearing capacity test in the field (large-scale test) at same stress rate.

1. Introduction

In order to decide the ice forces on sloping structures and safe bearing capacity of sea ice cover, it is very important to clarify the flexural strength and effective modulus (i.e., elastic modulus and Poisson's ratio) of real sea ice cover. But, measuring the flexural strength and the effective modulus by means of large-scale test is very difficult and takes plenty of time. The purpose of the present investigation is to make clear the difference in flexural strength and elastic modulus of sea ice which is produced by difference in testing method and testing scale. Three kinds of testing methods on flexural strength and elastic modulus were carried out
in different scales at Toppushi fishery harbour by Lake Saroma which was connected with the Okhotsk Sea by two channels. Three kinds of testing methods were,

1) Small-scale test; this test was carried out for the case of 3-point loaded and 4-point loaded simple beams with rectangular cross section. The span length was from 40 cm to 100 cm, and the height and the width of beams were 10 cm. Loading rate was changed in wide range.

2) Intermediate-scale test; this test was carried out for the case of insitu cantilever beam in the ice field. Beam size was 30 cm in thickness, 30 cm in width and 200 cm in length. The load direction was upward, and the stress rate was 0.03kg/cm²/sec.

3) Large-scale test; this test corresponds to bearing capacity test for the infinite floating ice plate subjected to a load uniformly distributed over a circular area (100 cm in diameter) in the field. In the test, the deflection and the load in the case of occurrence of initial cracks of ice cover were measured, and loading rate and stress rate were 15kg/sec and 0.03kg/cm²/sec respectively. By comparing the test values with the values of deflection and bearing capacity calculated from Kubo's theory, mean values of flexural strength and elastic modulus of ice cover were determined. In addition to these tests, the experiments on Poisson's ratio were carried out.

2. Testing Method and Apparatus
Experiments were carried out at Toppushi fishery harbour as shown in fig. 1. The ice used in simple beam tests were collected at sites C and cantilever tests were carried out at site B. The air temperature and the vertical distribution of ice temperature were measured at site A. Bearing capacity tests were carried out at sites 1 ~ 7. The ice of sites 1 ~ 4 were granular ice and different from the ice of sites 5,6 and 7. The mean ice temperature in day time was about -1.7 ~ -1.8°C.

1) Measurement of Poisson's ratio for sea ice
Poisson's ratio of sea ice has been measured by Linkov, Oliver and Ishida by means of seismic method. But as mentioned by Schwarz, it is important to distinguish between the dynamic and the static method. The authors adopted the static method, and a newly designed experimental apparatus, as shown in fig. 2, was made for the measurement of strain in radial direction in uniaxial compressive strength tests. In uniaxial compression tests, cylindrical cores were adopted for specimens (10 cm in diameter and 20 cm in height) and the three apparatus for radial strain were installed to specimen at intervals of 5 cm as shown in fig. 3. The vertical strain $\varepsilon_L$ and the radial strain $\varepsilon_I$ are obtained from eq. (1) and (2).

$$\varepsilon_L = \Delta l/l \quad \text{(1)}$$

$$\varepsilon_I = 2\pi\Delta Y/2\pi Y = \Delta Y/Y \quad \text{(2)}$$
Notations $\Delta$, $\Delta\ell$, $\gamma$ and $\Delta \gamma$ are shown in fig. 4. As the radial strain $\varepsilon_1$ was measured at three points, Poisson's ratio $\nu$ can be obtained from eq. (4)

$$\varepsilon_1 = (\varepsilon_1 + \varepsilon_2 + \varepsilon_3)/3 \quad (3)$$

$$\nu = \frac{\varepsilon_1}{\varepsilon_1} \quad (4)$$

(2) Insitu cantilever test

In order to obtain the elastic modulus $E$ and the flexural strength $\sigma_f$ of sea ice cover, the insitu cantilever test, as shown in fig. 5, was adopted as a intermediate-scale test and loading direction was upward. Stress rate $\dot{\sigma}_f$ and strain rate $\dot{\varepsilon}$ were continuously adjusted by a variable pulley. The measurements of load and deflection were carried out by means of load cell and displacement transducer respectively. The size of cantilever beams were 30 cm in width, 30 cm in thickness and 200 cm in length.

(3) Simply supported beam test

Simple beam test was adopted as a small-scale test, and the size of specimen was 10 cm in width $b$ and 10 cm in height $d$, and the span length $\ell$ was 40, 60, 80 and 100 cm. The test was carried out by means of two kinds of tests, 3 points loading test and 4 points loading test, and these tests were usually carried out within one hour since the time of the collection of sea ice.

(4) Bearing capacity test of infinite floating ice

When a load is placed on a floating ice sheet, the ice sheet is bent downward. If the ice sheet is elastic substance, the flexural strength $\sigma_f$ and the elastic modulus $E$ of the ice sheet can be obtained by means of making direct comparison of the experimental results of deflection and bearing capacity of ice sheet with the theoretical values. The flexural strength $\sigma_f$ and the elastic modulus $E$ obtained from bearing capacity tests were regarded as the values obtained from large-scale tests. The schematic of bearing capacity test is shown in fig. 6. The load and loading rate could be adjusted by means of pouring sea water into FRR water tank by submerged pump. Loading plate was made from steel and its diameter is 100 cm. The deflections of ice sheet were measured by 12 tilting levels and the accuracy of the measurement for deflections was ±0.2 mm. The load at the time of the occurrence of initial cracks (radial cracks) was adopted as the bearing capacity $P_{cr}$ and initial cracks were confirmed by means of the sound and the vibration of ice sheet.

3. Experimental Results

(1) Test results of Poisson's ratio

The measurements of Poisson's ratio $\nu$ for sea ice have been carried out by means of dynamic method. Oliver and Peshansky reported that the values for pack ice in the Arctic Ocean were 0.27 ~ 0.32 and 0.29 respectively. Experimental results on poisson's ratio are shown in fig. 7. Judging from the results, Poisson's ratio of
sea ice depends on the ice temperature and the stress rate $\dot{\sigma}_C$ and it may be seen that the lower the ice temperature becomes, the larger the values of Poisson's ratio become and the peak of Poisson's ratio $v$ exists within a range of $\dot{\sigma}_C = 1 \sim 10 \text{ kg/cm}^2 \cdot \text{sec}$. Experimental results of Poisson's ratio($\dot{\sigma}_C = 1.0 \text{ kg/cm}^2 \cdot \text{sec}$) are shown in Table-1. The mean value of Poisson's ratio $\bar{v}$ is about 0.100 and it is smaller than another researcher's results. The reason depends on porous ice and static method.

(2) Results of insitu cantilever test

In order to clarify the elastic modulus and the flexural strength of real sea ice, insitu cantilever test has been adopted. In this test, deflection of beam $\delta$ was measured accurately at the tip of beam. In order to eliminate the influence of deflection of ice sheet near the root of beam, the basement of rod which supported the displacement transducer could incline with the inclination of ice sheet. To compare the results of this test with the results of another tests, this test was carried out at constant stress rate $\dot{\sigma}_f = 0.03 \text{ kg/cm}^2 \cdot \text{sec}$. The test results are shown in Table-2. The mean flexural strength $\bar{\sigma}_f$ and the mean elastic modulus $\bar{E}$ are 2.995 kg/cm$^2$ and 9177 kg/cm$^2$ respectively. These experimental values coincide with the results obtained by Tabata in Okhotsk Sea at the same test conditions.

(3) Results of simple beam test

The tests were carried out by means of the 3 points loading and 4 points loading tests. Span length $l$ was variable from 40 cm to 100 cm and height $d$ and width $b$ of beam were 10 cm respectively. It is well known that the differences in testing method and in size of specimen produce different strengths of materials. Figure 8 shows the relation between the flexural strength $\sigma_f$ and span length $l$, and notation $\text{Dgr}$ is grain diameter of specimen. Within a range of $l/\text{Dgr} > 50$, $\sigma_f$ shows large change with the change of $l/\text{Dgr}$ and within a range of $l/\text{Dgr} < 50$, $\sigma_f$ shows constant value. The flexural strength obtained from 3 points loading test is nearly equal to that obtained from 4 points loading test. Figure 9 shows the relation between flexural strength and its stress rate $\dot{\sigma}_f$, and it may be seen that the flexural strength shows the maximum within a range of $\dot{\sigma}_f = 1 \sim 2 \text{ kg/cm}^2 \cdot \text{sec}$. The relation between the flexural strength and the ice temperature is shown in fig. 10. To compare the results of this simple beam test with the results of the insitu cantilever and bearing capacity tests, simple beam tests were carried out in much the same test conditions as another two tests and the results are shown in Table-3. The mean values of the elastic modulus $\bar{E}$ and flexural strength $\bar{\sigma}_f$ are 11747 kg/cm$^2$ and 3.548 kg/cm$^2$ respectively, and each value is larger than the values obtained from the insitu cantilever test.

(4) Results of bearing capacity test

It is the purpose of this test to measure the bearing capacity of ice sheet in Okhotsk Sea and to obtain the mean values of the elastic modulus and flexural strength of
ice sheet. The experimental results on deflection of ice sheet are shown in fig. 11, 12, and 13. The notations $W$, $P$, and $Y$ are deflection of ice sheet (cm), load (kg) and distance from the center of loaded area (m) respectively. The experimental results agree with Kubo's theory within a range of $P < 1500$ kg and within a range of $P > 1500$ kg, the deflection near the loaded area shows large values in comparison with the deflection curve calculated from Kubo's theory. The relations between the load $P$ (metric ton) and the deflection in center of loaded area $W_0$ (mm) are shown in fig. 14, 15, and 16. In the Figures, the initial cracks occur at the point of $\times$. From fig. 14 ~ 16, the relation between the deflection $W_0$ and the load $P$ can be regarded linear before the occurrence of initial cracks and this fact suggests that Kubo's theory based on elastic theory can be applied to the calculation of deflection of ice sheet within a range of $P \leq P_C^{\gamma}$. According to Kubo's theory, the equations for the deflection of uniform load over a circular area (radius $a$) and concentrated load are shown in eq. (5) and (6) respectively.

\[
W_0 = \frac{P}{8(\gamma D)} \frac{1}{\gamma^3} \tag{5}
\]

\[
W_0 = \frac{(1 + C_1)P}{\gamma \cdot \pi \cdot a^2} \tag{6}
\]

Notation $\gamma$ is specific weight of sea water, $a$ is the radius of the circular area to the uniform load and $D$ is the flexural rigidity of the ice sheet as shown in eq. (7).

\[
D = \frac{Eh^3}{12(1-\nu^2)} = Kh^3 \tag{7}
\]

$C_1$ is the function of $(a/\ell)$ and $C_2$ is shown as follow,

\[
\ell = (D/\gamma)^{1/3} \tag{8}
\]

The authors calculated the elastic modulus from $W_0/P$ obtained from fig. 14 ~ 16, eq. (5), (6) and (7) as the Poisson's ratio $\nu = 0.100$ as shown in Table-4. The mean value of elastic modulus $E$ obtained from bearing capacity test is 9258 kg/cm$^2$. The bearing capacity $P_{cr}$ is calculated from eq. (9)

\[
P_{cr} = \frac{1}{3} \cdot \frac{\pi}{1+\nu} \cdot \frac{\sigma_f^2 \cdot C_2 \cdot h^2}{C_2} \tag{9}
\]

In eq. (9), $\alpha = (a/\ell)$ and $C_2$ is the function of $\alpha$. The flexural strength of ice sheet as large-scale test can be calculated from eq. (9) by using $P_{cr}$ obtained from fig. 14 ~ 16, $E$ shown in Table-4 and $\nu = 0.100$. The results are shown in Table-5.

The mean value of flexural strength $\sigma_f$ is 2.738 kg/cm$^2$.

4. Conclusion

The results of this study may be stated as follows:

1. Poisson's ratio of sea ice in southern Okhotsk Sea changed with stress rate, strain rate and ice temperature.
2. The flexural strength and elastic modulus obtained from simple beam test (small-scale test) showed larger values in comparison with the values obtained another tests at same stress rate, and the difference between 3 points loading test and 4 points loading test has not an effect on the flexural strength.

3. The deflection curve and bearing capacity of ice sheet calculated from Kubo's theory agreed with the test results before the occurrence of initial cracks.

4. The flexural strength and elastic modulus obtained from insitu cantilever test (intermediate-scale test) agreed well with the values obtained from bearing capacity test in the ice field at same stress rate.

ACKNOWLEDGEMENT
We wish to express our profound gratitude for the extensive cooperation given by Mr. Toshiyuki Ono of Hokkaido University and Mr. Kazunori Nishiya, former student, in the experiments and arrangement of the data. This work was partly supported by the Scientific Research Funds from the Ministry of Education, Japan.

REFERENCES


fig. 1 Location of Toppushi fishery harbour and test sites

fig. 2 Apparatus for the measurement of strain in radial direction

UNIT: mm

fig. 3 Apparatus for radial strain installed to specimen at intervals of 5 cm

fig. 4 Definition sketch for the measurement of Poisson's ratio
fig. 5  
Schematic of insitu cantilever test

fig. 6  
Schematic of bearing capacity test

fig. 7  
Experimental results of Poisson's ratio for porous sea ice
Table - 1 Experimental results of Poisson's ratio at $\dot{\sigma}_c = 1.0 \text{ Kg/cm}^2 \cdot \text{sec}$.

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>$\rho$ (gr/cm$^3$)</th>
<th>$T$ (°C)</th>
<th>$\sigma_c$ (Kg/cm$^2$)</th>
<th>$\Delta l$ (mm)</th>
<th>$\epsilon_{x-1}$ $\times 10^{-6}$</th>
<th>$\epsilon_{1}$ $\times 10^{-6}$</th>
<th>$\epsilon_{2}$ $\times 10^{-6}$</th>
<th>$\epsilon_{3}$ $\times 10^{-6}$</th>
<th>$\epsilon_{4}$ $\times 10^{-6}$</th>
<th>$\nu$ = $\epsilon_{4}$ $\epsilon_{1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.895</td>
<td>-1.0</td>
<td>14.81</td>
<td>1.04</td>
<td>5.2</td>
<td>4.0</td>
<td>5.6</td>
<td>2.8</td>
<td>4.1</td>
<td>0.079</td>
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<tr>
<td>6</td>
<td>0.852</td>
<td>-1.1</td>
<td>16.46</td>
<td>0.96</td>
<td>4.8</td>
<td>3.3</td>
<td>4.5</td>
<td>2.6</td>
<td>3.5</td>
<td>0.073</td>
</tr>
<tr>
<td>7</td>
<td>0.862</td>
<td>-1.4</td>
<td>13.54</td>
<td>0.88</td>
<td>4.4</td>
<td>4.8</td>
<td>6.7</td>
<td>2.1</td>
<td>4.5</td>
<td>0.102</td>
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<tr>
<td>80</td>
<td>0.887</td>
<td>-1.5</td>
<td>15.20</td>
<td>1.20</td>
<td>6.0</td>
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<td>4.0</td>
<td>0.067</td>
</tr>
<tr>
<td>81</td>
<td>0.893</td>
<td>-2.0</td>
<td>20.10</td>
<td>0.52</td>
<td>2.6</td>
<td>2.9</td>
<td>4.0</td>
<td>2.3</td>
<td>3.1</td>
<td>0.119</td>
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<tr>
<td>82</td>
<td>0.894</td>
<td>-2.0</td>
<td>22.30</td>
<td>1.52</td>
<td>7.6</td>
<td>9.6</td>
<td>11.3</td>
<td>7.1</td>
<td>9.3</td>
<td>0.122</td>
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<tr>
<td>83</td>
<td>0.893</td>
<td>-1.8</td>
<td>15.10</td>
<td>1.28</td>
<td>6.4</td>
<td>4.2</td>
<td>19.3</td>
<td>2.8</td>
<td>8.8</td>
<td>0.138</td>
</tr>
</tbody>
</table>

$T = -1.54 \degree C$ $\dot{\sigma}_c = 16.8 \text{ Kg/cm}^2$ $\nu = 0.100$

Table - 2 Experimental results of insitu cantilever test

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>LOAD P (Kg)</th>
<th>LENGTH $l$ (cm)</th>
<th>WIDTH b (cm)</th>
<th>THICKNESS h (cm)</th>
<th>$\sigma_f$ (Kg/cm)</th>
<th>$\dot{\sigma}_f$ (Kg/cm $\cdot$ s)</th>
<th>$\delta$ (cm)</th>
<th>$E$ (Kg/cm)</th>
<th>$T$ (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>53.71</td>
<td>194</td>
<td>30.0</td>
<td>30.9</td>
<td>2.18</td>
<td>0.028</td>
<td>0.203</td>
<td>8730</td>
<td>-1.7</td>
</tr>
<tr>
<td>2</td>
<td>59.52</td>
<td>200</td>
<td>29.5</td>
<td>30.2</td>
<td>2.66</td>
<td>0.034</td>
<td>0.247</td>
<td>9490</td>
<td>-1.7</td>
</tr>
<tr>
<td>3</td>
<td>66.93</td>
<td>196</td>
<td>30.0</td>
<td>30.3</td>
<td>2.86</td>
<td>0.033</td>
<td>0.266</td>
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</tr>
<tr>
<td>4</td>
<td>81.56</td>
<td>198</td>
<td>30.0</td>
<td>30.4</td>
<td>3.50</td>
<td>0.035</td>
<td>0.325</td>
<td>9245</td>
<td>-1.7</td>
</tr>
<tr>
<td>5</td>
<td>89.38</td>
<td>199</td>
<td>30.5</td>
<td>30.4</td>
<td>3.79</td>
<td>0.038</td>
<td>0.352</td>
<td>9341</td>
<td>-1.7</td>
</tr>
</tbody>
</table>

LOAD DIRECTION---UPWARD $\sigma_f=2.995 \text{ Kg/cm}^2$ $E=9177 \text{ Kg/cm}^2$

**fig. 8** Relation between $\sigma_f$ and $l/D_{cr}$ for the 3-points and 4-points loading test
fig. 9 Relation between flexural strength $\sigma_f$ and stress rate $\dot{\sigma}_f$

![Graph showing relation between flexural strength $\sigma_f$ and stress rate $\dot{\sigma}_f$.]

- $b=d=10$ cm
- $l=100$ cm
- $\rho=0.87$ g/cm$^3$
- $T=-7$ °C

4 Points Loading

![Graph showing 4 Points Loading with $\sigma_f$ vs $\dot{\sigma}_f$.]

Table 3 Experimental results of simply supported beam test

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>LENGTH (cm)</th>
<th>HEIGHT (cm)</th>
<th>WIDTH (cm)</th>
<th>$\rho$ (g/cm$^3$)</th>
<th>$\ddot{\sigma}_f$ (Kg/cm$^2$·s$^{-1}$)</th>
<th>$E$ (Kg/cm$^2$)</th>
<th>$\sigma_f$ (Kg/cm$^2$)</th>
<th>$T$ (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>103</td>
<td>100</td>
<td>10</td>
<td>10</td>
<td>0.88</td>
<td>0.085</td>
<td>11504</td>
<td>3.62</td>
<td>-1.7</td>
</tr>
<tr>
<td>104</td>
<td>100</td>
<td>10</td>
<td>10</td>
<td>0.88</td>
<td>0.072</td>
<td>10887</td>
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<td>-1.8</td>
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<td>105</td>
<td>100</td>
<td>10</td>
<td>10</td>
<td>0.88</td>
<td>0.046</td>
<td>12162</td>
<td>3.78</td>
<td>-1.8</td>
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<td>106</td>
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<td>10</td>
<td>0.88</td>
<td>0.066</td>
<td>11058</td>
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<td>107</td>
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<td>10</td>
<td>0.88</td>
<td>0.042</td>
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<td>100</td>
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<td>10</td>
<td>0.87</td>
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<td>13155</td>
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<tr>
<td>112</td>
<td>100</td>
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<td>10</td>
<td>0.87</td>
<td>0.057</td>
<td>12005</td>
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<td>-1.8</td>
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<tr>
<td>113</td>
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<td>10</td>
<td>0.87</td>
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<td>11873</td>
<td>3.44</td>
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<tr>
<td>114</td>
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<td>10</td>
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<td>10526</td>
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<tr>
<td>115</td>
<td>100</td>
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<td>10</td>
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<td>0.058</td>
<td>12089</td>
<td>3.61</td>
<td>-1.8</td>
</tr>
</tbody>
</table>

$T=-1.8$ ºC  $\ddot{\sigma}_f=3.548$ Kg/cm$^2$  $E=11747$ Kg/cm$^2$
fig. 11
Experimental results on deflection of ice sheet

fig. 12
Experimental results on deflection of ice sheet

fig. 13
Experimental results on deflection of ice sheet
fig. 14 Deflection at the center of loaded area

![Diagram of fig. 14](image)

fig. 15 Deflection at the center of loaded area

![Diagram of fig. 15](image)

fig. 16 Deflection at the center of loaded area

![Diagram of fig. 16](image)

Table 5 Flexural strength $\sigma_f$ calculated from the results of bearing capacity test according to Kubo's theory

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>h (cm)</th>
<th>$a$</th>
<th>$C_2$</th>
<th>$P_{cr}$ (Kg)</th>
<th>$\sigma_f$ (Kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30</td>
<td>0.1663</td>
<td>0.03331</td>
<td>1590</td>
<td>2.235</td>
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<tr>
<td>2</td>
<td>35</td>
<td>0.1422</td>
<td>0.02591</td>
<td>1600</td>
<td>1.758</td>
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<tr>
<td>3</td>
<td>27</td>
<td>0.1628</td>
<td>0.03222</td>
<td>1600</td>
<td>2.803</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>0.1722</td>
<td>0.03521</td>
<td>2200</td>
<td>3.490</td>
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<tr>
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<td>27</td>
<td>0.1482</td>
<td>0.02770</td>
<td>1410</td>
<td>2.562*</td>
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<tr>
<td>6</td>
<td>27</td>
<td>0.1505</td>
<td>0.02541</td>
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<tr>
<td>7</td>
<td>27</td>
<td>0.1413</td>
<td>0.02565</td>
<td>1580</td>
<td>2.925*</td>
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</table>

* $\sigma_f = 2.738$ Kg/cm²

Table 4 Elastic modulus $E$ calculated from the results of bearing capacity test according to Kubo's theory

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>TYPE OF ICE</th>
<th>h (cm)</th>
<th>$\frac{W_p}{a} \times 10^{-1}$ (cm/Kg)</th>
<th>$K_{v=0.100}$ CONCENTRATED LOAD</th>
<th>$K_{v=0.100}$ DISTRIBUTED LOAD</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$K$ (Kg/cm²)</td>
<td>$E$ (Kg/cm²)</td>
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<tr>
<td>1</td>
<td>granular ice</td>
<td>30</td>
<td>13.25</td>
<td>320.03</td>
<td>3802</td>
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<tr>
<td>2</td>
<td>granular ice</td>
<td>35</td>
<td>9.72</td>
<td>374.34</td>
<td>4447</td>
</tr>
<tr>
<td>3</td>
<td>granular ice</td>
<td>27</td>
<td>12.71</td>
<td>477.24</td>
<td>5670</td>
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<tr>
<td>4</td>
<td>granular ice</td>
<td>28</td>
<td>14.25</td>
<td>340.31</td>
<td>4043</td>
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<tr>
<td>5</td>
<td>mosaic ice</td>
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<td>10.56</td>
<td>691.66</td>
<td>8217</td>
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<tr>
<td>6</td>
<td>mosaic ice</td>
<td>27</td>
<td>9.50</td>
<td>893.97</td>
<td>10620</td>
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<td>mosaic ice</td>
<td>27</td>
<td>9.60</td>
<td>836.27</td>
<td>9935</td>
</tr>
</tbody>
</table>

* $E = 9258$ Kg/cm²

547
CREEP OF S₂ ICE BEAMS AND PLATES

J-P. Nadreau, M.Sc.  Université Laval  Québec, Canada
B. Michel, Dr. Eng.  Université Laval  Québec, Canada

Résumé

Le présent article reprend l'analyse d'expériences faites sur des poutres de glace colonnaires d'assez grandes dimensions. Ces poutres étaient chargées de manière à produire une zone de flexion pure. Un système de quadrillage collé sur les poutres, permit de suivre l'évolution des sections droites de la zone centrale. Des résultats obtenus, il a été possible d'en déduire la forme de la répartition des contraintes au droit d'une section droite.

Une équation de déformation, basée sur le comportement des dislocations durant le fluage, nous a permis de décrire de manière très intéressante l'évolution de la flèche centrale de la poutre. Cette même équation utilisée également pour représenter l'évolution du fluage de plaques, s'est avérée très adéquate. Ces résultats très satisfaits devraient permettre de généraliser et de prédire finalement le fluage de couverts de glace.

Abstract

In this article an analysis of the results of experiments done on columnar ice beams is made. These relatively large beams were loaded in order to create a pure bending region. A grid, frozen on one of the faces of the beam, has been used to follow the evolution of cross sections in the central zone. From these results a distribution of stress along a cross section has been proposed.

An equation of strain with respect to time, based on dislocations' behavior during the creep process, has been used satisfactorily to represent the evolution of the central deflection of the beams. This equation has equally been used to represent the creep of ice plates. The results are interesting and encouraging and should lead to a general method to predict creep deformation of ice covers.
INTRODUCTION

The literature dealing with the flexion of ice beams reports mainly two kinds of experiments: On one hand, we can find experiments done in cold rooms with rather small samples (Krausz 1963, Lafleur 1970, Murat 1978). On the other hand, many experiments have been made in situ, but their only purpose was to obtain the maximum resisting moment, without, for most of them, documenting the evolution of the creep process (Drouin and Michel 1972, Gow 1977, Frederking and Hausler 1978).

More recently, Arctic oil research has motivated a significant amount of experiments on sea-ice covers (Vaudrey 1977, Tinawi and Murat 1978, Tryde 1979). The loads and the stress rates involved in these experiments were usually low, staying in the lower part of the ductile region.

In this paper we present the results of tests done on relatively large ice beams (7 x 12 x 100 cm³) which were loaded as heavily as possible in order to obtain higher strain rates and, if possible, tertiary creep (Nadreau, 1976).

THE TESTS

The ice

The ice used for these experiments was S₂ type ice. It was grown in tanks in the laboratory. A controlled technique of growing ice gives very homogeneous crystals: the c-axis is randomly oriented in the horizontal plan (±15°), once the first 5 cm of the cover are sawn off.

The beams

The beams were sawn out of the cover and machined to the final dimensions of 7 x 12 x 10 cm³. A grid made with thin elastic threads was then frozen on one of the larger faces of the beam. The mesh size was 2 cm x 2 cm.

The testing rig and the recording system

Each beam was set on still rests, distant 76 cm apart. The beam was loaded symmetrically at four points, creating uniform pure bending in the region between the central loading points.

Dial gauges were placed so that they could give the deflection of the mid point of the beam and the deflection of each loading point. A camera was used to record the evolution of the beam. A timer would make the camera take a picture at regular time intervals. On each picture, as shown in fig. 1, the following data was available:
- test number
- time t
- dial gauges readings
- grid deformation.

Fig. 1 — A beam being tested.

Test characteristics

Nine beams have been tested. Loads were equal to or greater than 50% of the elastic ultimate capacity. All beams, but one, were loaded up failure. Table 1 gives the characteristics of the tests.

<table>
<thead>
<tr>
<th>Test number</th>
<th>temperature (°C)</th>
<th>Initial load (kg)</th>
<th>Additional loads (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-3</td>
<td>99</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>-3</td>
<td>99</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>-3</td>
<td>66</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>-10</td>
<td>99</td>
<td>10 + 11</td>
</tr>
<tr>
<td>5</td>
<td>-10</td>
<td>120</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>-10</td>
<td>109</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>-10</td>
<td>99</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>-10</td>
<td>99</td>
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</tr>
<tr>
<td>9</td>
<td>-10</td>
<td>109</td>
<td>11</td>
</tr>
</tbody>
</table>

Table 1 — Tests characteristics.
RESULTS

Four out of the nine beams failed between 5 to 35 minutes after load application. As far as the other beams are concerned, fig. 2 shows the central deflection of beams #2 and #3. Beam #2 failed during transitory creep while beam #3 lasted for 8 days when the load was removed, permitting visco-elastic recovery. Figure 3 shows the evolution of the central deflection of beams #4, 8 and 9. The load on each of these beams was progressively increased during the test until secondary creep was considered reached. After each increase, transitory creep was short, compared to the initial transitory creep, and a new secondary strain rate was reached within about a day.

**Fig. 2** — Central deflection for beams at -3°C.

**Fig. 3** — Central deflection for beams at -10°C.
All the beams behaved as expected and followed the typical creep curves shown in fig. 4. Nevertheless, whatever the load could be, fracture occurred without accelerated creep and we never reached any kind of tertiary creep. We would believe, that, on the one hand, \( \kappa \) creep, defined as produced by micro-cracks (Michel 1978) cannot take place on the tensile side of the beam and that, on the other hand, the tests were not long enough to allow syntectonic recrystallization (Lliboutry 1964), defined as by \( \delta \) creep.

![Creep Curves Diagram](image)

**Fig. 4 — Form of creep curves in polycrystalline ice.**

**Evolution of the deflection rate**

From the preceding curves we obtained the deflection rate for each load. The relation between the load and the deflection rate has been computed as:

\[
\dot{\omega} = a \times P^{3.5}
\]  

[1] with \( \dot{\omega} \) = deflection rate  
\( P \) = applied load  
\( a \) = a constant being \( 3.2 \times 10^{-13} \) when \( \dot{\omega} \) is in cm/s and \( P \) in kg.

If we assume that deflection is linearly related to strain and that stress is directly related to load, we come up to the well known relation \( \dot{\varepsilon} = b \sigma^n \) where \( n \) varies, according to authors and types of ice, between 2 to 4. The value of 3.5 looks quite acceptable for polycrystalline ice and is particularly close to the round figure of 3 proposed by Friedel (1956), if we consider in these type of creep tests that we are never at the exact limit of secondary creep.
Evolution of strain on the beams

The grid frozen on the face of the beams has been used to follow the evolution of points situated in the pure bending region. Intersections of vertical and horizontal threads created 90 points, the position of which was read on the pictures with a stereocomparator. Each point was given coordinates x and y. This enables us to calculate strain and strain rate at resolved fibers of the beam. Fig. 5 gives, as an example, the evolution of strain at fibers 5 to -5 for beam #8. The important result obtained from this figure is obviously the fairly good linearity of strain distribution along a cross-section, during ductile deformation.

![Graph showing strain evolution for beam #8.](image)

Stress distribution and effective moment

For the beam represented in fig. 6, the linearity of strain distribution along a cross-section, leads to the fundamental relationship directly resulting from geometry, that is:

$$\varepsilon = \frac{\Delta \ell}{\ell} = \frac{y}{\rho} \text{ at the resolved fiber } y$$

(2)

at extreme fibers, where \( y = h_1 = h_2 \), we have \( \varepsilon_0 = \frac{h}{2\rho} \).

Data from these tests shows equally that strain is directly proportional to the deflection. A constant \( c \) linking strain to non dimensionnal deflection has been computed and leads to the following relation: \( \varepsilon = c \times z \) where \( z = \frac{\omega}{\ell} \). (\( \ell = \text{length of the beam} \).
As we know that strain rate is a power function of stress and that strain distribution remains linear along a cross-section, we can express \( \sigma_0 \), the stress at the extreme fiber, as:

\[
\sigma_0 = k \varepsilon_0^{1/n}
\]

[3]

The stress at fiber \( y \) then becomes:

\[
\sigma = \sigma_0 \left( \frac{2y}{h} \right)^{1/n} \quad \text{(if the neutral axis is at} \ \frac{h}{2} )
\]

[4]

The distribution of stress along a cross section is visualized in fig. 6b.

Once stress distribution along \( y \) is defined, it is simple to obtain the value of the resisting moment from moment definition.

\[
Mr = \int_{h_1}^{0} -\sigma y \, da + \int_{0}^{h_2} \sigma y \, da
\]

[5a]

where \( h_1, h_2 \) are the distances from the neutral axis of extreme fibers

\( \sigma \) is the stress at fiber \( y \)

\( da \) is a surface element.

If the beam has a uniform width \( b \) and if the neutral axis is situated at \( y = 0 \) then:

\[
Mr = 2b \int_{0}^{h/2} \sigma y \, dy
\]

[5b]
with equation [4] defining σ we obtain:

\[ Mr = \frac{n}{(1+2n)} \frac{\sigma_0}{2} bh^2 \]  \[6\]

For a pure elastic stress distribution we know that:

\[ Mr = \frac{\sigma_0}{6} bh^2 \]  \[7\]

In the case of a non-linear stress distribution we could write that:

\[ Mr = \lambda \sigma_0 \frac{bh^2}{6} \]  \[8\]

where \( \lambda \) can be defined as:

\[ \lambda = 6 \int_0^1 \frac{1}{\sigma y} dy \]  \[9\]

with \( y = \frac{v}{h} \) and \( \sigma = \frac{\sigma}{\sigma_0} \).

In our case \( \lambda \) becomes then: \( \lambda = \frac{3n}{(2n+1)} \)

With the usual of \( n = 3 \), the resisting moment is then increased by a value of \( \lambda = 1,3 \). We may recall that a total plastic stress distribution would give \( \lambda = 1,5 \).

**CREEP EQUATIONS**

**Dislocation behavior implications in ice**

From the theory of dislocations, several relations can be used in the case of ice behavior.

If we call \( n \) the density of mobile dislocations and \( v \) their velocity of displacement, the plastic shear rate is given by (Orowan 1934):

\[ \dot{\gamma} = b n v \]  \[10\]

where \( b \) is the Burgers vector found to be (Glen 1975) equal to 0,15 nm.

The velocity \( v \) has been computed for the climb dislocation mechanism, which is most often encountered in ice as: \( v_0 = 10^{-4} \) cm s\(^{-1}\) for a shear stress, \( \tau = 10^5 \) Pa at \( \theta = 0^\circ \) C (Michel 1978).
Friedel (1956) has shown that the velocity of climb is related to the stress field by \( v = D \tau \) where \( D \), being the coefficient of diffusion, can be expressed, with Arrhenius equation, by:

\[
D = D_0 \exp \left[ -\frac{Q}{RT} \right]
\]

where \( Q = \) activation energy in cal/mole K
\( R = \) universal gas constant 1,9862 cal/mole
\( T = \) temperature in K.

However, the number of mobile dislocations, has been empirically related to the deformation (Michel 1978) by the following expression:

\[
n = n_0 \left( 1 + \alpha \gamma^m \right) \left( \frac{\tau}{\tau_0} \right)^2
\]

where \( n_0 = \) initial number of dislocations
\( \alpha = \) coefficient of multiplication of dislocations
\( m = \) characteristic power of multiplication of mobile dislocations.

Combining the previous equations \([10],[11]\) and \([12]\) in the fundamental Orowan's equation we obtain (Michel 1978):

\[
\dot{\gamma} = b n_0 v_0 \left( 1 + \alpha \gamma^m \right) \left( \frac{\tau}{\tau_0} \right)^3 \exp \left[ -\frac{Q}{273 RT} \right]
\]

with the value of \( b \) and \( n_0 \) proposed earlier we finally get the general form of the creep law:

\[
\dot{\gamma} = 1.5 \times 10^{-12} n_0 \left( 1 + \alpha \gamma^m \right) \left( \frac{\tau}{\tau_0} \right)^3 \exp \left[ -\frac{Q\theta}{273 RT} \right]
\]

This expression can be simplified, for the steady state creep, to the following one:

\[
\dot{\gamma} = 1.5 \times 10^{-12} n_t \left( \frac{\tau}{\tau_0} \right)^3 \exp \left[ -\frac{Q\theta}{273 RT} \right]
\]

If we call \( n_t \) the total number of dislocation, thus justified by the observation of the creep rate which becomes constant after a deformation \( \gamma_t \). This total number of dislocations is then, for a reference stress \( \tau_0 \):

\[
n_t = n_0 \left( 1 + \alpha \gamma_t^m \right)
\]
The creep law for shear [15] can be transformed to be used when a normal stress $\sigma$ and a longitudinal strain $\varepsilon$ are involved (reference stress $\sigma_0$ must then be taken as equal to $2\tau_0$).

We have been able, with this creep law, to compute the initial and total numbers of dislocations for the ice of our beams. Results are shown in table 2 and compared to results computed (Michel 1978) from data of various authors.

<table>
<thead>
<tr>
<th>Ice type</th>
<th>Authors</th>
<th>$n_0$ (cm$^{-2}$)</th>
<th>$n_t$ (cm$^{-2}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>glacier ice</td>
<td>Duval 1976</td>
<td>$2.6 \times 10^2$</td>
<td>$2.9 - 3.4 \times 10^3$</td>
</tr>
<tr>
<td>$S_2$</td>
<td>Paradis 1978</td>
<td>$2.2 \times 10^3$</td>
<td>$2.4 - 4.6 \times 10^4$</td>
</tr>
<tr>
<td>$S_2$</td>
<td>Our study</td>
<td>$4 \times 10^2 - 1 \times 10^3$</td>
<td>$4 \times 10^3 - 8 \times 10^4$</td>
</tr>
</tbody>
</table>

Table 2 - Initial and total number of mobile dislocations in similar types of ice.

Creep model

Gagnon's experiments (1978) on ice plates showed that during creep, the deformation evolution at any point of a plate, simply loaded on its center, was affected by the same time function.

In our beam bending we can say that:

$$\varepsilon(x,t) = \varepsilon_0 \cdot f(t)$$  \[17\]

where $f(t) = 1$ when $t = 0$ at the beginning of loading. Using variables defined in fig. 6 we can write:

$$\frac{d^2 \omega}{dx^2} = \frac{1}{\rho} \cdot \frac{\varepsilon(x,t)}{h/2}$$  \[18\]

This leads, for the four points loaded beam, to the expression of the central deflection:

$$\omega = \frac{pL^3}{56.4EI} \cdot f(t)$$  \[19a\]

or

$$\omega = 0.213 \cdot \frac{p^2}{h} \cdot \varepsilon_0 \cdot f(t)$$  \[19b\]

when the geometry of the beam and of the loading is taken into account.

A model has been developed for shear stresses controlling the dislocations glide processes including the effect of anelasticity (Michel 1978, 1981).
\[ \dot{\varepsilon} = A \left[ 1 + \alpha \left( \frac{\varepsilon}{\varepsilon_r} \right)^m \right] \left[ 1 + \beta \left( 1 - \frac{\varepsilon}{\varepsilon_r} \right)^n \right] \left( \frac{\sigma}{\sigma_0} \right)^n \exp \left[ - \frac{Q}{RT} \right] \]  

[20]

where all parameters have a physical meaning:

\( n, Q, R \) and \( T \) = as defined above

\( A \) = a constant related to the initial number of dislocations

\( \alpha \) and \( m \) = coefficient and power of multiplication of mobile dislocations

\( \beta \) = relative importance of the anelastic deformation

\( \varepsilon_r \) = the maximum anelastic deformation

\( \varepsilon_t \) = the strain for which the number of mobile dislocations

becomes a constant.

Let's note that for \( \varepsilon \geq \varepsilon_r > \varepsilon_t \), \( \dot{\varepsilon} \) becomes the slope of permanent creep \( \dot{\varepsilon}_v \):

\[ \dot{\varepsilon}_v(t) = A (1 + \alpha) \left( \frac{\sigma}{\sigma_0} \right)^n \exp \left[ - \frac{Q}{RT} \right] = \dot{\varepsilon}_v \]  

[21]  

The value of \( \beta \) can be found from the slope at the origin for \( \dot{\varepsilon} = 0 \):

\[ \dot{\varepsilon}_o = A (1 + \beta) \left( \frac{\sigma}{\sigma_0} \right)^n \exp \left[ - \frac{Q}{RT} \right] \]

thus:

\[ \frac{(1 + \alpha)}{(1 + \beta)} = \frac{\dot{\varepsilon}_v}{\dot{\varepsilon}_o} \]  

[22]

Since we showed that the central deflection of the beam was directly related to the deformation at extreme fibers, we can assume that the deflection rate is related to the deflection by a similar expression such as:

\[ \frac{d\bar{\omega}}{dt} = (1 + \bar{\alpha} \bar{\omega}) \left[ 1 + \beta (1 - \bar{\omega})^n \right] \]  

[23]

where the reduced variables are:

\( \bar{\omega} = \frac{\omega}{\omega_r} \); \( \omega_r \) being the total anelastic deflection during transitory creep

\[ \bar{t} = \frac{\dot{\omega} \dot{\omega}_v}{(1 + \alpha) \omega_r} \); \( \dot{\omega}_v \) being the secondary deflection rate

\( \bar{\alpha} = \alpha \left( \frac{\omega \dot{\omega}}{\omega_t} \right) \); \( \omega_t \) being the deflection at the beginning of steady creep

in these expressions, \( \dot{\omega}_v \) is an equivalent steady creep deflection rate.
Equation [23] has been used to represent our creep deflection curves (figs 2 and 3) in their first loading period. It is shown in full lines on the figures. Parameters have been chosen to make the equation fit the curve: n has been kept equal to 3 in all cases and parameters values which have been used vary for m from 0,3 to 0,5 and for $\bar{a}$ from 3 to 7.

For comparisons purposes we have adjusted this equation [23] to Gagnon's plates deformation (1978).

The ice used for these plates was of the same $S_2$ type as ours and the curves considered are that of the central deflection of the plate.

It is important to notice that the values of the parameters remain in the same order of magnitude than that used for the beams. Table 3 gives the value of the parameters used for each curve. They vary for m from 0,3 to 0,5 and for $\bar{a}$ from 2 to 12 which is rather a narrow range for ice whose dislocations characteristics differ much from one sample to the others.

<table>
<thead>
<tr>
<th></th>
<th>m</th>
<th>$\bar{a}$</th>
<th>$\omega_v$ (mm s$^{-1}$)</th>
<th>$\omega_o$ (mm)</th>
<th>$\omega_r$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td># 2</td>
<td>0,5</td>
<td>5</td>
<td>2,1x10$^{-4}$</td>
<td>0,25</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0,5</td>
<td>5</td>
<td>2,8x10$^{-5}$</td>
<td>1,40</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0,5</td>
<td>3</td>
<td>3,8x10$^{-5}$</td>
<td>0,76</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0,4</td>
<td>7</td>
<td>3,2x10$^{-5}$</td>
<td>0,76</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>0,3</td>
<td>7</td>
<td>4,3x10$^{-5}$</td>
<td>0,89</td>
</tr>
<tr>
<td>Plates</td>
<td># 2</td>
<td>0,3</td>
<td>10</td>
<td>9,7x10$^{-6}$</td>
<td>0,90</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0,5</td>
<td>7</td>
<td>1,7x10$^{-5}$</td>
<td>2,50</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0,3</td>
<td>2</td>
<td>2,2x10$^{-5}$</td>
<td>3,25</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0,3</td>
<td>7</td>
<td>6,9x10$^{-6}$</td>
<td>0,50</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>0,5</td>
<td>12</td>
<td>1,1x10$^{-5}$</td>
<td>1,75</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0,3</td>
<td>10</td>
<td>6,9x10$^{-6}$</td>
<td>0,30</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0,5</td>
<td>2</td>
<td>5,0x10$^{-6}$</td>
<td>0,05</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>0,5</td>
<td>9</td>
<td>2,5x10$^{-5}$</td>
<td>2,6</td>
</tr>
</tbody>
</table>

Table 3 - Parameters used in equation [23]

Figure 7 shows how good is the fit of the equation [23] adjusted to the data obtained by Gagnon (1978).
CONCLUSION

The linearity of strain along a cross section have allowed us to propose a stress distribution and an effective moment evaluation. The creep model involving the dislocations glide and climb processes can be applied successfully in the case of ice beams as well as for ice plates to give a general method to predict time dependant deflections of ice covers.

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DIFFERENT KINDS OF REINFORCING MATERIALS HAVE BEEN TESTED IN ORDER TO FIND OUT THE POSSIBLE INCREASE IN THE BEARING CAPACITY AND DUCTILITY OF AN ICE-COVER. A NUMBER OF FIFTY ICE-BEAMS HAVE BEEN TESTED BOTH IN THE FIELD AND IN THE LABORATORY. IT HAS BEEN SHOWN THAT BY RATHER SIMPLE MEANS, FOR INSTANCE BY WOODEN REINFORCEMENT, IT IS POSSIBLE TO INCREASE THE BEARING CAPACITY ABOUT THREE TIMES AND THE DUCTILITY UP TO TWENTY TIMES. THE ICE-COVER CAN BE SUPPLIED WITH REINFORCEMENT BOTH IN THE UPPER SURFACE AND IN THE LOWER SURFACE. IT HAS MOREOVER BEEN SHOWN THAT RATHER HIGH TENSILE STRESSES IN THE REINFORCEMENT CAN BE UTILIZED WHEN THE ICE-COVER IS LOADED ABOVE THE CRACKING STAGE. AT ULTIMATE LOAD, TENSILE STRESSES IN STEEL-REINFORCEMENT ARE FOUND TO BE IN THE VICINITY OF 400 MPa. SHEAR OR HEAVY BOND SLIP CAN BE THE CAUSE OF FAILURE WHEN TOO LARGE AMOUNT OF REINFORCEMENT IS USED.

IN ORDER TO GET MORE ACCURATE DESIGN RECOMMENDATIONS, A NUMBER OF ICE-BEAMS HAVE BEEN STUDIED IN THE LABORATORY. THE STRESS-DISTRIBUTION IN THE COMPRESSION ZONE AND ITS CHANGE WITH THE LOADING TIME HAVE BEEN STUDIED IN SOME CREEP-TESTS. A BENDING THEORY THAT INCLUDES THE EFFECT OF CREEP IN THE COMPRESSION ZONE HAS BEEN DERIVED. THE AGREEMENT BETWEEN THEORY AND EXPERIMENT IS SUFFICIENTLY GOOD.
INTRODUCTION

The possibility to increase the bearing capacity of an ice cover with different kinds of reinforcement has been investigated for some time at the Division of Structural Engineering, University of Luleå, Sweden. Preliminary investigations [1] have shown that it is possible to increase the bearing capacity far above the ultimate load for unreinforced ice and, what is perhaps equally important, the ductility can be increased up to ten times if a proper amount of reinforcement is provided. For service loads the creep-properties are extremely important and it was found necessary to study both experimentally and theoretically the behaviour of reinforced ice-beams subjected to sustained loading. To the knowledge of the author no creep-tests of cracked reinforced ice-beams have been reported and the main interest is thus here concentrated to the behaviour in a cracked stage.

TEST PROGRAM

This investigation consists of five beams with dimensions and reinforcement arrangements according to figure 1. The reinforcement consists of two deformed steel-bars

![Test-beams diagram](image)

Fig 1  Test-beams

φ8 with a yield-strength of $\sigma_y = 400$ MPa. The ice-beams were made of fresh water in an interior mould of polyeten-plastics. This interior mould was placed in an exterior mould of plywood and the space between the moulds was filled with 70 mm insulation in order to simulate natural conditions when freezing starts from the surface. The freezing of the beams took place at a temperature of $\theta = -30^\circ$C. Together with the beams a number of prisms were made in order to test the compression strength. It seems that the properties of the ice are close to those of columnar $S_2$-ice [2].

The test-beams have been divided into two series where the level of the sustained load and the temperature have been varied. The bending moment $M_c$ and the temperature
at the sustained loading-test are summarized in table 1 below. The cracking moment $M_{cR}$ and the ultimate moment $M_u$ of the beams are also given in table 1.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\Theta$</th>
<th>$M_c$</th>
<th>$M_{cR}$</th>
<th>$M_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kNm</td>
<td>kNm</td>
<td>kNm</td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>-10</td>
<td>3.50</td>
<td>2.50</td>
<td>4.50</td>
</tr>
<tr>
<td>B2</td>
<td>-10</td>
<td>3.50</td>
<td>2.50</td>
<td>4.50</td>
</tr>
<tr>
<td>B3</td>
<td>-10</td>
<td>3.50</td>
<td>2.50</td>
<td>5.00</td>
</tr>
<tr>
<td>B4</td>
<td>-15</td>
<td>2.50</td>
<td>2.00</td>
<td>4.00</td>
</tr>
<tr>
<td>B5</td>
<td>-15</td>
<td>2.50</td>
<td>1.50</td>
<td>4.00</td>
</tr>
</tbody>
</table>

It is thus clear that the creep-tests correspond to a fully cracked stage. The beams were loaded with hydraulic jacks according to figure 2. The load was increased in equal steps corresponding to an increase of the bending moment of 0.5 kNm. At each loading step the strains at six different levels were measured on both sides of the beam in the mid section.

From these measurements the curvature and the position of the neutral axis were calculated. For each loading step also the strains in the reinforcement bars were measured by strain gages fitted to the bars. During the sustained loading the same measurements were made in intervals of five minutes during half an hour. Thereafter, the load was increased in equal steps to failure. The failure mode could in most cases be described as bending-shear-failure. The stresses in the reinforcement
amounted to 320 - 400 MPa at failure. For the beams B4 and B5 the failure load was affected by a certain deterioration of the anchorage-zone. It seems that the ice at $\Theta = -15^\circ \text{C}$ is more sensitive to the thermal incompatibility with regard to the reinforcement than the ice at $\Theta = -10^\circ \text{C}$.

THEORETICAL ANALYSIS

During the increase of the load to the level corresponding to the sustained loading, the behaviour of the beam was considered elastic with due regard to cracking. However, limitations of the compressive stresses in the compressive zone were made in agreement with some prism-tests where $\sigma_p = 1,5 \text{ to } 2,0 \text{ MPa}$ was obtained. The depth of the compressive zone $x$ thus consists of two parts $x_e$ and $x_p$ according to figure 3.

![Figure 3: Theoretical assumptions of the behaviour in the elastic stage](image)

With $\alpha = E_r/E_i$ the quotient of the modulus of elasticity for steel and ice respectively and with $b_r = \alpha \cdot A_r/b$ the equilibrium of compressive ice-stresses and the tensile force in the reinforcement can be written as:

$$x_p = \frac{2b_r (d-x_e)-x_e^2}{2(x_e+b_r)}$$

(1)

For the sake of clarity the compatibility assumptions illustrated in figure 3 and resulting in (1) are summarized below:

(a) Linear strain distribution

(b) No bond slip between reinforcement and ice
A complete neglect of tensile ice-stresses with regard to bending

The bending-moment equilibrium requires that the external bending moment \( M \) corresponds to:

\[
M = \sigma_p b \left[ x_p(d-0,5x_p) + 0,5x_e(d-x_p - \frac{x_e}{3}) \right]
\]  
(2)

For a given moment \( M \) the corresponding values of \( x_e \) and \( x_p \) can be solved from (1) and (2) by iteration. For the tested beams according to figure 1 \( M_c = 3,50 \) kNm corresponds to \( x_e = 1,1 \) cm and \( x_p = 5,5 \) cm assuming \( \sigma_p = 1,5 \) MPa and \( E_i = 1,0 \times 10^4 \) MPa.

For the creep-behaviour during the sustained loading it is assumed that the strain and stress development in the compressive zone can be predicted by Glen's equation for permanent creep of ice:

\[
\dot{\varepsilon} = \left( \frac{\sigma}{\sigma_c} \right)^n
\]  
(3)

The value of the reference stress \( \sigma_c \) is temperature-dependent and the exponent \( n \) depends on the structure of the ice. A decrease in temperature from \(-10^0\)C to \(-15^0\)C will approximately correspond to an increase of \( \sigma_c \) with 30 %. In the following for the theoretical analysis, a value of \( n = 3,0 \) has been chosen and all tests have been evaluated for two values of \( \sigma_c \), namely \( \sigma_c = 1,5 \times 10^8 \) and \( \sigma_c = 2,0 \times 10^8 \). (\( \sigma \) in Pa and \( \dot{\varepsilon} \) in s\(^{-1}\)). A comprehensive study of the prediction of creep in ice with Glen's equation has been given by Michel and Paradis [3] and it is referred to [3] for a more precise study.

The strains and stresses in the elastic stage thus comprises the initial values of the creep stage and according to eq. (3) the elastic behaviour is completely neglected. This is a rather rough approximation but the achieved simplicity and accuracy seem to justify it.

For a stress \( \sigma_z \) on level \( z \) according to figure 3, the strain rate \( \dot{\varepsilon}_z \) thus can be written:

\[
\dot{\varepsilon}_z = \kappa \cdot z = \left( \frac{\sigma_z}{\sigma_c} \right)^n
\]  
(4)

where \( \kappa \) = curvature.
The stress $\sigma_z$ can thus be written as:

$$\sigma_z = \sigma_c \cdot \kappa^{1/n} \cdot z^{1/n} \quad (5)$$

corresponding to a bending moment:

$$M_c = \kappa^{1/n} \cdot b \sigma_c \frac{n}{n+1} \cdot \frac{x}{n} \cdot \frac{n+1}{x} \{d - \frac{n}{z^{n+1}} \cdot x\} \quad (6)$$

If the tensile force in the steel is put equal to the resultant of the compressive stresses an expression of the maximum edge-strain $\varepsilon_{\text{max}}$ can be derived:

$$\varepsilon_{\text{max}} = \frac{b \sigma_c}{E \cdot A_r} \cdot \frac{x}{d-x} \cdot \frac{n}{n+1} \cdot \kappa^{1/n} \cdot \frac{n+1}{x} \quad (7)$$

For a sustained loading time of $t$ seconds the edge strain can also be written as:

$$\varepsilon_{\text{max}} = \varepsilon_{\text{el}} + \int_0^t \kappa x \, dt \quad (8)$$

where $\varepsilon_{\text{el}}$ is the edge strain corresponding to the elastic stage. During the loading time $t$ the depth of the compressive zone $x$ increases and the rate of the curvature $\kappa$ decreases and it is only possible to solve the problem by an incremental step procedure. Eq. (8) is thus rewritten as:

$$\sum_{r=1}^i \kappa(r) \cdot x(r) \cdot \Delta t(r) = \varepsilon(i) - \varepsilon_{\text{el}} \quad (9)$$

$$t = \sum_{r=1}^i \Delta t(r) \quad (10)$$

The procedure starts with the $x$-value corresponding to the elastic stage $x = x_0$ and runs as follows:

$$x(1) = x_0 + \Delta x \to \kappa(1) \text{ from } (6), \varepsilon_{\text{max}} \text{ from } (7)$$

$$\Delta t(1) \text{ from } (9)$$

$$x(2) = x(1) + \Delta x \to \kappa(2) \text{ from } (6), \varepsilon_{\text{max}} \text{ from } (7)$$

$$\Delta t(2) \text{ from } (9)$$

etc.
TEST RESULTS

The creep-stage is characterized by a continuous redistribution of strains and stresses and a continuous increase in the height of the compression zone. For design purpose the strain $\varepsilon_r$ in the reinforcement and the curvature $\kappa$ are of interest.

To illustrate the agreement between theory and experiment the variation of the height $x$ of the compression zone has been given in figures 4 and 5, respectively, during the sustained loading, and in figures 6 and 7 the corresponding variation of the curva-

---

Fig 4 Increase of the depth $x$ of the compression zone with time.
$\theta = -10^\circ C$. $M_c = 3,50$ kNm.

Fig 5 Increase of the depth $x$ of the compression zone with time.
$\theta = -15^\circ C$. $M_c = 2,50$ kNm.

---

Fig 6 Increase of curvature $\kappa$ with time. $\theta = -10^\circ C$. $M_c = 3,50$ kNm.

Fig 7 Increase of curvature $\kappa$ with time. $\theta = -15^\circ C$. $M_c = 2,50$ kNm.
ture $k$ is presented. The variations of the strain $\varepsilon_r$ are to be seen in figures 8 and 9. It is to be noticed that during the time of load-increase (about twenty minutes) some creep has taken place and the experimental values have been presented as if the whole sustained load has been acting during all the time.

Fig 8 Increase of reinforcement strain $\varepsilon_r$ with time. $\theta = -10^\circ$C. $M_c = 3,50$ kNm.

Fig 9 Increase of reinforcement strain $\varepsilon_r$ with time. $\theta = -15^\circ$C. $M_c = 2,50$ kNm.
The overwhelming impression from these figures is the great creep-deflections that occur during a rather short time of about half an hour. See figures 6 and 7. The deflections have increased to at least twice the elastic values. In spite of this the strain increase in the reinforcement is rather moderate which is important for the design of the reinforcement. See figures 8 and 9.

Concerning the agreement between theory and experiment it seems that the theory is accurate enough for practical design purpose. On account of the fact that tensile stresses are neglected in the theoretical analysis, it is natural that the experimental values of $x$ are greater than the theoretical ones and this also explains some discrepancy between theoretical and experimental values of $\varepsilon_r$. This discrepancy is, however, mostly on the safe side.

SUMMARY AND CONCLUSIONS

A theoretical analysis of the creep behaviour of ice-beams in a cracked stage has been outlined. The theory has been used to evaluate a few experimental creep tests and the agreement between theory and experiment seems sufficiently accurate for design purpose. It is obvious from this study that creep cannot as a rule be neglected by design of structural reinforced ice covers. It seems also appropriate that the effect of creep on the ultimate bending moment for reinforced ice covers is analysed to some extent.

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THE REACTION OF A FLOATING ICE SHEET TO SIMPLE LOADS
AND CERTAIN CLASSES OF VEHICLES AND MACHINES.

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                                               Fairbanks AK 99701

ABSTRACT

A floating ice sheet will support a vehicle or other load but is affected by
and reacts to the load. It is sensitive to two aspects of the load: its weight,
and the pattern of loading. The effect of weight is well understood but that of
the pattern of loading is not. Several loading patterns, each with a gross weight
of 10,000 lbs, were compared. The maximum tensile stress they would generate in an
ice sheet 12 inches thick ranged from 176 to 76 pounds per square inch demonstrating
that the pattern of loading does have a strong effect on the ice.

The tensile stress under a load can be calculated using Thin Plate analytical
techniques. It is affected by the thickness of the ice so it is calculated for
several thicknesses. The resulting ice thickness-tensile stress or ice response
table can then be converted to a Bearing Constant value.

Ice reactions, as Bearing Constant values, can be found for a number of typical
machines or vehicles of a group that varies only in a few characteristics such as
size and weight. Using standard statistical techniques and the proper independent
variables, a general function can be found for the entire class.

General functions were found for the Bearing Constant values of bulldozers and
similar tracked machines and for cars and light trucks. In both cases, the Bearing
Constant values are proportional to the square root of the gross weight and inversely proportional to the area of ice involved in direct support of the load.

Thin Plate solutions, Bearing Constant values and some of the techniques de-
scribed in this paper put ice travel of a rational engineering basis
INTRODUCTION

An ice sheet reacts to two aspects of a load: its weight and the pattern of loading. The importance of weight is well known and empirical methods of calculating the ice thickness required for a load are based on the weight of the load [4,5,14]. However, the ice also reacts to the pattern of loading - a matter that has not been explored in detail. This paper will investigate the effect of the pattern of loading and show its importance.

A floating ice sheet is formed on a water surface and is supported by water pressure on the bottom of the ice. If a load is placed on the ice sheet, it is depressed at and near the load and stresses are generated in it. The important stresses are the tensile stresses in the bottom of the ice sheet since ice is weak in tension; if these stresses exceed the strength of the ice, the ice will begin to fail. Consequently, the objective is to keep the tensile stresses less than the tensile strength of the ice when putting a load on the ice.

THIN PLATE THEORY AND THE BEARING CONSTANT

Finding the Tensile Stress.

It is not feasible to measure the tensile stress in the bottom of an ice sheet directly but techniques have been developed to calculate it. The most useful approach considers the ice sheet to be a thin plate floating on the ice and analyzes its reactions to a load; consequently, it is called the Thin Plate method. The theory and mathematics are complicated but have been solved. Nevel [20] wrote a CRREL computer program to solve Thin Plate problems and more recently [17] programmed a solution on a programmable pocket calculator. The Bearing Capacity program and a variant program written by the author in BASIC were used to find the solutions in this paper.

Thin Plate Theory and Use.

Thin Plate theory is described by Nevel [15-19], Wortley [21], Kerr [12] and others. The solutions use the mechanical properties of the ice, the stiffness of the ice sheet, the support offered by the water, the ice thickness, the weight of the load and the pattern of loading to find the tensile stress in the ice. It may be necessary to investigate the stresses at several points to find the maximum tensile stresses. If solutions are found for several ice thicknesses at the point where the tensile stresses are maximum, an ice thickness-maximum tensile stress table or curve can be developed.

The studies in this paper were for cold fresh-water ice. Ice thicknesses are in inches and stress in pounds per square inch (psi). Young's Modulus is 800,000 psi and Poisson's Ratio 1/3. The loads will be moving slowly or parked for a short time so the ice reacts elastically. Assur [1] has stated that the Thin Plate solutions are "almost perfect" so the tensile stresses obtained from them will be considered to be the real
tensile stresses in the ice – there is no better information.

The John Deere Model 350 bulldozer is used to demonstrate Thin Plate techniques. This 10,500-pound (lb) machine rides on two tracks 14 inches (in) wide, separated 48 in on centers with an ice contact length of 69 in. Table 1 shows the maximum tensile stresses under this machine on cold fresh-water ice of different thicknesses. This table is a standard ice thickness-tensile stress table or ice reaction table one can obtain from a Thin Plate computer program. Figure 1 is a plot of the data.

Table 1. Ice Reaction to the JD 350

<table>
<thead>
<tr>
<th>Ice Thickness (in)</th>
<th>Max. Tensile Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>144.6</td>
</tr>
<tr>
<td>12</td>
<td>106.9</td>
</tr>
<tr>
<td>14</td>
<td>82.4</td>
</tr>
<tr>
<td>16</td>
<td>65.7</td>
</tr>
<tr>
<td>18</td>
<td>53.7</td>
</tr>
</tbody>
</table>

Table 1 and Figure 1 show that the tensile stress decreases as the ice thickness increases in a non-linear manner. One can enter the table or curve with an allowable tensile stress (140 psi is standard for cold fresh-water ice) and find the corresponding ice thickness. For 140 psi, the ice thickness required for the JD 350 is about 10 in. Thin Plate solutions of this sort are invaluable for planning or evaluating ice travel.

The Bearing Constant.

If the weight of the JD 350 is changed, perhaps by removing the bulldozer or installing a backhoe, the tensile stresses in table 1 are no longer correct. It would be necessary to find new solutions for the new weights. Similarly, new solutions would be required if the type of ice is changed. The situation would be even more difficult for a cargo-carrying vehicle since numerous solutions might be needed for different cargo and gross weights. It can be seen that Thin Plate solutions could lead to large numbers of tables or curves.

Johnson [9] has shown that an ice reaction table can be described by a power curve which can be rearranged into the form

\[ h \sigma_t^6 = C_b \]  

(1)

where \( h \) is the ice thickness in inches and \( \sigma_t \) is the tensile stress in psi. \( C_b \) is the Bearing Constant value for the ice reaction table for a particular machine or other load. The Bearing Constant value for the JD 350 is 198 units. When the Bearing Constant value for a machine is known, it can be inserted into equation (1) and used to find the stress.
for a given ice thickness or the ice thickness found for a given stress. For example, if we want to know the ice thickness giving a tensile stress of 120 psi under the JD 350 bulldozer

\[ h \sigma_t^{1.6} = 198. \quad h = C_b / \sigma_t^{1.6} = 198 / 120^{1.6} = 198 / 17.7 = 11.2 \text{ inches}. \]

Bearing Constant values will be used extensively in this paper so their properties should be described. Equation (1) is an empirical relationship but it fits most ice reaction tables very well - well enough that it may reflect a fundamental relationship for load-bearing ice. They have the advantage that they are independent of ice thickness. Each value applies to the vehicle and ice used in the original table.

**ICE REACTIONS TO SIMPLE LOADING PATTERNS**

A gross weight of 10,000 lbs was used to study ice reactions to simple loading patterns. This weight was placed on an ice sheet as a single load and then as two, three and four loads. Each load was applied by a pneumatic tire with a tire pressure of 50 psi and the multiple loads were separated by an arbitrary 80 in. Table 2 shows the Bearing Constants, ice thickness giving a tensile stress of 140 psi (\( h_{140} \)), and tensile stress developed in 12 in of ice (\( \sigma_{12} \)) for each load. Other loads using the same gross weight but approaching and then representing a standard light truck were used. Finally, ice reactions under the JD 350 are included.

<table>
<thead>
<tr>
<th>Load</th>
<th>( C_b )</th>
<th>( h_{140} )</th>
<th>( \sigma_{12} )</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>267</td>
<td>13.8</td>
<td>176.0</td>
<td>Single 10,000-lb load.</td>
</tr>
<tr>
<td>2</td>
<td>222</td>
<td>11.4</td>
<td>129.5</td>
<td>Two 5,000-lb loads @ 80 in.</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>10.3</td>
<td>108.8</td>
<td>Three 3,333-lb loads @ 80 in.</td>
</tr>
<tr>
<td>4</td>
<td>180</td>
<td>9.3</td>
<td>91.3</td>
<td>Four 2,500-lb loads, 80 X 80 in rectangle.</td>
</tr>
<tr>
<td>5</td>
<td>173</td>
<td>9.0</td>
<td>85.5</td>
<td>Pickup, 65 X 131 in, 2,500-loads each wheel.</td>
</tr>
<tr>
<td>6</td>
<td>161</td>
<td>8.3</td>
<td>75.8</td>
<td>Load 5, tire pressure 5 psi.</td>
</tr>
<tr>
<td>7</td>
<td>184</td>
<td>9.5</td>
<td>94.7</td>
<td>Load 5, 60% of weight on rear wheels.</td>
</tr>
<tr>
<td>8</td>
<td>198</td>
<td>10.2</td>
<td>107.0</td>
<td>JD 350, 10,000 lbs on two tracks.</td>
</tr>
</tbody>
</table>

Examining loads 1-4, the Bearing Constant, \( h_{140} \) and \( \sigma_{12} \) all decrease as the number of loads increase. The ice required to support the same weight decreases from 13.8 to 9.3 inches. The stress generated in ice 12 inches thick also drops from 176.0 to 91.3 psi - a decrease of about 50%. There is no question but that the pattern of loading has a strong effect on the ice reaction to the load.

Load 5 with the dimensions of a pickup truck generates about the same reaction as the hypothetical vehicle of load 4. When it is mounted on low-pressure tires, the ice reaction drops slightly. When the load is distributed unequally, the ice reaction
increases. Finally, the ice reaction to the JD 350 is not particularly low. Bearing Constant values range from 267 to 161 units and are proportional to the ice thickness required at a constant tensile stress.

If one wished a vehicle with a minimum ice response, he would select load 6, the pickup with equal loads on the four wheels and with low-pressure tires. However, the load-spreading effect of the low-pressure tires does not demonstrate a large benefit over the hard wheels of load 5 and the JD 350 of load 8 does not demonstrate that the load-spreading effect of tracks is of much value. Load-spreading running gear is designed for soft surfaces (i.e., sand) but an ice sheet provides a hard travel surface. The ice sheet is still sensitive to the presence of the load even when load-spreading running gear are used.

Loads 1-4 show that important advantages are gained when a load is divided up into a number of smaller spaced loads. This type of information can be used in designing vehicles and machines for use on ice where the ability to travel on thinner ice is an advantage.

ICE REACTIONS TO CLASSES OF VEHICLES

While the ice reaction to an individual machine - such as the JD 350 - is of interest and value to the owner of the bulldozer, it would be of much greater value if general solutions could be found for entire classes of machines and vehicles. A class would be a number of generally similar machines or vehicles which would vary in some characteristics such as size and weight. Two classes were investigated - medium-weight and heavy bulldozers and similar tracked machines, and cars and light trucks.

**Tracked Machines.**

Thirteen standard bulldozers ranging from the Caterpillar D3 weighing 14,000 lbs to the giant Allis Chalmers HD41 with a mounted ripper, weighing 140,000 lbs, were examined [8]. Each had the narrowest tracks and track spacing available for that model and was the type a contractor would normally have for dirt work. Each had a mounted bulldozer blade. The weight of a bulldozer can be changed substantially by mounting additional equipment such as winches and rippers. This problem was met by treating them as separate machines when the mounted equipment was changed. Bearing Constant values were calculated for each machine and a least squares fit of the logarithmic values of Bearing Constants and gross weights gave the equation

$$C_b = 1.61 W^{-0.52}$$

(2)

where W is the machine weight in pounds. The correlation coefficient, r, was 0.99 and the standard error of the estimate 2.5 units. This showed that the bulldozers could be treated as a class. It also showed that the ice reaction is proportional to
the square root of the weight (this has been believed for many years and is the basis of empirical solutions of the load-ice thickness problem).

Equation (2) overestimated the ice reaction to a number of other tracked machines and vehicles such as a low ground pressure bulldozer, the Army M113 personnel carrier and the M60 tank by 5-10 percent. These machines had larger track areas and track separations than bulldozers of the same weight. The effect of larger track contact areas was discounted but the fact that they used larger ice support areas due to the greater track separation distances was investigated. A new function, Gross Loaded Area (GLA) was developed to quantify the ice support area. For the tracked machines, it was defined as the entire ice area under and between the tracks: the area, in square feet of the rectangle defined by the outer edges of the tracks. A new least squares equation was found:

$$C_b = 2.31 W^5 - 6.05 \text{GLA} - 23$$  \hspace{1cm} (3)

with a correlation coefficient, $r$, of 0.999 and standard error of the estimate of 2.3 units. The coefficient of GLA was negative meaning that increasing the support area decreases the ice reaction. This is also the effect found earlier with the simple loads.

$C_b$ values in the above study ranged from 200 to 672 units so an uncertainty of 2 units is inconsequential. The study showed that the ice reaction for these machines is proportional to the square root of the weight and inversely proportional to the area of ice support. Finally, equation (3) is a general solution for this class of machine, developed from Thin Plate solutions, that can be used without further computer solutions by inserting the $C_b$ value from equation (3) into equation (1).

**Cars and Light Trucks.**

A somewhat similar study was made of cars and light trucks. The important independent variables were again found to be $W^5$ and GLA. The fitting equation was [11]

$$C_b = 2.2 W^5 - 0.26 \text{GLA} - 18$$  \hspace{1cm} (4)

with GLA defined as the area within the polygon defined by the centers of the four wheels. The study was restricted to vehicles with four wheels. The correlation coefficient, $r$, was 0.999 and the standard error of the estimate was 2.5 units. The equation is good for cars and light trucks weighing up to 10,000 lbs but not for vehicles with dual or tandem rear ends.

**HEAVY TRUCKS**

A study was made of the ice reactions to heavy trucks but it proved difficult to group them into a single class since they are "variable load" vehicles. The tracked bulldozers and other machines and vehicles were "constant weight" machines designed...
for various types of work rather than for carrying cargo. Consequently, their weight and their ice reactions are fixed. A minor exception is the M113 personnel carrier whose weight ranges from 19,600 lbs with a minimum crew and load to 23,520 lbs with a full combat load. However, this moderate change in weight was allowed for by treating it as two different machines.

Almost all trucks, on the other hand, are designed and used to carry cargo so their weight may vary by a factor of 2 between the empty and fully-loaded weights. The pattern of ice reaction is complicated further since the "critical" wheel, that under which the tensile stress is greatest, will probably change as the cargo load increases. These problems are illustrated by the ice reaction pattern of the Army M35A2 2-1/2 ton cargo truck - this is the standard Army 6 X 6 [9].

With an empty weight of 13,530 lbs, the M35A2 can carry 10,000 lbs of cargo. It rides on ten wheels, 2 on the front axle and 8 on the two rear axles. Figure 2 shows the geometry of the wheels which are numbered for reference purposes. Tensile stresses were found under the left-hand wheels since those on the right would be equal due to symmetry.

<table>
<thead>
<tr>
<th>Weight (lb)</th>
<th>Tire Pressure, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empty</td>
<td>X-Country</td>
</tr>
<tr>
<td>Payload</td>
<td>0 5,000</td>
</tr>
<tr>
<td>Front Axle</td>
<td>6,450 6.645 6,840</td>
</tr>
<tr>
<td>Int. Axle</td>
<td>3,540 5,942 8,344</td>
</tr>
<tr>
<td>Rear Axle</td>
<td>3,540 5,942 8,344</td>
</tr>
<tr>
<td>Highway</td>
<td>50</td>
</tr>
<tr>
<td>X-Country</td>
<td>35</td>
</tr>
<tr>
<td>Mud</td>
<td>15</td>
</tr>
</tbody>
</table>

Figure 2. Dashplate data for M35A2 truck. Wheel numbers added for computer analysis.
In examining figure 2, two wheels on the left side of the truck are likely critical wheels. When the truck is empty, the front axle carries almost half the weight of the truck so wheel 9 is an obvious wheel to check. When the truck is loaded, the entire cargo weight is carried by the 8 wheels of the tandem dual rear end. Since the rear end has a load equalizing system, all wheels on the two rear axles will carry the same load. However, the tensile stress under a wheel is a combination of the stress generated by the wheel itself and stresses generated by other wheels and transmitted through the ice to the location of the wheel. Stresses attenuate with distance so the wheel with the closest neighbors will have the highest stresses. The inside duals have closer neighbors than the outside duals so wheels 2 and 6 are candidates. However, wheel 6 is closer to the front axle than wheel 2 so wheel 6 should be critical if any of the rear wheels ever become so.

The tensile stresses under wheels 6 and 9 were calculated for a number of cargo weights. Some were obtained from the dashplate (figure 2) and others were found by interpolation. Tensile stress and ice thickness values were converted to Bearing Constant values for the individual wheels. The data are shown in Table 3 and figure 3.

Table 3. Ice Reactions to Wheels 9 and 6, M35A2

<table>
<thead>
<tr>
<th>Cargo Wt. (lbs)</th>
<th>Vehicle Wt. (lbs)</th>
<th>Bearing Constants (Wheel 9)</th>
<th>Bearing Constants (Wheel 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>13,530</td>
<td>200*</td>
<td>181</td>
</tr>
<tr>
<td>2,500</td>
<td>16,029</td>
<td>209*</td>
<td>204</td>
</tr>
<tr>
<td>4,000</td>
<td>17,530</td>
<td>216</td>
<td>219*</td>
</tr>
<tr>
<td>5,000</td>
<td>18,529</td>
<td>--</td>
<td>228*</td>
</tr>
<tr>
<td>7,500</td>
<td>21,028</td>
<td>--</td>
<td>250*</td>
</tr>
<tr>
<td>10,000</td>
<td>23,528</td>
<td>--</td>
<td>270*</td>
</tr>
</tbody>
</table>

*Critical wheel

Table 3 and figure 3 show that wheel 9 has the highest Bearing Constant value and is the critical wheel until the payload rises to about 3,500 lbs where the two wheels have equal Bearing Constant values. With a higher payload, wheel 6 becomes critical. The pattern of Bearing Constant for this and other trucks is that the Bearing Constant is a dog-legged function of payload and gross weight. Since the ice reaction to a single truck is of this form, it is difficult to combine the ice reactions of a number of trucks into a general function with the necessary accuracy.

Figure 3. Ice Reactions to Wheels 9 and 6, M35A2 with Changes in Cargo Weight.
LIMITATIONS OF THIN PLATE SOLUTIONS

Thin Plate solutions are for "perfect" ice sheets; those that are isotropic, homogeneous, elastic and infinite. There are few infinite ice sheets so the practical meaning of this term is that irregularities and abnormalities in an ice sheet are sufficiently distant from the load that they do not affect the behavior of the ice under the load. There are many conditions in naturally-occurring ice sheets that depart from the "perfect" so that engineering judgment may be required.

CONCLUSIONS

A load placed on a floating ice sheet will develop stresses in the ice. If the tensile stresses in the bottom of the ice sheet exceed the tensile strength of the ice, the ice will begin to fail. Two factors, the weight of the load and its pattern of loading, control the magnitude of the stresses in the ice. The effect of weight is well understood but the effect of the pattern of loading is not as well known.

Tensile stress levels in the ice can be calculated using a precise Thin Plate analytical technique which develops a table of ice thickness-maximum tensile stress for the load and the ice. This ice response table can be converted to a Bearing Constant value using a rearranged power curve of the form

$$ h\sigma_t^{0.6} = C_b $$

where \( C_b \) is the unique Bearing Constant for the ice response table and thus for the given load and ice conditions. Bearing Constant values are measures of ice response independent of ice thickness and are convenient for the analysis of ice response.

A number of simple loads with a gross weight of 10,000 lbs were examined and their ice responses found. The ice response dropped rapidly as the original single load was divided into several spaced smaller loads. This analysis shows that the loading pattern on an ice sheet strongly affects the ice reaction. Load-spreading techniques using low-pressure tires or tracks are not very effective in reducing the ice reaction since the ice sheet is still sensitive to the presence and magnitude of the load(s).

General equations were developed for two classes of machines and vehicles - tracked bulldozers and similar machines, and cars and light trucks. Ice responses were found to be a function of the square root of the weight of the load and to the gross loaded area of the ice used to directly support the vehicle. These general equations make it possible to find ice responses to individual machines and vehicles of the classes with approximately Thin Plate accuracy without requiring individual computer solutions.
REFERENCES

11. Johnson, P.R. 1980c. A guide for operating cars and light trucks on a floating ice sheet using thin plate analytical solutions. Published by Phil Johnson Engineering, Fairbanks AK.